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Course Description

Secrets of the Manual
April 27, 2017

This presentation will highlight efficient ways to take advantage of the various design aids in the AISC Steel Construction Manual (14th Edition). The speaker will walk through the different sections of the Manual and highlight useful resources. The speaker will then focus on shortcuts for connection design, including: stiffeners, web yielding/crippling, welds, and more. Shortcuts for member design including composite members will also be reviewed. Learning these shortcuts are a must for those designing steel structures and checking designs.
Learning Objectives

• Locate important design shortcuts in the Manual
• Identify appropriate uses for eccentrically loaded bolt group tables
• Apply table shortcuts for beam bearing and column stiffener checks
• Design beams using composite beam tables in the Manual

Secrets of the Manual

Based on the 14th Edition Manual and AISC 360-10

Presented by
Carol Drucker, PE, SE
Drucker Zajdel Structural Engineers
Chicago, IL
Manual Organization - General Parts

- Part 1: Dimension and Properties
- Part 2: General Design Considerations

Manual Organization Main Member Design

- Part 3: Flexural Members
- Part 4: Compression Members
- Part 5: Tension Members
- Part 6: Members Subject to Combined Forces
Manual Organization - Connections

• Part 7: Design of Bolts
• Part 8: Design of Welds
• Part 9: Design of Elements
• Part 10: Simple Shear Connections
• Parts 11, 12: Moment Connections
• Part 13: Bracing and Truss Connections
• Part 14: Base Plates, Anchor Rods, Column Splices
• Part 15: Hanger Connections, Brackets

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Manual Organization - Main Member/Connections

• Part 16: Specification, Commentary, and Codes
  – AISC 360-10
    » Chapter A- N: Main Member, Stability, Connections, Fabrication, etc
    » Appendix 1- Appendix 8: Ponding, Fatigue, Fire, Existing, Stability, Analysis, etc
    » Commentary for Chapter A-N
    » Commentary for Appendix 1-8
  – RCSC: Specification for Structural Joints Using High-Strength Bolts
  – AISC: 303-10: Code of Standard Practice
Manual Organization—Main Member/Connections

- Part 17: Miscellaneous Data and Mathematical Information
  - Metric Shapes
  - Gage metal thickness
  - Coefficient of thermal expansion
  - Material Weights
  - Properties of Shapes

- General Nomenclature

- Index

---

Secrets of the Manual

- Part 1: Structural Properties
  - $A$, $d$, $t_w$, $b_f$, $t_p$ etc
  
  \[
  A = \frac{\pi d^2}{4} = \pi R^2 = 0.785398 \ d^2 = 3.141593 \ R^2
  \]
  
  \[
  c = \frac{d}{2} = R
  \]
  
  \[
  I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4} = 0.049087 \ d^4 = 0.785398 \ R^4
  \]
  
  \[
  S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4} = 0.098175 \ d^3 = 0.785398 \ R^3
  \]
  
  \[
  r = \frac{d}{4} = \frac{R}{2}
  \]
  
  \[
  Z = \frac{d^3}{6}
  \]

**Secrets of the Manual**

**Part 1: Structural Properties**
- Angle Gage in Table 1-7A
- Welded Flat Widths HSS

**Part 2: General Design Considerations**
- Bearing Connections
- Table 2-4: Preferred Material Specification - Shapes
- Table 2-6: ASTM Fasteners (Bolts, Rods)
- ASD to LRFD \[ \Omega = \frac{1.5}{\phi} \]
Part 3: Main Member

- Table 3-2: Selection by $Z_x$
- Table 3-6: Maximum Total Uniform Load
- Table 3-10: Available Moment vs Unbraced Length
- Table 3-23: Beam Shear, Moment, and Deflection

Part 4, Part 5, Part 6: Main Member

- Table 4-1: Compression of W-Sections
- Table 4-8 to 4-10: Axial Compression Double Angles
- Table 4-12: Eccentrically Loaded Single Angles
- Table 4-22: $kl/r$ Table
Secrets of the Manual

• Part 4, Part 5, Part 6: Main Member

  – Table 4-8 to 4-10: Axial Compression Double Angles
  \[
  \left(\frac{KL}{r}\right)^2 = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{a}{\eta}\right)^2}
  \]  
  (E6-1)
  
  (i) When \(\frac{a}{\eta} \leq 40\)
  \[
  \left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_a
  \]  
  (E6-2a)

  (ii) When \(\frac{a}{\eta} > 40\)
  \[
  \left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_a^2 + \left(\frac{a}{\eta}\right)^2}
  \]  
  (E6-2b)

Manual Organization

• Part 4-6: Main Member

  – Table 4-12: Eccentrically Loaded Single Angles

  • The effects of eccentricity can be neglected using an effective slenderness ratio if (Spec E5):
    – Member are loaded at the ends in compression through same on leg
    – Member attached by welding or by a minimum of two bolts
    – No transfer loading
Secrets of the Manual

• Part 4, Part 5, Part 6: Main Member

– Table 4-22: KL/r Table

The critical stress, \( F_{cr} \), is determined as follows:

(a) When \( KL \leq 4.71 \left( \frac{E}{F_y} \right) \) (or \( \frac{F_y}{F_e} \leq 2.25 \))

\[ F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y \]  

(E3-2)

