NCSEA Structural Engineering Exam Review Course

Vertical Forces Review

Steel Design – Spring 2017

Presented by Rafael Sabelli, S.E.

Outline

- General steel design
  - AISC 360: Specification for Structural Steel Buildings
- Design problems
  - From Alan Williams, PhD, SE

    Structural Engineering

    PE License Review Problems & Solutions 8th Ed.
- Systems covered under "Lateral Forces Review"

General Steel Design

- Tension members
- Compression members
- Base plates
- Beams
- Trusses
- Beam-columns
- Connections—welded
- Connections—bolted
- Moment connections
- Weld design
- (Composite steel design)

Design Problems

- Members
  - Tension
  - Compression
  - Flexure
  - Combined compression and flexure
- Connections
  - (Bolts)
  - Welds
  - Connecting elements
  - Concentrated forces on members

AISC 360

Specification for Structural Steel Buildings

AISC 360

A. General Provisions
B. Design Requirements
C. Design for Stability
D. Design of Members for Tension
E. Design of Members for Compression
F. Design of Members for Flexure
G. Design of Members for Shear
H. Design of Members for Combined Forces
I. Design of Composite Members
J. Design of Connections
K. Design of HSS and Box Member Connections
L. Design for Serviceability
M. Fabrication and Erection
N. Quality Control and Quality Assurance

AISC 360 (Appendices)

1. Design by Inelastic Analysis
2. Design for Ponding
3. Design for Fatigue
4. Structural Design for Fire Conditions
5. Evaluation of Existing Structures
6. Stability Bracing for Columns and Beams
7. Alternative Methods of Design for Stability
8. Approximate Second-Order Analysis

A. General Provisions

A1. Scope
- 1. Seismic Applications
- 2. Nuclear Applications

A2. Referenced Specifications, Codes, and Standards

A3. Material
- 1. Structural Steel Materials
  - 1a. ASTM Designations
  - 1b. Unidentified Steel
  - 1c. Rolled Heavy Shapes
  - 1d. Built-Up Heavy Shapes
- 2. Steel Castings and Forgings
- 3. Bolts, Washers, and Nuts
- 4. Anchor Rods and Threaded Rods
- 5. Consumables for Welding
- 6. Headed Stud Anchors

A4. Structural Design Drawings and Specifications

B. Design Requirements

B1. General Provisions

B2. Loads and Load Combinations

B3. Design Basis

1. Required Strength
2. Limit States
3. Design for Strength Using LRFD
4. Design for Strength Using ASD
5. Design for Stability
6. Design of Connections
   - 6a. Simple Connections
   - 6b. Moment Connections
7. Moment Redistribution in Beams .
8. Diaphragms and Collectors
9. Design for Serviceability
10. Design for Ponding
11. Design for Fatigue
12. Design for Fire Conditions
13. Design for Corrosion Effects
14. Anchorage to Concrete

B4. Member Properties

1. Classification of Sections for Local Buckling
2. Design Wall Thickness for HSS
3. Gross and Net Area

B5. Fabrication, Erection, and Quality Control

B6. Evaluation of Existing Structures

Design Basis

For LRFD:

$$R_u \le \phi R_n$$

where

R_u = required strength (LRFD)

R_n = nominal strength specified in Chapters C through K

ϕ = resistance factor specified in Chapters C through K

ϕR_n = design strength

Design Basis

For ASD:

$$R_a \leq R_n/\Omega$$

where
R_a = required strength (ASD)
R_n = nominal strength specified in Chapters C through K
Ω = resistance factor specified in Chapters C through K
R_n/Ω = allowable strength

Design Basis

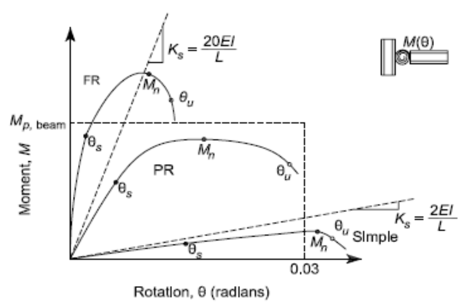


Fig. C-B3.3. Classification of moment-rotation response of fully restrained (FR), partially restrained (PR) and simple connections.

Design Basis

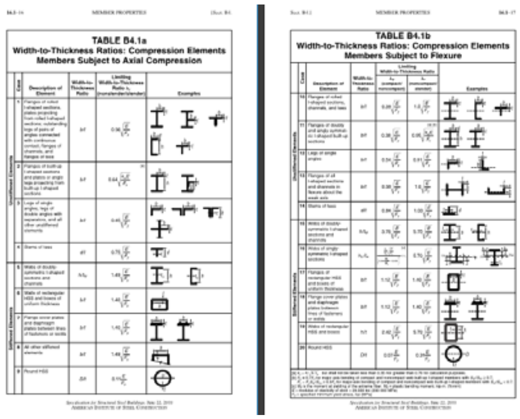
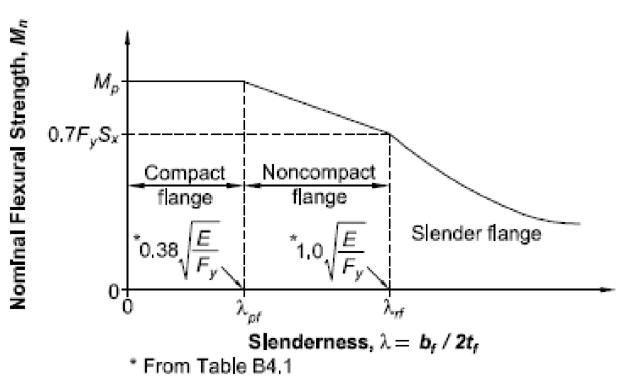


TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements
Members Subject to Axial Compression

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements
Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
stiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\frac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64\sqrt{\frac{k_cE}{F_y}}$ [a]	

Design Basis



C. Design for Stability

C1. General Stability Requirements
1. Direct Analysis Method of Design
2. Alternative Methods of Design

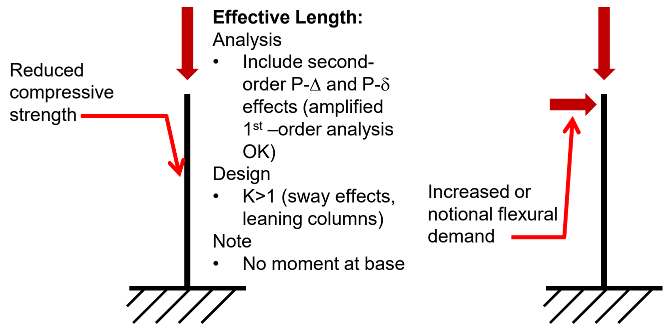
C2. Calculation of Required Strengths
1. General Analysis Requirements
2. Consideration of Initial Imperfections
   - 2a. Direct Modeling of Imperfections
   - 2b. Use of Notional Loads to Represent Imperfections
3. Adjustments to Stiffness

C3. Calculation of Available Strengths

C. Design for Stability

- Three options for analysis of stability and second-order effects
  - Second-order drift/first-order drift ≤ 1.5
    - Direct analysis method (Chapter C)
    - Amplified first-order method (Appendix 8: B1-B2)
    - Other second-order analysis
  - When second-order drift/first-order drift > 1.5
    - Direct analysis method

K factor vs Direct Analysis Method



Effective Length:
Analysis
- Include second-order P-Δ and P-δ effects (amplified 1st –order analysis OK)

Design
- K>1 (sway effects, leaning columns)

Note
- No moment at base

DAM:
Analysis
- Model imperfections (or use notional loads)
- Reduce stiffness
- Include second-order P-Δ and P-δ effects