(b) When \( KL > 4.71 \left( \frac{E}{F_y} \right) \) (or \( \frac{F_y}{F_e} > 2.25 \))

\[ F_{cr} = 0.877 F_y \]  

(E3-3)

Secrets of the Manual

Table 4-22 (continued)

Available Critical Stress for Compression Members

<table>
<thead>
<tr>
<th>KL</th>
<th>50 ksi</th>
<th>70 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>r</td>
<td>ksi</td>
<td>ksi</td>
</tr>
<tr>
<td>121</td>
<td>10.3</td>
<td>15.4</td>
</tr>
<tr>
<td>122</td>
<td>10.1</td>
<td>15.2</td>
</tr>
<tr>
<td>124</td>
<td>9.94</td>
<td>14.9</td>
</tr>
</tbody>
</table>

Secrets of the Manual

• Table 4-22 – Use for Whitmore Check for Gussets

\[ \frac{KL}{r} = \frac{KL \sqrt{12}}{l_{gusset}} \]


Secrets of the Manual

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Part 7: Bolts
- Table 7-1, 7-2, and 7-3: Bolt Shear and Tension Values
- Table 7-4: Bolt Dimensions
- Table 7-6: Coefficients C for Eccentrically loaded bolt groups
- Table 7-15 and 7-16: Bolt Installing Tolerance
Bolt Installation-TC Bolts

Example: gwyinc.com

Secrets of the Manual

BOLT TENSION

\[ r_i = \frac{M_c}{n_b d_m} \]

- See Manual Fig. 7-7
• See Manual Fig. 7-6 Case I

\[ M_n = \sum n_i (r_i) d_i \]

• See Manual Fig. 7-7 Expanded
(similar to DG4* and DG16*)

*free download for AISC members!
Secrets of the Manual

• Part 8: Welds

- Figure 8-16: Susceptible and Improved Details
- Table 8-2: Prequalified Welds (AWS)
- Table 8-3: Electrode Strength Coefficient, $C_1$
- Table 8-4 to Table 8-11: Coefficients $C$ for Eccentrically Loaded Weld Groups

\[ C_1 = \frac{(0.9)(80\, \text{ksi})}{70\, \text{ksi}} \]

Secrets of the Manual

• Part 8: Welds: Table 8-12

- Cost of Welds
- Fillet and Groove Welds

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>Fillet Welds</th>
<th>30 Bead</th>
<th>45 Bead</th>
<th>30 Groove Angle</th>
<th>45 Groove Angle</th>
<th>90 Groove Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{5}{16} )</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \frac{5}{8} )</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8-12: Approximate Number of Passes for Welds

*Note: Thickness for groove welds.
Secrets of the Manual

- Part 9: Connections
  - Cope Beams: Single Coped and Double Coped
  - Prying Action
  - Rotational Ductility
  - Filler and Shims
  - Table 9-4: Beam Bearing
**Part 9: Shear Rupture at Welds**

- $t_{\text{min}}$ to develop weld:
  - Shear Rupture = Fillet Weld Strength
  $$t_{\text{min}} = \frac{\phi_s(0.6)(F_w)(0.707)(w)}{\phi(0.6)(F_u)} \quad \text{(Manual Eq. (9–2))}$$
  $$= \frac{0.75(0.60)(70 \text{ ksi})(0.707)(\frac{D}{16})}{0.75(0.6)(F_u)}$$
  $$= \frac{2(1.392)(D)}{0.75(0.6)(F_u)}$$
  $$= \frac{6.19D}{F_u} \quad \text{(Manual Eq. (9–3))} \quad \text{(2-Sided Connections)}$$

**Part 10: Connections**

- Figure 10-3: Encroachment
- Table 10-1 to Table 10-9: Connection Tables
- Figures: Skewed Plate Welds

---

Fig. 10-3. Fillet encroachment (riding the fillet).
Skewed Plate Welds

Skewed Plate Welds

Table 8-2 (continued)

Single V-groove weld (A)
Blank plate (B)
Fillet (T)
Groove (K)

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Joint Designation</th>
<th>Base Metal Thickness D (in. unfilled)</th>
<th>Groove Preparation</th>
<th>Allow. Welding Preheat</th>
<th>Total Weld Unit (U)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMAW</td>
<td>BTG-P4</td>
<td>U</td>
<td>0</td>
<td>0.5</td>
<td>+10</td>
<td>+10</td>
</tr>
<tr>
<td>SMAW</td>
<td>BTG-P4-SP</td>
<td>U</td>
<td>0</td>
<td>0.5</td>
<td>+10</td>
<td>+10</td>
</tr>
<tr>
<td>SAW</td>
<td>TC-P4</td>
<td>U</td>
<td>0</td>
<td>0.5</td>
<td>+10</td>
<td>+10</td>
</tr>
</tbody>
</table>
• PLs Over ½”

![Diagram of Skewed Plate Welds]

Support

3/16” Max Gap
for Fillet Weld

WO
WA

Support

S(E)
WA
BTC-P4

W

Skewed Plate Welds

• 1-Sided Welds

![Diagram of 1-Sided Welds]

CJP

PJP+REINFORCING FILLET
(AWS Annex A)
**Plate Welds**

- **Alternate Bent Plate Detail Large Skews**
  - Typically 1/2" Max Plate Thickness
  - Design from Bend Line
  - “Design of Skewed Connections” by Larry Kloiber and William A. Thornton, EJ 3rd Quarter 2001