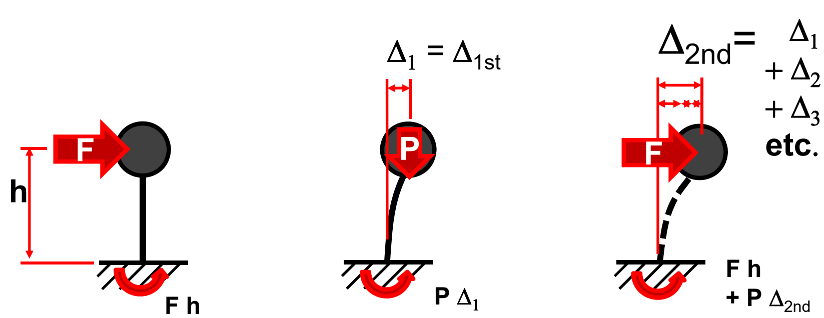
Design
- K=1

Note
- Base moment for design

Effective Length Method

Direct Analysis Method

P-Delta



1st order forces…

…cause displacement…

…leading to 2nd order effects

$$\Delta_{2nd} = \Delta_{1st}\,[1/(1 - {}^{P\Delta_{1st}}/_{Fh})]$$

Direct Analysis Method

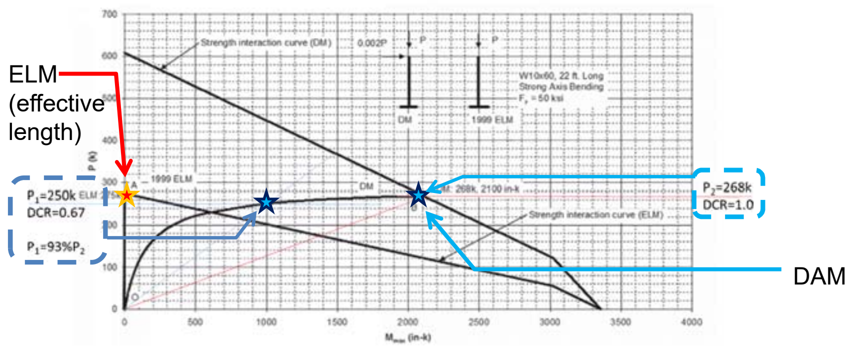


Fig. 3-1. Direct analysis method versus 1999 AISC Specification effective length method, cantilever column with axial load application to failure.

System Stability (C2)

Analysis Requirements
- Axial, Flexure, Shear deformations
- Second-order effects
- Include gravity (for correct P-Δ)
- Analyze at LRFD level

Initial imperfections

Stiffness for analysis:
- $K' = \tau_a \tau_b K$ (axial, flexural, shear)
  - $\tau_a = 0.8$ stiffness of lateral-load-resisting members
  - $\tau_b = F(P_r/P_y)$ flexural stiffness of lateral-load-resisting members

C. Stability Analysis and Design

Member Strength and Stability (C3)

- Chapters E, F, G, H, and I
- Appendix 6 (stability bracing)

Alternate Methods of Stability Analysis and Design

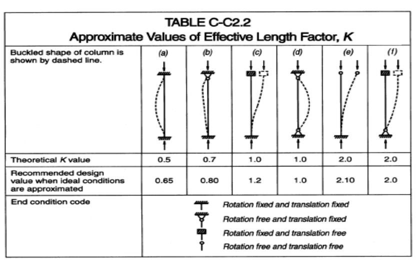
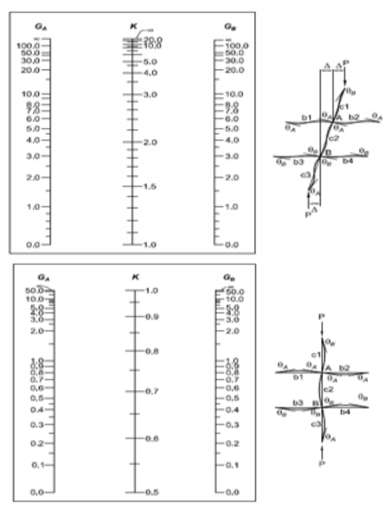


TABLE C-C2.2
Approximate Values of Effective Length Factor, K

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	Rotation fixed and translation fixed; Rotation free and translation fixed; Rotation fixed and translation free; Rotation free and translation free					



Alternate Methods of Stability Analysis and Design

$M_r = B_1 M_{nt} + B_2 M_{lt}$

$P_r = P_{nt} + B_2 P_{lt}$

$V_r = V_{nt} + B_2 V_{lt}$

$R_r = R_{nt} + B_2 R_{lt}$

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad B_2 = \frac{1}{1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \qquad \Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$

$\underline{P\delta}$ $\qquad$ $\underline{P\Delta}$

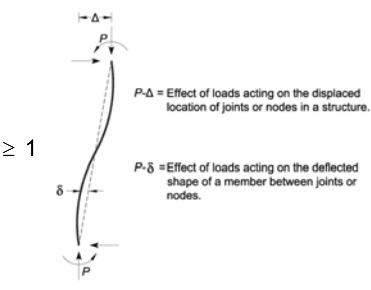


Fig. C-C2.1. P-Δ and P-δ effects in beam-columns.

Simplified Stability Analysis and Design

1. Perform 1st order analysis
   Minimum notional lateral loads
2. Find lateral load corresponding to drift limit
   OK to use less than code
3. Compute Total Gravity/Lateral Shear
   At LRFD level; ASD gravity amplified by 1.6
4. Amplify analysis forces by factor from Table 2-1
   Factors vary by story and direction
5. Determine K factors
   K=1 if factor ≤1.1
6. Check drift limit in step 2 with amplification

Simplified Stability Analysis and Design

TABLE 2-1
Multipliers for Use With the Simplified Method

3. Gravity/Lateral Shear

Design Story Drift Limit	Load Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	**5**	**10**	**20**	**30**	**40**	**50**	**60**	**80**	**100**	**120**
H/100	1	1.1	1.1	1.3	**1.5**/1.4	When ratio exceeds 1.5, simplified method requires a stiffer structure.					
H/200	1	1	1.1	1.1	1.2	1.3	**1.4**/1.3	**1.5**/1.4			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	**1.5**/1.4		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	**1.4**/1.3	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Note: Where two values are provided, the value in bold is the value associated with $R_m = 0.85$.

D. Design of Members for Tension

D1. Slenderness Limitations

D2. Tensile Strength

D3. Effective Net Area

D4. Built-Up Members

D5. Pin-Connected Members
1. Tensile Strength
2. Dimensional Requirements

D6. Eyebars
1. Tensile Strength
2. Dimensional Requirements

D. Design of Members for Tension

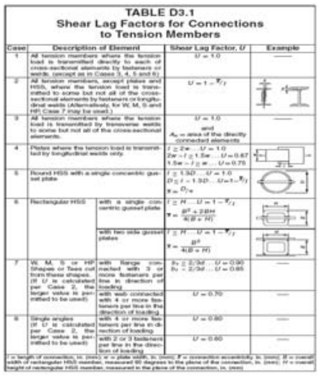
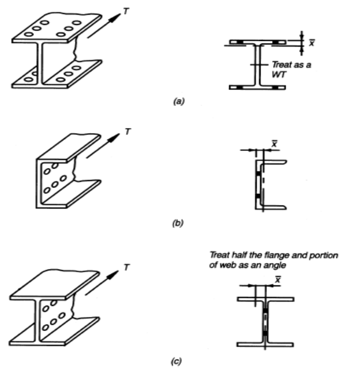


TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Example



E. Design of Members for Compression

E1. General Provisions

E2. Effective Length

E3. Flexural Buckling of Members without Slender Elements

E4. Torsional and Flexural-Torsional Buckling of Members without Slender Elements

E5. Single Angle Compression Members

E6. Built-Up Members

1. Compressive Strength
2. Dimensional Requirements

E7. Members with Slender Elements

1. Slender Unstiffened Elements, Q_s
2. Slender Stiffened Elements, Q_a

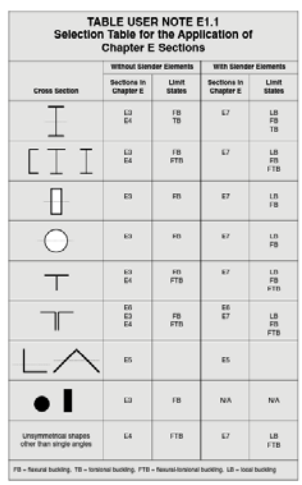


TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

Cross Section	Without Slender Elements: Sections in Chapter E	Without Slender Elements: Limit States	With Slender Elements: Sections in Chapter E	With Slender Elements: Limit States
	E3 E4	FB TB	E7	LB FB TB
	E3 E4	FB FTB	E7	LB FB FTB
	E3	FB	E7	LB FB
	E3	FB	E7	LB FB
	E3 E4	FB FTB	E7	LB FB FTB
	E6 E3 E4	FB FTB	E6 E7	LB FB FTB
	E5		E5	
	E3	FB	N/A	N/A
Unsymmetrical shapes other than single angles	E4	FTB	E7	LB FTB

FB = flexural buckling, TB = torsional buckling, FTB = flexural-torsional buckling, LB = local buckling

E. Design of Members for Compression

$Kl/r \leq 4.71\sqrt{\frac{E}{QF_y}}$ $\qquad$ $Kl/r > 4.71\sqrt{\frac{E}{QF_y}}$

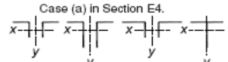
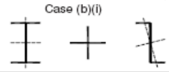
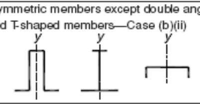
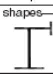
or $F_e \geq 0.44F_y$: $\qquad$ or $F_e < 0.44F_y$:

$$F_{cr} = \left[0.658^{QF_y/F_e'}\right]F_y \qquad F_{cr} = 0.877F_e'$$

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

E. Design of Members for Compression

TABLE C-E4.1
Selection of Equations for Torsional and Flexural-Torsional Buckling

Type of Cross Section	Applicable Equations in Section E4
Double angle and T-shaped members—Case (a) in Section E4. 	E4-2 and E4-3
All doubly symmetric shapes and Z-shapes—Case (b)(i) 	E4-4
Singly symmetric members except double angles and T-shaped members—Case (b)(ii) 	E4-5
Unsymmetrical shapes—Case (b)(iii) 	E4-6

F. Design of Members for Flexure

F1. General Provisions

F2. Doubly-Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis
1. Yielding
2. Lateral-Torsional Buckling

F3. Doubly-Symmetric I-Shaped Members With Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis
1. Lateral-Torsional Buckling
2. Compression Flange Local Buckling

F4. Other I-Shaped Members With Compact or Noncompact Webs Bent About Their Major Axis
1. Compression Flange Yielding
2. Lateral-Torsional Buckling
3. Compression Flange Local Buckling
4. Tension Flange Yielding

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 Local Buckling
4. Tension Flange Yielding

F6. I-Shaped Members and Channels Bent About Their Minor Axis
1. Yielding
2. Flange Local Buckling

F7. Square and Rectangular HSS and Box-Shaped Members
1. Yielding
2. Flange Local Buckling
3. Web Local Buckling

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F. Design of Members for Flexure

F8. Round HSS
1. Yielding
2. Local Buckling

F9. Tees and Double Angles Loaded in the Plane of Symmetry
1. Yielding
2. Lateral-Torsional Buckling
3. Flange Local Buckling of Tees
4. Local Buckling of Stems

F10. Single Angles
1. Yielding
2. Lateral-Torsional Buckling
3. Leg Local Buckling

F11. Rectangular Bars and Rounds
1. Yielding
2. Lateral-Torsional Buckling

F12. Unsymmetrical Shapes
1. Yielding
2. Lateral-Torsional Buckling .
3. Local Buckling

F13. Proportions of Beams and Girders
1. Strength Reductions for Members With Holes in the
Tension Flange
2. Proportioning Limits for I-Shaped
3. Cover Plates
4. Built-Up Beams
5. Unbraced Length for Moment Redistribution

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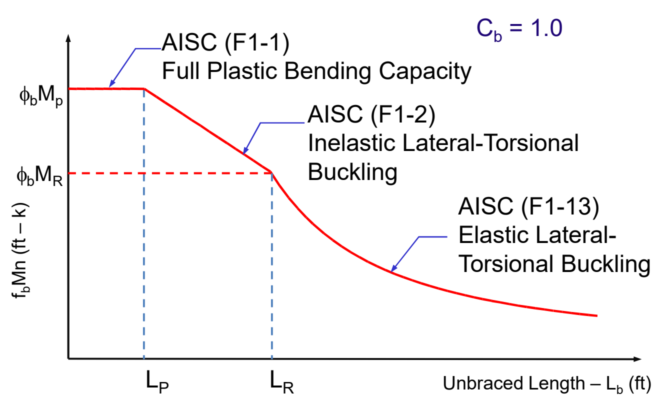
F. Design of Members for Flexure

- Limit states
  - Yielding (Y)
  - Lateral-torsional buckling (LTB)
  - Flange local buckling (FLB) and web local buckling (WLB)
    - Compact
    - Noncompact
    - Slender

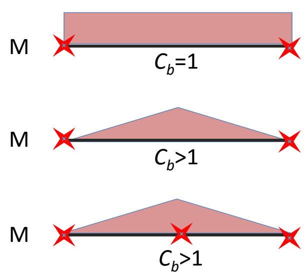
Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	NC	Y,LTB, FLB, TFY
F5		C, NC, S	S	Y,LTB, FLB, TFY
F6		C, NC, S	N/A	Y FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes	N/A	N/A	All limit states

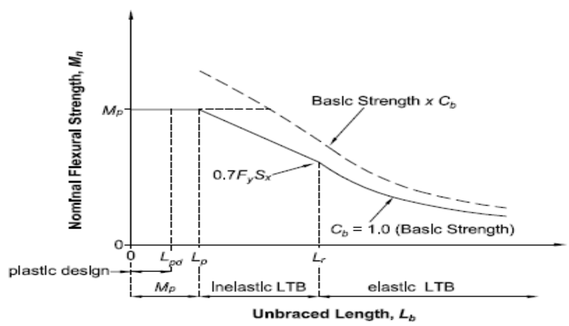
Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = Local Buckling, C = compact, NC = noncompact, S = slender

Limit States of Yielding and LTB



Limit States of Yielding and LTB





Limit States of Yielding and LTB

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
P	None Load at midpoint	1.32
	At load point	1.67 1.67
P P	None Loads at third points	1.14
	At load points Loads symmetrically placed	1.67 1.00 1.67
P P P	None Loads at quarter points	1.14
	At load points Loads at quarter points	1.67 1.11 1.11 1.67

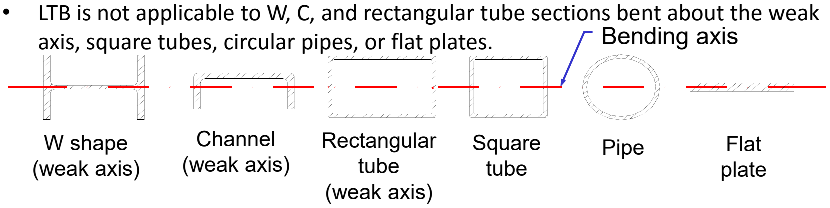
Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
w	None	1.14
	At midpoint	1.30 1.30
	At third points	1.45 1.01 1.45
	At quarter points	1.52 1.06 1.06 1.52
	At fifth points	1.56 1.12 1.00 1.12 1.56

Note: Lateral bracing must always be provided at points of support per AISC *Specification* Chapter F.

Limit States of Yielding and LTB

- Lateral-torsional buckling (LTB) cannot occur if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane.
- LTB is not applicable to W, C, and rectangular tube sections bent about the weak axis, square tubes, circular pipes, or flat plates.



Lateral-torsional buckling cannot occur in these cases.

G. Design of Members for Shear

G1. General Provisions

G2. Members With Unstiffened or Stiffened Webs

1. Shear Strength
2. Transverse Stiffeners

G3. Tension Field Action

1. Limits on the Use of Tension Field Action
2. Nominal Shear Strength With Tension Field Action
3. Transverse Stiffeners

G4. Single Angles

G5. Rectangular HSS and Box Members

G6. Round HSS

G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes

G8. Beams and Girders With Web Openings

H. Design of Members for Combined Forces and Torsion

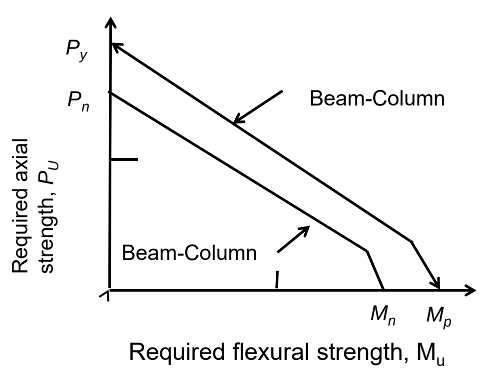
H1. Doubly and Singly Symmetric Members Subject To Flexure and Axial Force
  1. Doubly and Singly Symmetric Members in Flexure and Compression
  2. Doubly and Singly Symmetric Members in Flexure and Tension
  3. Doubly Symmetric Members in Single Axis Flexure and Compression