**Plate Welds**

- **Examples when rotational ductility and a fillet weld size of 5/8t is not always needed**
  - Bracing Connections
  - Moment Connections
**Secrets of the Manual**

- **Part 11-Part 12:** Partially Retrained and Fully Restrained Moment Connections
  
  - Extended End-Plate Moment Connections
    - Design assumptions 1-11
      - End-plate effective width = \( b_f + 1 \) in.
      - CJP welds are treated as PJP welds between beam flange-to-web fillets. No weld access holes
    - Also see DG4* and DG16*

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Secrets of the Manual

• Part 13: Bracing
  – See DG29 and Design Examples

  DESIGN EXAMPLES
  Version 14.0

Link to design examples: www.aisc.org/publications/steel-construction-manual-resources/

Secrets of the Manual

• Part 14: Beam Bearing Plates, Column Base Plates, Anchor Rods, and Column Spices
  – Table 14-2: Recommended Maximum Sizes for Anchor-Rod Holes in Base Plates (also see DG1*)
  – Tables 14-3: Cases I-XII

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Secrets of the Manual
Part 15: Hanger Connections, Bracket Plates, and Crane Rail Connections

- Figure 15-2: Bracket Plates
- Table 15-3: \( Z_{net} \) plates
- Table 15-4 to 15-6: Clevis, Pins, and Turnbuckles

Part 15: Brackets Cont.
• **Part 15: Clevises and Pins**
  - For design: *Specification* Section D5 and J7
  - Example: clevelandcityforge.com

---

**Secrets of the Manual**

• **Part 16: Specification for Structural Steel Buildings (360-10),**

  1. **Shear Strength**
     
     This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

     The nominal shear strength, \( V_n \), of unstiffened or stiffened webs according to the limit states of shear yielding and shear buckling, is

     \[
     V_n = 0.6F_y A_w C_s \tag{G2-1}
     \]

     (a) For webs of rolled I-shaped members with \( h/w \leq 2.34 \sqrt{E/F_y} \):

     \[
     \phi_s = 1.00 \text{ (LRFD)} \quad \Omega_s = 1.50 \text{ (ASD)}
     \]

     and

     \[
     C_s = 1.0 \tag{G2-2}
     \]

     **User Note:** All current ASTM A6 W, S and HP shapes except W4×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for \( F_y = 50 \text{ ksi (345 MPa)} \).
Secrets of the Manual

• Part 16: Specification for Structural Steel Buildings (360-10),

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, the web shear coefficient, $C_v$, is determined as follows:

(i) When $h/t_w \leq 1.10 \sqrt[3]{k_c E / F_y}$

$$C_v = 1.0$$  

(G2-3)

(ii) When $1.10 \sqrt[3]{k_c E / F_y} < h/t_w \leq 1.37 \sqrt[3]{k_c E / F_y}$

$$C_v = \frac{1.10 \sqrt[3]{k_c E / F_y}}{h/t_w}$$  

(G2-4)

(iii) When $h/t_w > 1.37 \sqrt[3]{k_c E / F_y}$

$$C_v = \frac{1.51 k_c E}{(h/t_w)^2 F_y}$$  

(G2-5)

---

Secrets of the Manual

• RCSC

- Masking Requirements: Fig C-3.1

- Section 4: PT and SC Joint Applications

- Section 6: Washer Requirements

- Section 8: Bolt Installation Methods

---

Table 6.1. Washer Requirements for Pretensioned and Slip-Critical Bolted Joints with Oversized and Slotted Holes in the Outer Ply

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Nominal Bolt Diameter, d_b, in.</th>
<th>Hole Type in Outer Ply</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Oversized</td>
<td>Short-Slotted</td>
</tr>
<tr>
<td>A235 or F1852</td>
<td>( \leq 1 )</td>
<td>ASTM F436 *</td>
</tr>
<tr>
<td>A490 or F2280</td>
<td>( &gt; 1 )</td>
<td>ASTM F436 washer with 1/8 in. thickness ***</td>
</tr>
</tbody>
</table>

* This requirement shall not apply to heads of round head tension-control bolt assemblies that meet the requirements in Section 2.7 and provide a bearing circle diameter that meets the requirements of ASTM F1852 or F2280.

** The plate washer or bar shall be of structural-grade steel material, but need not be hardened.

*** Alternatively, a 1/8 in. thick plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.
Secrets of the Manual

- Part 16: Code of Standard Practice
  - Section 3.1.2: Delegate Design
    - Fabricator shall submit representative samples of connections
    - Shop and Erection drawing review
  - Section 6.4 Fabrication Tolerance
  - Section 10: AESS Steel

Connection Details for Delegated Design

- Show all information to allow for checking
- Member forces and sizes
- Slopes
- ASD or LRFD with Specification Edition
- Plate thickness, edge distances, etc
Bearing Example

• Given:
  – AISC Specification 360-10
  – AISC 14th Edition, LRFD
  – W Shapes ASTM A992, $F_y = 50$ ksi
  – 1" cap plate, Grade 50, $F_y = 50$ ksi
  – 7/8" dia. A325-N, STD holes, $\phi_r v = 24.3$ kip
  – Load $P_u = 200$ kips
  – $d \geq e \geq d/2$