H2. Unsymmetric and Other Members Subject to Flexure and Axial Force

H3. Members Subject to Torsion and Combined Torsion, Flexure, Shear and/or Axial Force
  1. Round and Rectangular HSS Subject to Torsion
  2. HSS Subject to Combined Torsion, Shear, Flexure, and Axial Force
  3. Non-HSS Members Under Torsion and Combined Stress

H4. Rupture of Flanges With Holes Subject to Tension

H. Design of Members for Combined Forces and Torsion



I. Design of Composite Members

I1. General Provisions
1. Concrete and Steel Reinforcement
2. Nominal Strength of Composite Sections
   - 2a. Plastic Stress Distribution Method
   - 2b. Strain-Compatibility Method
3. Material Limitations
4. Classification of Filled Composite Sections for Local Buckling

I2. Axial Force
1. Encased Composite Columns
   - 1a. Limitations
   - 1b. Compressive Strength
   - 1c. Tensile Strength
   - 1d. Shear Strength
   - 1e. Load Transfer
   - 1f. Detailing Requirements
2. Filled Composite Columns
   - 2a. Limitations
   - 2b. Compressive Strength
   - 2c. Tensile Strength
   - 2d. Shear Strength
   - 2e. Load Transfer

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I. Design of Composite Members

I3. Flexure
1. General
   - 1a. Effective Width
   - 1b. Strength During Construction
2. Composite Beams With Steel Headed Stud or Steel Channel Anchors
   - 2a. Positive Flexural Strength
   - 2b. Negative Flexural Strength
   - 2c. Strength of Composite Beams With Formed Steel Deck
   - 2d. Load Transfer Between Steel Beam and Concrete Slab
3. Strength of Concrete-Encased and Filled Members
4. Filled Composite Members
   - 4a. Limitations
   - 4b. Flexural Strength

I4. Shear

I5. Combined Axial Force and Flexure

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SE EXAM REVIEW COURSE — March 2017

I. Design of Composite Members

I6. Load Transfer

1. General Requirements
2. Force Allocation
3. Force Transfer Mechanisms
4. Detailing Requirements

I7. Composite Diaphragms and Collector Beams

I8. Steel Anchors

1. General
2. Steel Anchors in Composite Beams
3. Steel Anchors in Composite Components

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SE EXAM REVIEW COURSE — March 2017

J. Design of Connections

J1. General Provisions

1. Design Basis
2. Simple Connections
3. Moment Connections
4. Compression Members With Bearing Joints
5. Splices in Heavy Sections
6. Weld Access Holes
7. Placement of Welds and Bolts
8. Bolts in Combination With Welds
9. High-Strength Bolts in Combination With Rivets
10. Limitations on Bolted and Welded Connections

J2. Welds

1. Groove Welds
   - 1a. Effective Area
   - 1b. Limitations
2. Fillet Welds
   - 2a. Effective Area
   - 2b. Limitations
3. Plug and Slot Welds
   - 3a. Effective Area
   - 3b. Limitations
4. Strength
5. Combination of Welds
6. Filler Metal Requirements
7. Mixed Weld Metal

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J. Design of Connections

TABLE J2.5
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{BM} or F_w) kips (N)	Effective Area (A_{BM} or A_w) in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
COMPLETE-JOINT-PENETRATION GROOVE WELDS					
Tension Normal to weld axis	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section J2.6.
Compression Normal to weld axis	Strength of the joint is controlled by the base metal				Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal				Matching filler metal shall be used.[c]
PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS					
Tension Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See J2.1a	
Compression Column to Base Plate and column splices designed per J1.4(a)	Compressive stress need not be considered in design of welds joining the parts.				
Compression Connections of members designed to bear other than columns as described in J1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60F_{EXX}$	See J2.1a	
Compression Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90F_{EXX}$	See J2.1a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
Shear	Base	Governed by J4			
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	See J2.1a	

TABLE J2.5 (cont.)
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{bm} or F_w) kips (N)	Effective Area (A_{BM} or A_w) in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[d]	See J2.2a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear Parallel to faying surface on the effective area	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	J2.3a	

[a] For matching weld metal see AWS D1.1, Section 3.3.
[b] Filler metal with a strength level one strength level greater than matching is permitted.
[c] Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80$, $\Omega = 1.88$ and $0.60\ F_{EXX}$ as the nominal strength.
[d] Alternatively, the provisions of J2.4(a) are permitted provided the deformation compatibility of the various weld elements is considered. Alternatively, Sections J2.4(b) and (c) are special applications of J2.4(a) that provide for deformation compatibility.

J. Design of Connections

J3. Bolts and Threaded Parts
1. High-Strength Bolts
2. Size and Use of Holes
3. Minimum Spacing
4. Minimum Edge Distance
5. Maximum Spacing and Edge Distance
6. Tension and Shear Strength of Bolts and Threaded Parts
7. Combined Tension and Shear in Bearing-Type Connections
8. High-Strength Bolts in Slip-Critical Connections
9. Combined Tension and Shear in Slip-Critical Connections
10. Bearing Strength at Bolt Holes
11. Special Fasteners
12. Tension Fasteners

J. Design of Connections

TABLE J3.2
Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	Nominal Tensile Stress, F_{nt}, ksi (MPa)	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310) [a][b]	24 (165) [b][c][f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	0.75 F_u [a][d]	0.40 F_u
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	0.75 F_u [a][d]	0.50 F_u

[a] Subject to the requirements of Appendix 3.
[b] For A307 bolts the tabulated values shall be reduced by 1 percent for each $^1/_{16}$ in. (2 mm) over 5 diameters of length in the grip.
[c] Threads permitted in shear planes.
[d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D, which shall be larger than the nominal body area of the rod before upsetting times F_y.
[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Appendix 3.
[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in. (1270 mm), tabulated values shall be reduced by 20 percent.

J. Design of Connections

J4. Affected Elements of Members and Connecting Elements
1. Strength of Elements in Tension
2. Strength of Elements in Shear
3. Block Shear Strength
4. Strength of Elements in Compression
5. Strength of Elements in Flexure

J5. Fillers
1. Fillers in Welded Connections
2. Fillers in Bolted Connections

J6. Splices

J7. Bearing Strength

J8. Column Bases and Bearing on Concrete

J9. Anchor Rods and Embedments

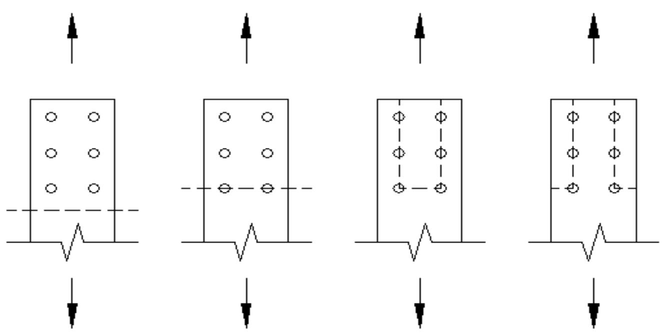
J10. Flanges and Webs With Concentrated Forces
1. Flange Local Bending
2. Web Local Yielding
3. Web Crippling
4. Web Sidesway Buckling
5. Web Compression Buckling
6. Web Panel Zone Shear
7. Unframed Ends of Beams and Girders
8. Additional Stiffener Requirements for Concentrated Forces
9. Additional Doubler Plate Requirements for Concentrated Forces

J. Design of Connections

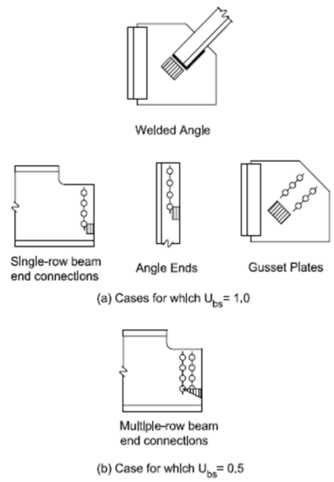
TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-slot (Width × Length)
$1/2$	$9/16$	$5/8$	$9/16 \times 11/16$	$9/16 \times 1\,1/4$
$5/8$	$11/16$	$13/16$	$11/16 \times 7/8$	$11/16 \times 1\,9/16$
$3/4$	$13/16$	$15/16$	$13/16 \times 1$	$13/16 \times 1\,7/8$
$7/8$	$15/16$	$1\,1/16$	$15/16 \times 1\,1/8$	$15/16 \times 2\,3/16$
1	$1\,1/16$	$1\,1/4$	$1\,1/16 \times 1\,5/16$	$1\,1/16 \times 2\,1/2$
$\geq 1\,1/8$	$d + 1/16$	$d + 5/16$	$(d + 1/16) \times (d + 3/8)$	$(d + 1/16) \times (2.5 \times d)$

J. Design of Connections



J. Design of Connections



Moment Connections

- Members receiving concentrated loads from moment connection are checked for applicable limit states
  - Flange local bending
  - Web local yielding
  - Web crippling
  - Web sidesway buckling
  - Web compression buckling
    - Cantilever case; compression transfer through web
  - Web panel-zone shear

Moment Connections

- A commonly missed item is the web panel-zone shear in columns (lateral loads; moment transfer to column).
  - If column $P_u <= 0.4A_gF_y$:

$$R_v = 0.60 \times F_y \times d_c \times t_w$$

  - If column $P_u > 0.4A_gF_y$:

$$R_v = 0.60 \times F_y \times d_c \times t_w \times \left(1.4 - \frac{P_u}{A_g F_y}\right)$$

  - ϕ = 0.9 and equations are assuming panel-zone deformations are not considered in analysis
  - d_c = column depth

K. Design of HSS and Box Member Connections

K1. Concentrated Forces On HSS
1. Definitions of Parameters
2. Round HSS
3. Rectangular HSS

K2. HSS-To-HSS Truss Connections
1. Definitions of Parameters
2. Round HSS
3. Rectangular HSS

K3. HSS-To-HSS Moment Connections
1. Definitions of Parameters
2. Round HSS
3. Rectangular HSS

K4. Welds of Plates and Branches to Rectangular HSS

L. Design for Serviceability

L1. General Provisions
L2. Camber
L3. Deflections
L4. Drift
L5. Vibration
L6. Wind-Induced Motion
L7. Expansion and Contraction
L8. Connection Slip

M. Fabrication and Erection

M1. Shop and Erection Drawings
M2. Fabrication
1. Cambering, Curving, and Straightening
2. Thermal Cutting
3. Planing of Edges
4. Welded Construction
5. Bolted Construction
6. Compression Joints
7. Dimensional Tolerances
8. Finish of Column Bases
9. Holes for Anchor Rods
10. Drain Holes
11. Galvanized Members

M3. Shop Painting
1. General Requirements
2. Inaccessible Surfaces
3. Contact Surfaces
4. Finished Surfaces
5. Surfaces Adjacent to Field Welds

M4. Erection
1. Alignment of Column Bases
2. Bracing
3. Alignment
4. Fit of Column Compression Joints and Base Plates
5. Field Welding
6. Field Painting

N. Quality Control

N. Quality Control

TABLE N5.4-1
Inspection Tasks Prior to Welding

Inspection Tasks Prior to Welding
Welding procedure specifications (WPSs) available
Manufacturer certifications for welding consumables available
Material identification (type/grade)
Welder identification system[1]
Fit-up of groove welds (including joint geometry) • Joint preparation • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location) • Backing type and fit (if applicable)
Configuration and finish of access holes
Fit-up of fillet welds • Dimensions (alignment, gaps at root) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)
Check welding equipment

[1] The fabricator or erector, as applicable, shall maintain a system by which a weld or member can be identified. Stamps, if used, shall be the low-stress type.

TABLE N5.4-2
Inspection Tasks During Welding

Inspection Tasks During Welding	QC	QA
Use of qualified welders	O	O
Control and handling of welding consumables • Packaging • Exposure control	O	O
No welding over cracked tack welds	O	O
Environmental conditions • Wind speed within limits • Precipitation and temperature	O	O
WPS followed • Settings on welding equipment • Travel speed • Selected welding materials • Shielding gas type/flow rate • Preheat applied • Interpass temperature maintained (min./max.) • Proper position (F, V, H, OH)	O	O
Welding techniques • Interpass and final cleaning • Each pass within profile limitations • Each pass meets quality requirements	O	O

TABLE N5.4-3
Inspection Tasks After Welding

spection Tasks After Welding	QC	QA
	O	O
ion of welds	P	P
ceptance criteria usion on	P	P
	P	P
	P	P
weld tabs removed (if required)	P	P
	P	P
or rejection of welded joint or member	P	P

r plates, continuity plates or stiffeners has been performed in the k-area, visually for cracks within 3 in. (75 mm) of the weld.

Appendix 1. Design by Inelastic Analysis

1.1. General Provisions

1.2. Ductility Requirements
   1. Material
   2. Cross Section
   3. Unbraced Length
   4. Axial Force

1.3. Analysis Requirements
   1. Material Properties and Yield Criteria
   2. Geometric Imperfections
   3. Residual Stress and Partial Yielding Effects

Appendix 2. Design for Ponding

2.1. Simplified Design for Ponding

2.2. Improved Design for Ponding

Appendix 3. Design for Fatigue

3.1. General
3.2. Calculation of Maximum Stresses and Stress Ranges
3.3. Plain Material and Welded Joints
3.4. Bolts and Threaded Parts
3.5. Special Fabrication and Erection Requirements

Appendix 4. Design for Fire Condition

- 4.1. General Provisions
  - 4.1.1. Performance Objective
  - 4.1.2. Design by Engineering Analysis
  - 4.1.3. Design by Qualification Testing
  - 4.1.4. Load Combinations and Required Strength
- 4.2. Design for Fire Conditions by Analysis
  - 4.2.1. Design-Basis Fire
    - 4.2.1.1. Localized Fire
    - 4.2.1.2. Post-Flashover Compartment Fires
    - 4.2.1.3. Exterior Fires
    - 4.2.1.4. Fire Duration
    - 4.2.1.5. Active Fire Protection Systems
  - 4.2.2. Temperatures in Structural Systems Under Fire Conditions
  - 4.2.3. Material Strengths at Elevated Temperatures
    - 4.2.3.1. Thermal Elongation
    - 4.2.3.2. Mechanical Properties at Elevated Temperatures
  - 4.2.4. Structural Design Requirements
    - 4.2.4.1. General Structural Integrity
    - 4.2.4.2. Strength Requirements and Deformation Limits
    - 4.2.4.3. Methods of Analysis
    - 4.2.4.4. Design Strength
- 4.3. Design by Qualification Testing
  - 4.3.1. Qualification Standards
  - 4.3.2. Restrained Construction
  - 4.3.3. Unrestrained Construction

Appendix 5. Evaluation of Existing Structures

5.1. General Provisions

5.2. Material Properties
- 5.2.1 Determination of Required Tests
- 5.2.2 Tensile Properties
- 5.2.3 Chemical Composition
- 5.2.4 Base Metal Notch Toughness
- 5.2.5 Weld Metal
- 5.2.6 Bolts and Rivets

5.3. Evaluation by Structural Analysis
- 5.3.1 Dimensional Data
- 5.3.2 Strength Evaluation
- 5.3.3 Serviceability Evaluation

5.4. Evaluation by Load Tests
- 5.4.1 Determination of Load Rating by Testing
- 5.4.2 Serviceability Evaluation

5.5. Evaluation Report

Appendix 6. Stability Bracing for Columns and Beams

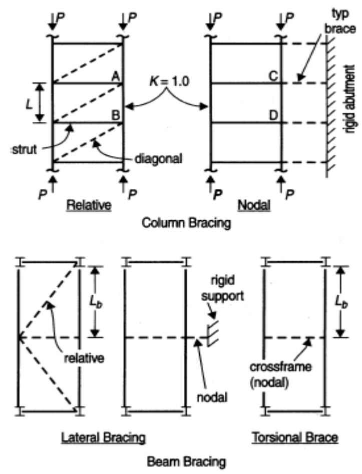
6.1. General Provisions

6.2. Columns
1. Relative Bracing
2. Nodal Bracing

6.3. Beams
1. Lateral Bracing
   - 1a. Relative Bracing
   - 1b. Nodal Bracing
2. Torsional Bracing
   - 2a. Nodal Bracing
   - 2b. Continuous Torsional Bracing

6.4. Beam-Column Bracing

Appendix 6. Stability Bracing for Columns and Beams



Appendix 7. Alternative Methods of Design For Stability

7.1. General Requirements

7.2. Effective Length Method

7.3. First-Order Analysis Method

Appendix 8. Approximate Second-Order Analysis

8.1. Limitations

8.2 Calculation Procedure

1. Multiplier B1 for P-δ Effects
2. Multiplier B2 for P-Δ Effects

Design Problems
Members

Example 2.3

- From Alan Williams, Ph. D., SE

  Structural Engineering

  PE License Review Problems & Solutions 8th Ed.