Efficient Use of Tables

Web Local Yielding ($\phi = 1.00$)

– AISC Specification Eq. J10-3

$$\phi R_u = \phi F_y f_v (2.5k + l_v)$$

Determine $l_v$
**Efficient Use of Tables**

- Length of bearing
  
  \[ l_b = t_{ws} + 5 t_{pl} \]
  
  \[ = 0.440\text{in.} + 5(1.00\text{in.}) \]
  
  \[ = 5.44\text{in.} \]

- Web Local Yielding \((\phi = 1.00)\)
  
  \[ l_b = 5.44\text{in.} \]
  
  - AISC Specification Eq. J10-3
    
    \[ \phi R_u = \phi F_{yw} t_w (2.5k + l_b) \]
    
    \[ = 1.00(50\text{ksi})(0.600\text{in.})(2.5(1.54\text{in.}) + (5.44\text{in.})) \]
    
    \[ = 279\text{kips} \geq P_u = 200\text{kips} \quad \text{o.k.} \]

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member, \(d\),

\[ R_u = F_{yw} t_w (2.5k + l_b) \quad \text{(J10-3)} \]
Efficient Use of Tables

- Web Local Crippling \( (\phi = 0.75) \)
  
  - *AISC Specification* Eq. J10-4

\[
R_n = 0.80r_w^2 \left[ 1 + \frac{3}{d} \left( \frac{t_w}{t_f} \right) \right] \sqrt{\frac{EF_{yw}f_f}{t_w}}
\]

(a) When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to \( d/2 \):

\[
\phi = \phi + \sqrt{\frac{EF_{yw}f_f}{t_w}}
\]

\[
= \left( 0.80 \right) \left( 0.600 \text{ in.} \right)^2 \left( 1 + 3 \left( \frac{5.44 \text{ in.}}{35.6 \text{ in.}} \right) \left( \frac{0.600 \text{ in.}}{0.790 \text{ in.}} \right)^{1.5} \right) \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}} \left( \frac{0.790 \text{ in.}}{0.600 \text{ in.}} \right)}
\]

\[
= 389 \text{ kips} \geq P_a = 200 \text{ kips} \quad \text{o.k.}
\]
Efficient Use of Tables

- Web Local Yielding \( (\phi = 1.00) \)
  \( l_b = 5.44 \text{ in.} \)
  - AISC 14th Edition Eq. 9-45a

\[
\phi R_u = \phi R_1 + l_b(\phi R_2)
\]

\[
\phi R_u = \phi R_1 + l_b(\phi R_2) \geq 0.0 \text{ kips/in.}
\]

\[
= 116 \text{ kips} + (5.44 \text{ in.}) \left(30.0 \text{ kips/in.}\right)
\]

\[
= 279 \text{ kips} \geq P_u = 200 \text{ kips} \quad \text{o.k.}
\]

- Web Local Crippling \( (\phi = 0.75) \)
  \( l_b = 5.44 \text{ in.} \)
  - AISC 14th Edition Eq. 9-49a

\[
\phi R_u = 2\left[\phi R_3 + l_b(\phi R_4)\right]
\]

\[
= 2\left[149 \text{ kips} + (5.44 \text{ in.}) \left(8.32 \text{ kips/in.}\right)\right]
\]

\[
= 389 \text{ kips} \geq P_u = 200 \text{ kips} \quad \text{o.k.}
\]
Efficient Use of Tables

Column Stiffener Checks

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W Shapes ASTM A992, $F_Y = 50$ ksi
  - 3/4”x9” flange plates, grade 50, $F_Y = 50$ ksi
  - 7/8” dia. A325-N, STD holes, $\phi_{rv}=24.3$ kip
  - Load $M_u = 300$ kips-ft

<table>
<thead>
<tr>
<th>Flange Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_\beta = \frac{M_u}{d}$</td>
</tr>
<tr>
<td>$= \frac{300 \text{kips-ft}}{23.6 \text{in.}}$</td>
</tr>
<tr>
<td>$= 153$ kips</td>
</tr>
</tbody>
</table>
**Efficient Use of Tables**

- **Web Local Yielding** \((\phi = 1.00)\)
  \[
  \phi R_u = \phi F_{yw} t_w \left( 5k_{des} + t_{pl} \right)
  \]
  \[
  = 1.00(50 \text{ksi})(0.440 \text{in.})(5(1.31 \text{in.}) + (0.75 \text{in.}))
  \]
  \[
  = 161 \text{ kips} \geq P_u = 153 \text{kips} \quad \text{o.k.}
  \]

- **Web Local Crippling** \((\phi = 0.75)\)
  - AISC Specification Eq. J10-4
  \[
  \phi R_u = \phi 0.80 t_w \left( 1 + 3 \left( \frac{t_{pl}}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right) \sqrt{\frac{EF_{yw} t_f}{t_w}}
  \]
  \[
  = 0.75(0.80)(0.440 \text{in.})^2 \left( 1 + 3 \left( \frac{0.75 \text{in.}}{14 \text{in.}} \right) \left( \frac{0.440 \text{in.}}{0.710 \text{in.}} \right)^{1.5} \right) \sqrt{\frac{(29000 \text{ksi})(50 \text{ksi})(0.710 \text{in.})}{(0.440 \text{in.})}}
  \]
  \[
  = 192 \text{ kips} \geq P_u = 153 \text{kips} \quad \text{o.k.}
  \]
**Efficient Use of Tables**