  – Page 143
- Illustration of member design
- Added issues: Analysis and second-order effects

Example 2.3

- Given
  – Job crane system as shown in Exhibit 2.3, consisting of a vertical pipe column, horizontal I-beam monorail, and diagonal rod

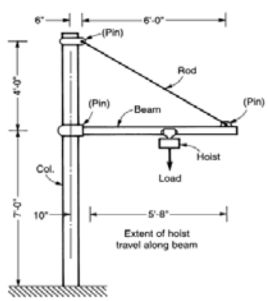
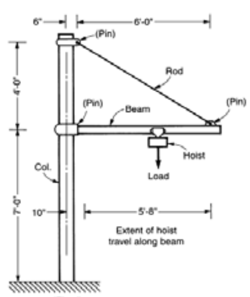


Exhibit 2.3

Example 2.3

- Criteria
  - Hoist weight = 250 lbs
  - Maximum lifted load = 3000 lbs
  - Impact factor = 1.25 (on lifted load)
- Materials
  - Rod = ¾-inch diameter: $F_{y\,=}$ 36 kips per square inch
  - Beam = S5 × 10 (I beam): $F_{y\,=}$ 36 kips per square inch
  - Column = 8-inch nominal diameter, standard weight pipe: $F_{y\,=}$ 35 kips per square inch
- Assumptions
  - Neglect the rod and beam weight
  - Neglect deflections
  - Assume all connections are adequate
  - Neglect eccentricity of all connections
  - Assume support conditions shown as “Pin” or “Fixed”

Example 2.3



Required

1. Maximum design axial force in the road and its adequacy to support the design vertical loads
2. Maximum design axial and bending forces in the beam and its adequacy to support the design vertical loads
3. Maximum design axial and bending forces in the column and its adequacy to support the design vertical loads

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Approach to Part 1

Goal: Determine force in rod and check adequacy

- Determine load
- Determine design condition
- Calculate force in rod (required strength)
- Calculate nominal strength
- Apply
  - resistance factor (LRFD), or
  - safety factor (ASD)
- Compare available and required strength

Determine Load

Applied load

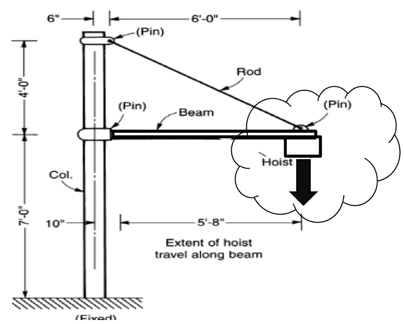
- The maximum applied load due to the hoist weight and the lifted load is given.

W = (250 + 1.25 × 3000) / 1000
= 4 kips
D + L

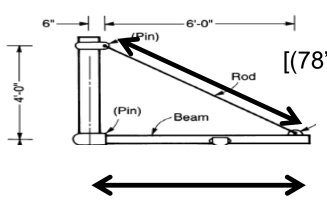
Given.
Minimum 20% increase for machinery per ASCE 7 4.7.2

LRFD:
1.2 D + 1.6 L = 1.2(0.25) + 1.6(1.25*3)
W = 6.7 kips

Determine Design Condition



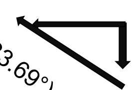
Calculate Force in Rod



$[(78")^2 + (48")^2]^{1/2} = 91.6"$

$6' - 0" + 6" = 78"$

$\tan^{-1}(48/78) = 33.69°$



6.7 kips/sin(33.69°) = 12.1 kips

$\boxed{R_u = 12.1 \text{ kips}}$

Calculate Nominal Strength—Apply Resistance Factor

- $R_n = A_g F_y$
  $= {}^{\pi}/_4({}^3/_4)^2$ 36 ksi = 15.9 kips

$\phi = 0.9$

$\phi R_n = 14.3$ kips

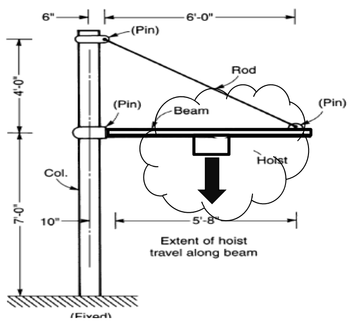
$\phi R_n > R_u = 12.$kips OK

Approach to Part 2

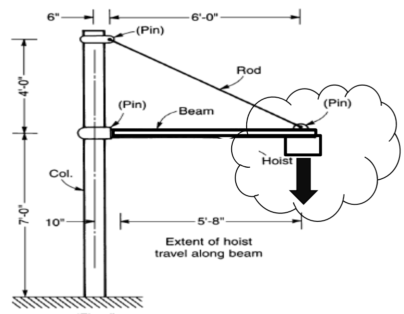
Goal: Determine axial and bending forces in beam and check adequacy

- Determine design condition
- Calculate forces in beam (required strength)
- Calculate nominal strengths (axial and flexural)
- Apply
  - resistance factor (LRFD), or
  - safety factor (ASD)
- Compare available and required strength

Determine Design Condition



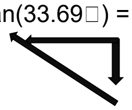
Maximum flexure



Maximum axial

Calculate Force in Beam Axial

½ (6.7 kips)/tan(33.69□) = 5.03 kips

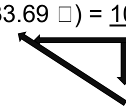


½ 6.7 kips

R_u = 5.03 kips

Maximum flexure

6.7 kips/tan(33.69 □) = 10.1 kips



6.7 kips

R_u = 10.1 kips

Maximum axial

Calculate Force in Beam—Flexure

$M_u = {}^{PL}/_{4} = 6.7 \text{ kips} * (6')/4$

$\boxed{R_u = 10.1 \text{ ft-kips}}$

Maximum flexure

$\boxed{R_u = 0 \text{ ft-kips}}$

Maximum axial

Calculate Force in Beam—Flexure

$$B_1 = \frac{C_m}{1 - \alpha P_r \ / \ P_{e1}} \geq 1 \qquad \text{(C2-2)}$$

$$\frac{\pi^2 E I}{(K_1 L)^2} = 684\text{kips}$$

$C_m = 1 - 0.2\ P_u / P_{e1}$ Table C-C2.1

Take as 1.0

$B_1 = 1.007$

Take as 1.0

Calculate Nominal Strength—Flexure

S5×10 is compact

Check yielding, LTB

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB

Yield

$$M_n = Z F_y$$

$$= 5.67 \text{ in.}^3 \times 36 \text{ ksi} = 204 \text{ kips-in.}$$

$$= 17.4 \text{ ft-kips}$$

Calculate Nominal Strength—Flexure

- Lateral Torsional Buckling

S5×10

F_y = 36 kips per square inch
S_x = 4.90 inches^3
A = 2.93 inches^2
r_y = 0.638 inches
L_p = 2.66 feet
L_r = 14.4 feet (for C_b = 1)
(BF) = 0.341

This section is compact, and the allowable bending stress is dependent on the value of the maximum unbraced length of the compression flange, which is assumed to be

$$L_b = 6 \text{ feet}$$

The bending coefficient is obtained from AISC Table 3-1 as

$$C_b = 1.32$$

Hence,

$$L_p < L_b < L_r$$

Inelastic buckling governs, and the allowable flexural strength in the absence of axial load is obtained from AISC Equation (F2-2)

Calculate Nominal Strength—Flexure

- Lateral Torsional Buckling

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

$$M_n = C_b \left[M_p - BF \left(L_b - L_p \right) \right] = 21.46 \text{ kips}$$

Yielding controls

$\phi M_n = \phi Z F_y$ = (0.9)17.4 ft-kips = 15.7 ft-kips

$M_u / \phi M_n$ = 0.643

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Calculate Nominal Strength—Axial

KL/r_y = 1.0 × 6 × 12 / 0.638
= 113

ϕF_{cr} = 16.5 ksi AISC Table 4-22

$\phi P_n = \phi F_{cr} A_g$ = 48.5 kip

$P_u / \phi P_n$ = 0.10 (with max. moment)

$P_u / \phi P_n$ = 0.20 (axial only) OK

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Compare Available and Required Strength (combined)