- **Web Compression Buckling** \( (\phi = 0.9) \)
  - *AISC Specification* Eq. J10-8
  \[
  \phi R_n = \phi \frac{24 t_w^2 \sqrt{E F_y}}{h} = (0.9) \frac{24 (0.440 \text{in.})^3 \sqrt{(29000 \text{ksi})(50 \text{ksi})}}{14 \text{in.} - 2(1.31 \text{in.})} = 194 \text{kips} \geq P_\beta = 153 \text{kips} \text{ o.k.}
  \]

- **Flange Local Bending** \( (\phi = 0.9) \)
  - *AISC Specification* Eq. J10-1
  \[
  \phi R_n = \phi 6.25 F_m \frac{t_f^2}{t_f} = 0.9 (6.25)(50 \text{ksi})(0.710 \text{in.})^2 = 142 \text{kips} \leq P_\beta = 153 \text{kips} \text{ n.g.}
  \]
  Stiffeners are needed

See Blodgett 5.7-8
**Efficient Use of Tables**

- **Web Local Yielding (φ = 1.00)**
  - AISC 14th Edition Eq. 4-2a
  
  \[ \phi R_n = P_{wo} + P_{wi} \phi I_b \]  
  
  \[ \phi R_n = 144 + 22.0 \times 0.75 \text{ in.} \]  
  
  \[ = 161 \text{ kips} \geq 153 \text{ kips} \text{ o.k.} \]

- **Web Local Crippling (φ = 0.75)**
  - AISC 14th Edition Eq. 9-49a
  
  \[ \phi R_n = 2 \left( \phi R_3 + I_b \phi R_4 \right) \]  
  
  \[ \phi R_n = 2 \left( 88.8 + 0.75 \times 9.29 \text{ in.} \right) \]  
  
  \[ = 192 \text{ kips} \geq 153 \text{ kips} \text{ o.k.} \]
**Efficient Use of Tables**

- **Web Compression Buckling** \((\phi = 0.9)\)
  - AISC 14th Edition Eq. 4-3a

\[
\phi R_n = P_{wb}
\]

\[
\phi R_n = \frac{P_{wb}}{P_{fl}}
\]

\[
= 194 \text{ kips} \geq P_{fl} = 153 \text{ kips} \quad \text{o.k.}
\]

- **Flange Local Bending** \((\phi = 0.9)\)
  - AISC 14th Edition Eq. 4-4a

\[
\phi R_n = P_{fb}
\]

\[
\phi R_n = \frac{P_{fb}}{P_{fl}}
\]

\[
= 142 \text{ kips} \leq P_{fl} = 153 \text{ kips} \quad \text{n.g.}
\]

**Stiffeners are needed**
Polling Question

Understanding Tables

Determine End Reaction from UDL

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W14x22 ASTM A992, $F_y = 50$ ksi
  - $L = 20$ ft
  - End Reaction = 50% UDL
Understanding Tables

• Shear Based on Flexural Yielding Strength \((\phi = 0.9)\)
  - AISC Specification Eq. F2-1
  \[
  \phi M_n = \phi F_y Z_x
  \]
  \[
  = 0.9(50 \text{ ksi})(33.2 \text{ in.}^3)
  \]
  \[
  = 1494 \text{ kip-in (1 ft/12 in.)}
  \]
  \[
  = 125 \text{ kip-ft}
  \]
  \[
  w_{f,\text{flexure}} = \frac{\phi M_n}{l}
  \]
  \[
  = \frac{125 \text{ kip-ft}(8)}{20 \text{ ft}}
  \]
  \[
  = 49.8 \text{ kips}
  \]

• Shear Based on Shear Strength \((\phi = 1)\)
  - AISC Specification Eq. G2-1
  \[
  \phi V_n = \phi(0.6)(F_y)(A_v)(C_v)
  \]
  \[
  = 1(0.6)(50 \text{ ksi})(13.7 \text{ in.})(0.23 \text{ in.})(1)
  \]
  \[
  = 94.5 \text{ kips}
  \]
  \[
  w_{f,\text{shear}} = \phi V_n(2)
  \]
  \[
  = 94.5 \text{ kips}(2)
  \]
  \[
  = 189 \text{ kips}
  \]
Understanding Tables

• Determine End Reaction Based on Percent UDL

\[ V_u = \min \left( \left( \frac{w l_{\text{flexure}}}{100}\% \right), \frac{w l_{\text{shear}}}{2} \right) \]

= \min(49.8 \text{ kip} \times 0.50, 189 \text{ kips})

= \min(24.9 \text{ kip}, 94.5 \text{ kip})

= 24.9 \text{ kip}

\[ w_f = 49.8 \]

• UDL Cautions:

– For 100% composite, \( \phi M_n = 297 \text{ kip-ft.} \)

This gives:

\[ V_u = \frac{\phi M_n (8)}{l(2)} \]

= \frac{297 \text{ kip-ft} \times 8}{20 \text{ ft} \times 2}

= 59.4 \text{ kips} = 119\% \text{ UDL}
Understanding Tables

• UDL Cautions:
  
  – For Point Loads

\[ \phi V_{u, \text{max}} = \phi(0.6)(F_v)(A_w)(C_v) \]
\[ = 1(0.6)(50 \text{ ksi})(13.7 \text{ in.})(0.23 \text{ in.})(1) \]
\[ = 94.5 \text{ kips} = 190\% \text{ UDL} \]

User Note: All current ASTM A6 W, S and HP shapes except W44x230, W40x149, W16x135, W33x118, W30x90, W24x55, W16x26 and W12x14 meet the criteria stated in Section G2.1(a) for \( F_v = 50 \text{ ksi} \) (345 MPa).