Chapter H

$P_u / \phi P_n < 0.2$

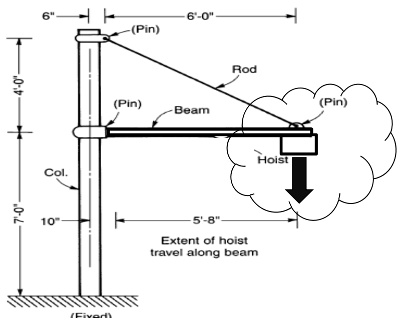
$\frac{1}{2} P_u / \phi P_n + M_u / \phi M_n = 0.69$ OK

Approach to Part 3

Goal: Determine axial and bending forces in column and check adequacy

- Determine design condition
- Calculate forces in column (required strength)
- Calculate nominal strengths (axial and flexural)
- Apply
  - resistance factor (LRFD), or
  - safety factor (ASD)
- Compare available and required strength

Determine Design Condition



Maximum axial, Maximum flexure

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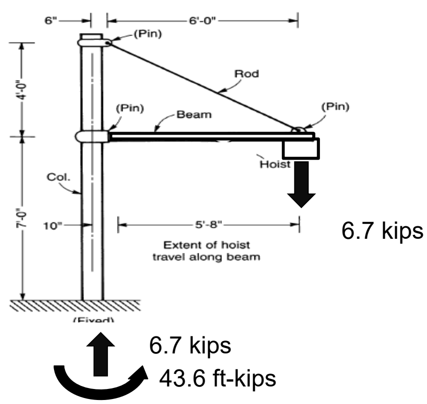
Calculate Forces in Column

First-order analysis



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Calculate Forces in Column

$\Delta_1 < \frac{1}{2} M_1 L^2 / EI = 2.17$ in.

(L = 11 ft conservatively)

P Δ = 14.5 kip in. = 1.21 kip ft.

$\Delta_2 \sim \Delta_1 (M_1 + P\,\Delta) / M_1 = 1.03$

$\Delta_2 / \Delta_1 = 1.03$

Use second-order analysis, effective length method

$\Delta_2 / \Delta_1 < 1.1$, K=1

Minimum lateral force > 0.02 Y = 0.134 kips

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Calculate Force in Beam—Flexure

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$$

$$\frac{\pi^2 EI}{(K_1 L)^2} = 1119 \text{ kips}$$

$K_1 = 1$; assume no lateral translation

$C_m = 1$

$B_1 = 1.01$ M_u = 1.01 (43.6 ft-kips + 0.13 kips × 11ft) = 45.5 ft-kips

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Calculate Nominal Strength—Flexure

8-inch pipe is compact

Check yielding

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F8		N/A	N/A	Y, LB

Yield

M_n = $Z F_y$ = 20.8 in.^{3} × 35 ksi

= 728 kips-in.= 60.7 ft-kips

$M_u / \phi M_n$ = 0.76

Calculate Nominal Strength—Axial

KL/r_y = 1 × (11ft)(12 in./ft)/(2.95 in.) = 44.75

(K = 1 per amplified first-order analysis)

ϕF_{cr} = 28.4 ksi AISC Table 4-22

$\phi P_n = \phi F_{cr} A_g$ = 223 kip

$P_u / \phi P_n$ = 0.03

Compare Available and Required Strength (Combined)

Chapter H

$P_u / \phi P_n < 0.2$

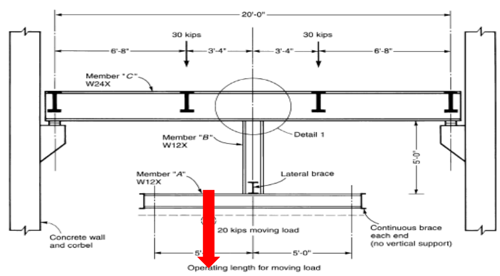
$\frac{1}{2} P_u / \phi P_n + M_u / \phi M_n = 0.77$ OK

Example 2.6

- From Alan Williams, Ph. D., SE
  Structural Engineering
  PE License Review Problems & Solutions 8th Ed.
  - Page 166
- Illustration of connection limit states

Example 2.6

- Given
  - Structural steel framing supported on concrete corbels and walls as shown in the building cross-section in Exhibit 2.6. Loadings: Vertical loads on member “C” and a moving load on member “A” are shown in the cross-section.



Building Cross Section

Example 2.6

- Materials
  - Grade 50 Steel
  - E70XX welding rods
- Assumptions
  - Do not consider weight of steel members
  - Allowance for impact and lateral load is not required
  - Ignore deflections

Required

Provide calculations and complete the sketch in the workbook of the welded connection for Detail 1 for the joint at the center line of member “C.” Indicate on this sketch any welds and/or additional plates.

Example 2.6

W12 × 30 member "B" :

- Yield stress F_{yb} = 50 ksi
- Web thickness t_w = 0.260 inches
- Web depth d_w = 11.42 inches
- Flange width b = 6.52 inches
- Flange thickness t_b = 0.44 inches
- Depth d_b = 12.3 inches
- Area A_B = 8.79 square inches

W24 × 62 member "C":

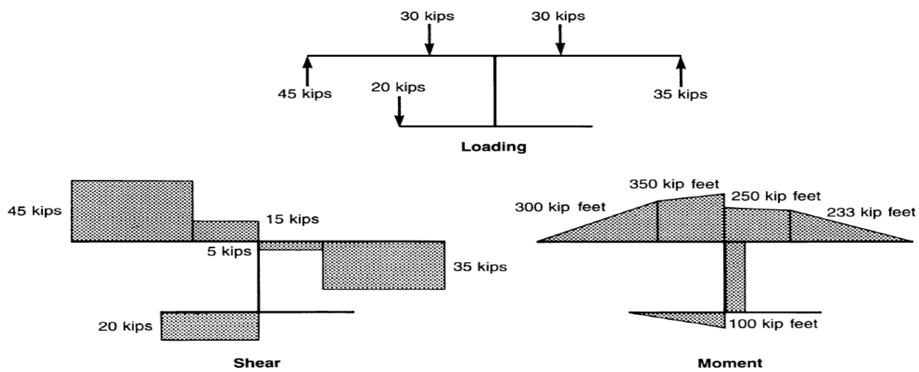
- Yield stress F_{yb} = 50 ksi
- Web thickness t = 0.43 "
- Fillet depth k = 1.09 inches
- Web depth between fillets d_c = 21.5"
- Flange thickness t_f = 0.59 "
- Depth d = 23.7 "
- Flange width b_f = 8.99 "
- Area A = 18.2 square inches

Part 2 Approach

Goal: Check appropriate connection limit states
design any reinforcement

- Analyze connection
- Identify and check limit states
  - Flange weld strength
  - Web weld strength
  - Panel zone shear
  - Local web yielding
  - Web crippling
  - Flange local bending
- Design reinforcement

Analyze Connection

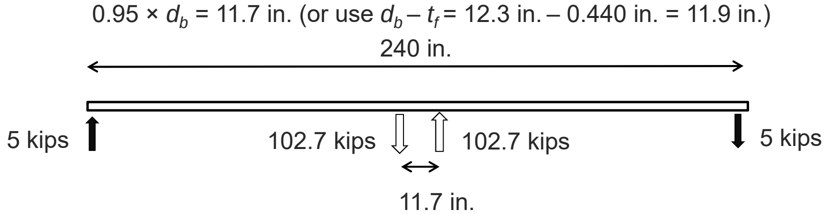


Analyze Connection

20kips × 5 ft = 100 kip-ft. = 1200 kip-in.

1200 kip-in./240 in. = 5 kip

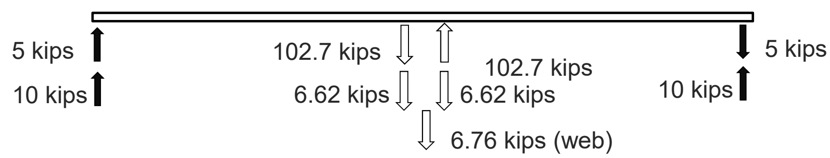
0.95 × d_b = 11.7 in. (or use $d_b - t_f$ = 12.3 in. – 0.440 in. = 11.9 in.)



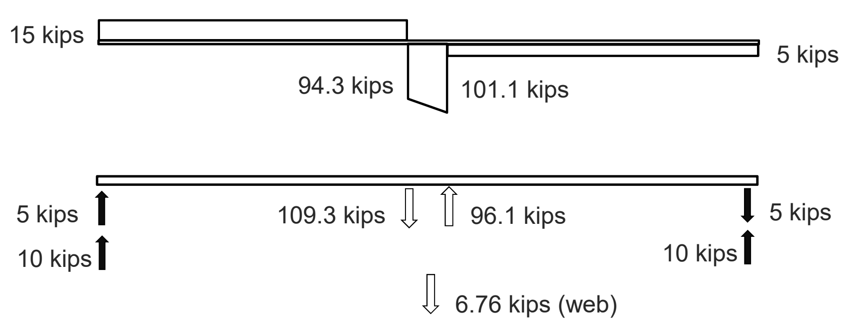
Analyze Connection

$$T_w = TA_w/A_B$$
$$= T \times t_w \times d_w/A_B$$
$$= 20 \times 0.26 \times 11.42/8.79$$
$$= 6.76 \text{ kips}$$

$$T_f = (T - T_w)/2$$
$$= (20 - 6.76)/2$$
$$= 6.62 \text{ kips}$$



Analyze Connection



Identify and Check Limit States

Flange Connection

To develop the full flexural and axial capacity of the flanges of member "B," it is necessary for the flanges to be connected to member "C" with full-penetration groove welds

Identify and Check Limit States

Web connection

The web thickness of member "B" is 0.43 inches, and the minimum allowable fillet weld size is given by AISC Table J2.4 as 3/16 inch. The allowable force on a 3/16-inch fillet weld is obtained from AISC Table J2.5 as

$$\begin{aligned} q &= 3 \times 0.928 \\ &= 2.78 \text{ kips per inch} \end{aligned}$$

The strength of the double 3/16-inch fillet weld in tension is

$$\begin{aligned} P_w &= 2qd_w \\ &= 2 \times 2.78 \times 11.42 \\ &= 64 \text{ kips} \\ &> 6.76 \text{ kips} \end{aligned}$$

Hence, the 3/16-inch fillet weld is satisfactory.

Identify and Check Limit States

- Panel zone shear

From AISC Section J10.6, the web capacity is

$$F_w = 0.4F_{yc} \times t \times d$$
$$= 0.4 \times 36 \times 0.43 \times 23.70$$
$$= 146.75$$

Hence, the web of member "C" need not be reinforced.

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Identify and Check Limit States

- Local web yielding

The available capacity of the W24 × 62 beam web for local yielding is given by AISC Equation (J10-2) as

$$R_n/W = F_y t_w (N + 5k)/1.5$$

where
N = length of bearing
= thickness of beam flange of member "B"
= 0.44 inch
k = distance from outer face of flange to web toe of fillet
= 1.09 inches
t_w = web thickness
= 0.43 inch

$$R_n / W = 50 \times 0.43(0.44 + 5 \times 1.09)/1.5$$
$$= 84.42 \text{ kips}$$

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Identify and Check Limit States

- Web crippling
  - When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to *d* / 2:

$$R_n = 0.80t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad \text{(J10-4)}$$

= 108 kips OK

Identify and Check Limit States

- Flange local bending < 109 kips

  From AISC Equation (J10-1), the available capacity of the flange for flange local bending is given as

$$\begin{aligned} P_{fb} &= 6.25t_f^2F_y/W \\ &= 6.25 \times 0.59^2 \times 50 / 1.67 \\ &= 65.14\text{...governs} \end{aligned}$$

- Reinforcement is required

Design Reinforcement

The force delivered to the stiffener is then

$$R_{st} = P_t - P_{fb}$$
$$= 109.32 - 65.14$$
$$= 44.18 \text{ kips}$$

The required area of a pair of stiffeners is then

$$A_{st} = R_{st}/(F_{yst}/W)$$
$$= 44.18/(36/1.67)$$
$$= 2.05 \text{ inches}^2$$

Design Reinforcement

Using a pair or stiffener plates 4 inches × ½ inch, with a 1 ½-inch corner clip, as recommended in AISC Commentary Section J10.8, provides a stiffener area at the column flange of

$$A_{st} = 2 \times (4 - 1.5) \times 0.5$$
$$= 2.5 \text{ inches}^2$$
$$> 2.05 \text{ inches}^2 \ldots \text{ satisfactory}$$

The minimum stiffener thickness required is defined by AISC Section J10.8

$$t_{st} = t_b/2$$
$$= 0.44/2$$
$$= 0.22 \text{ inch}$$
$$< 0.5 \text{ inch} \ldots \text{ satisfactory}$$

The minimum stiffener width required is defined by AISC Section J10.8 as

$$b_{st} = b_f/3 - t/2$$
$$= 8.99/3 - 0.43/2$$
$$= 2.78 \text{ inches}$$
$$< 4 \text{ inches} \ldots \text{ satisfactory}$$

Design Reinforcement

The maximum width/thickness ratio is defined by AISC Section J10.8 as

$$b_{st} / t_{st} = 15$$

The actual width/thickness ratio provided is

$$\begin{aligned} b_{st} / t_{st} &= 4 / 0.5 \\ &= 8 \\ &< 15 \text{ ... satisfactory} \end{aligned}$$

Design Reinforcement

1B. Stiffener Weld Requirements

The available capacity of the welded portion of the pair of stiffeners in tension is given by

$$\begin{aligned} P_{st} &= A_{st} F_{yst} / \Omega \\ &= 2.5 \times 36/1.67 \\ &= 54 \text{ kips} \\ &> 44.18 \text{... satisfactory} \end{aligned}$$

The ends of the stiffeners are connected to the column flange with 5/16 fillet welds on both sides. The allowable force on a 1/16-inch, E70XX-grade fillet weld is obtained from AISC Table J2.5 as

q = 0.928 kips per inch per 1/16 inch

Design Reinforcement

To develop the unbalanced flange force, the required fillet weld size in sixteenths of an inch is

$$\begin{aligned} D &= R_{st}/4q(b-1.5) \\ &= 44.18/(4 \times 0.928 \times 2.5) \\ &= 4.76 \text{ sixteenths} \end{aligned}$$

From AISC Table J2.4, the minimum size of fillet weld required for the 0.5-inch thick stiffener is

w = 3/16 inch

Use a weld size of

w = 5/16 inch

The unbalanced flange force to be transmitted by fillet welds to the web is

R_{st} = *44.18 kips*

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Design Reinforcement

E70XX fillet welds are provided on one side of the pair of stiffeners, and the total length provided after allowing for the 1½-inch corner clip is

$$\begin{aligned} R_{st} &= 12 - 1.5 \\ &= 10.5 \text{ inches} \end{aligned}$$

To develop unbalanced flange force, the required fillet weld size in 16ths of an inch is

$$\begin{aligned} D &= R_{st}/2qR_{st} \\ &= 44.18/(2 \times 0.928 \times 10.5) \\ &= 2.26 \text{ sixteenths} \end{aligned}$$

From AISC Table J2.4, the minimum size of fillet weld required for the 0.43-inch thick beam web is

w = 3/16 inch

Use of a weld size of

w = 3/16 inch

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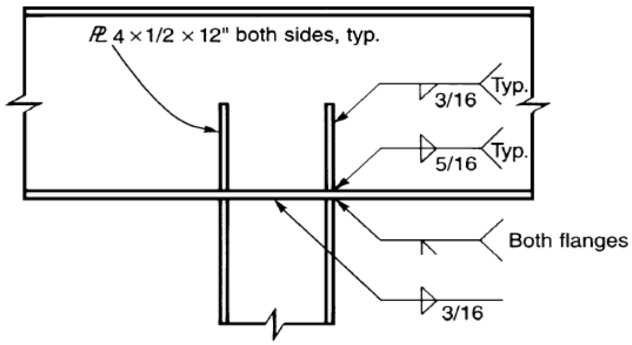
Connection Design



Structural Design Standards Relevant for Steel Design

In order of precedence of controlling requirements for forces and steel design:

- International Building Code (IBC 2012 Edition)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) (forces only)
- (AISC 360-10): Specification for Structural Steel Buildings, with Associated Commentaries

Recommended References and Additional Study Materials

- Williams, Alan and David Fanella. *Structural Engineering PE License Review Problems and Solutions, 8th Edition.* Kaplan Publishing, 2011.
- Salmon, Charles, John E. Johnson, and Faris A. Malhas. *Steel Structures: Design and Behavior*. Prentice Hall, 2008.
- AISC. "Manual of Steel Construction (Design Examples)." http://www.aisc.org/content.aspx?id=24314.
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