Understanding Tables

• Working backwards:
  
  – Given: Connection strength, Find: \( \phi V_{n, \text{connection}} \)

Minimum beam length, \( l_{\text{min}} \)

\[ l_{\text{min, flexure}} = \frac{\phi M_{\phi}(8)(\% \text{ UDL})}{100\%(\phi V_{n, \text{connection}})} \]

\[ l_{\text{min, shear}} = \frac{\phi M_{\phi}(8)}{\phi V_{\phi}(2)} \]

\[ l_{\text{min}} = \max(l_{\text{min, flexure}}, l_{\text{min, shear}}) \]
**Understanding Tables**

- Working backwards:
  - Given: Connection strength, $\phi V_{u,connection} = 71$ kip
  - Find: Minimum beam length, $l_{min}$

  \[
  l_{min, flexure} = \frac{\phi M_u (8)(\%UDL)}{100(\phi V_{u,connection})} = \frac{(125 \text{ kip-ft})(8)(50\%)}{100(71 \text{ kip})} = 7.04 \text{ ft}
  \]

  \[
  l_{min, shear} = \frac{\phi M_u (8)}{\phi V_u (2)} = \frac{(125 \text{ kip-ft})(8)}{(94.5 \text{ kip})(2)} = 5.29 \text{ ft}
  \]

  \[
  l_{min} = \max(l_{min, flexure}, l_{min, shear}) = 7.04 \text{ ft}
  \]

---

**Understanding Tables**

**Maximum Total Uniform Load Tables**

**Table 3-6. W-Shapes—Maximum Total Uniform Load**

Maximum total uniform loads on braced ($L_o \leq L$) simple-span beams bent about the strong axis are given for W-shapes with $F_y = 50$ ksi (ASTM A992). The uniform load constant, $\phi W_o$, or $W_o/\phi L_b$ (kip-ft), divided by the span length, $L (l)$, provides the maximum total uniform load (kips) for a braced simple-span beam bent about the strong axis. This is based on the available flexural strength as discussed for Table 3-2.

The strong-axis available shear strength, $\phi V_u$ or $V_u/\phi L_b$, can be determined using the tabulated value. Above the heavy horizontal line in the tables, the maximum total uniform load is limited by the strong-axis available shear strength.

The tabulated values can also be used for braced simple-span beams with equal concentrated loads spaced as shown in Table 3-22a if the concentrated loads are first converted to an equivalent uniform load.
Efficient Use of Tables

Instantaneous Center of Rotation Method
Spandrel example

- Given:
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W Shapes ASTM A992, $F_y = 50$ ksi
  - 3/8" plates, grade 50, $F_y = 50$ ksi
  - 7/8" dia. A325-N, STD holes, $\phi_r = 24.3$ kip
  - Bolt spacing: 3" vertical, 3" horizontal
  - Load: Façade moment $M_f = 100$ kip-in

\[ I_x = \sum_{i=1}^{6} x_i^2 = 6(1.5 \text{ in.})^2 \]
\[ = 13.5 \text{ in}^2 \]

\[ I_y = \sum_{i=1}^{6} y_i^2 = 4(3 \text{ in.})^2 \]
\[ = 36 \text{ in}^2 \]

\[ I_p = I_x + I_y \]
\[ = 49.5 \text{ in}^2 \]
Efficient Use of Tables

\[ R_x = \frac{M_{x,\text{max}}(y)}{I_p} = \frac{(100 \text{kip-in})(1.5 \text{in})}{49.5 \text{in}^2} = 3.03 \text{kip} \]

\[ R_y = \frac{M_{y,\text{max}}(x)}{I_p} = \frac{(100 \text{kip-in})(3.0 \text{in})}{49.5 \text{in}^2} = 6.06 \text{kip} \]

\[ R = \sqrt{R_x^2 + R_y^2} = \sqrt{(3.03 \text{kip})^2 + (6.06 \text{kip})^2} = 6.78 \text{kip} \]

Efficient Use of Tables

\[ M_{\text{max},1} = M_f \left( \frac{\phi r_n}{R} \right) = 100 \text{kip-in} \left( \frac{24.3 \text{kips}}{6.78 \text{kips}} \right) = 360 \text{kips-in.} \]

Use the Instantaneous Center of Rotation Method with \( C' \)

\( C' = 15.8 \) (From Table 7-7)

\[ M_{\text{max},2} = C'(\phi r_n) = (15.8 \text{in})(24.3 \text{kips}) = 384 \text{kip-in.} \]

\[ \frac{M_f}{R} = \frac{100 \text{kip-in}}{6.78 \text{in.}} = 14.8 \text{in.} \leq C' = 15.8 \text{ in} \]
Efficient Use of Tables

\[ C' = \sum l_i \left( 1 - e^{-\left( \frac{100}{\lambda_{max}} \right)} \right)^{0.55} \text{ in.} \quad \text{Manual Eq. (7-21)} \]

Efficient Use of Tables

- Façade and Gravity Moments Should Act Opposite

\[ \begin{align*}
M_r + M_f &= V_g + V_r \\
\text{FACADE} &+ \text{GRAVITY}
\end{align*} \]
Efficient Use of Tables

\[ \text{FACADE} \quad \rightarrow \quad M_f \quad M_f \]
\[ \text{GRAVITY} \quad \rightarrow \quad V_g \quad e \quad \quad \frac{M_f}{V_g} \quad e' = e + \frac{M_f}{V_g} \]
\[ \text{TOTAL} \quad \rightarrow \quad V_g \quad M_g \]

Efficient Use of Tables

\[ \text{NON EXTENDED SPANDREL CONNECTIONS} \]
Efficient Use of Tables

- Pick a clevis: 2" rod, \( F_y = 36 \text{ ksi} \), full strength
  \[
  A_{rod} = \frac{\pi (d_{rod}^2)}{4} = \frac{\pi (2 \text{ in.})^2}{4} = 3.14 \text{ in.}^2
  \]
  \[
  T_{max} = \phi A_{rod} F_y = 0.9 \left( 3.14 \text{ in.}^2 \right) (36 \text{ ksi}) = 102 \text{ kips}
  \]
  Select Clevis Number 6,
  Design strength = 135 kips \( \geq 102 \text{ kips} \)

Secrets of Manual

- Check Double-Angle Tolerance using Fig 10-3

Fig. 10-3. Fillet encroachment (riding the fillet).
Secrets of Manual

- Check Double-Angle Tolerance using Fig 10-3

\[
W_{\text{max}} = T + (2) E_{\text{cr}} = 6 \frac{1}{8} \text{ in.} + (2) \frac{3}{16} \text{ in.} = 6 \frac{1}{2} \text{ in.}
\]

\[
W = t_w + (2) h_{\text{angle}} = \frac{7}{16} \text{ in.} + (2) 3 \text{ in.} = 6 \frac{7}{16} \text{ in.}
\]

\[
W \leq W_{\text{max}} \text{ o.k.}
\]

Efficient Use of Tables

Determine Strength of Column at Grid B/1

2nd Floor, TOS=10'-0"

3rd Floor, TOS=25'-0"

Roof, TOS=40'-0"
Determine Strength of Column at Grid B/1

- **Given:**
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W14x90 ASTM A992, $F_y = 50$ ksi
    - $r_x = 6.14$ in.
    - $r_y = 3.70$ in.
  - $L_x = 30$ ft, $L_y = 15$ ft
  - $K = 1.0$

---

Determine Unbraced Length

- Since unbraced length differs in the two axes, select member based on $y$-$y$ axis then check equivalent $y$-$y$ axis length for $x$-$x$ axis.

\[
\begin{align*}
KL_x &= KL_y \\
KL_x &= \frac{KL_y}{r_y} \\
KL_y &= \frac{KL_x}{r_x} \\
KL_{y,EQ} &= \left( \frac{r_x}{r_y} \right) \frac{KL_x}{K}
\end{align*}
\]
Efficient Use of Tables

• Determine Unbraced Length

\[
\frac{r_x}{r_y} = \frac{6.14 \text{ in.}}{3.70 \text{ in.}} = 1.66 \quad \text{(See Table T4-1a)}
\]

\[
KL_{y,EQ} = KL_x \cdot \frac{\frac{r_x}{r_y}}{1.66} = 30.0 \text{ ft} = 18.1 \text{ ft} \quad \text{(Controls)}
\]

\[
KL_y = 15.0 \text{ ft}
\]
Efficient Use of Tables

• Check Using Specification

\[ \frac{KL_x}{r_x} = \frac{1(30.0 \text{ ft})(12 \text{ in.}/\text{ft})}{6.14 \text{ in.}} = 58.6 \]  

\[ \frac{KL_y}{r_y} = \frac{1(15 \text{ ft})(12 \text{ in.}/\text{ft})}{3.7 \text{ in.}} = 48.6 \]

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

The nominal compressive strength, \( F_n \), shall be determined based on the limit state of flexural buckling.

\[ F_n = F_{cr} A_g \]  

The critical stress, \( F_{cr} \), is determined as follows:

(a) When \( \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \) 

\[ F_{cr} = \frac{0.658 F_y}{K_L} \]  

(b) When \( \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} \) 

\[ F_{cr} = 0.877 F_y \]

where

\( F_y \) = elastic buckling stress determined according to Equation E3-4, as specified in Appendix 7, Section 7.2.2(b), or through an elastic buckling analysis, as applicable; ksi (MPa)

\[ F_{cr} = \frac{\pi^2 E}{KL} \]  

(Controls)

\[ x \]

\[ y \]

\[ r \]

\[ E \]

\[ F \]

\[ \pi \]

\[ \phi \]

\[ \phi F_{cr} = (0.9)38.9 \text{ ksi} = 35.0 \text{ ksi} \]

\[ \phi P_{cr} = 0.9 F_{cr} A_y = (35.0 \text{ ksi})(26.5 \text{ in.}^2) = 927 \text{ kips} = 926 \text{ kips} \quad \text{o.k.} \]
**Efficient Use of Tables**

Determine Strength of Beam Between Grids A/2 to B/2

- **Given:**
  - AISC Specification 360-10
  - AISC 14th Edition, LRFD
  - W18x35 ASTM A992, $F_y = 50$ ksi
  - $L = 30$ ft
  - Braced points only at beam-to-girder connections
  - Equally spaced bays
  - $C_b = 1.0$, conservative

---

**Efficient Use of Tables**

- From Table 3-2
  
  $L_p = 4.31$ ft
  
  $L_r = 12.3$ ft

  $y - y_0 = \frac{y_1 - y_0}{x_1 - x_0}$

  Solving for $y$: $y = y_0 + (y_1 - y_0) \frac{x - x_0}{x_1 - x_0}$

  $\phi M_{xx} = \phi M_{px} + \left( \phi M_{xx} - \phi M_{px} \right) \frac{L - L_p}{L_r - L_p}$

  $= 249$ kip-ft + $(151$ kip-ft $- 249$ kip-ft) $\left( \frac{10 \text{ ft} - 4.31 \text{ ft}}{12.3 \text{ ft} - 4.31 \text{ ft}} \right)$ $= 179$ kip-ft
Efficient Use of Tables

• Check if Compact for Flexure

<table>
<thead>
<tr>
<th>Shape</th>
<th>$W_{18}$</th>
<th>$W_{12}$</th>
<th>$W_{16}$</th>
<th>$W_{14}$</th>
<th>$W_{10}$</th>
<th>$W_{8}$</th>
<th>$W_{12}$</th>
<th>$W_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$W_{18}$:35</td>
<td>448</td>
<td>437</td>
<td>426</td>
<td>415</td>
<td>404</td>
<td>393</td>
<td>382</td>
<td>371</td>
</tr>
<tr>
<td>$W_{12}$:45</td>
<td>300</td>
<td>290</td>
<td>280</td>
<td>270</td>
<td>260</td>
<td>250</td>
<td>240</td>
<td>230</td>
</tr>
<tr>
<td>$W_{16}$:36</td>
<td>250</td>
<td>240</td>
<td>230</td>
<td>220</td>
<td>210</td>
<td>200</td>
<td>190</td>
<td>180</td>
</tr>
<tr>
<td>$W_{14}$:38</td>
<td>200</td>
<td>190</td>
<td>180</td>
<td>170</td>
<td>160</td>
<td>150</td>
<td>140</td>
<td>130</td>
</tr>
<tr>
<td>$W_{10}$:49</td>
<td>150</td>
<td>140</td>
<td>130</td>
<td>120</td>
<td>110</td>
<td>100</td>
<td>90</td>
<td>80</td>
</tr>
<tr>
<td>$W_{8}$:58</td>
<td>100</td>
<td>90</td>
<td>80</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>$W_{12}$:40</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>20</td>
<td>10</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$W_{10}$:45</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
</tbody>
</table>

1. Shape exceeds compact limit for flexure with $f_y = 50$ ksi.
2. Shape does not meet the $h/b_p$ limit for shear in AISC Specification Section 5.2.1(a) with $f_y = 50$ ksi; therefore, $\phi_s = 0.99$ and $\Omega_{uy} = 1.67$.

Efficient Use of Tables

• From Manual Table 3-2, $C_b = 1$ (See Table 3-1 for other $C_b$)
Efficient Use of Tables

F1. GENERAL PROVISIONS

The design flexural strength, \( \phi_b M_{n} \), and the allowable flexural strength, \( M_{u}/\Omega_b \), shall be determined as follows:

1. For all provisions in this chapter
   \( \phi_b = 0.90 \) (LRFD)
   \( \Omega_b = 1.67 \) (ASD)

   and the nominal flexural strength, \( M_{n} \), shall be determined according to Sections F2 through F13.

2. The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.

3. For singly symmetric members in single curvature and all doubly symmetric members:

   \[ C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_C} \]  

   (F1-1)

Table 3-1

<table>
<thead>
<tr>
<th>Lateral Bracing</th>
<th>( C_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Along Span</td>
<td></td>
</tr>
<tr>
<td>None</td>
<td></td>
</tr>
<tr>
<td>Load at midpoint</td>
<td>0.75</td>
</tr>
<tr>
<td>At load point</td>
<td>0.75</td>
</tr>
<tr>
<td>None</td>
<td>0.75</td>
</tr>
<tr>
<td>Load at third point</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Efficient Use of Tables

• Deflection

Table 3-23

Shears, Moments and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD

Total Equiv. Uniform Load \( wL \)

\( R = \frac{wL}{2} \)

\( V_x = \frac{wL}{2} \left( 1 - \frac{x}{L} \right) \)

\( M_{\text{max}} \) (at center) \( = \frac{wL^2}{8} \)

\( M_x \) (at center) \( = \frac{wL}{2} \left( 1 - \frac{x}{L} \right) \)

\( \Delta_{\text{max}} \) (at center) \( = \frac{wL^4}{24EI} \left( \frac{1}{2} + \frac{x}{L} \right) \)

\( \Delta_x \) \( = \frac{wL^4}{24EI} \left( \frac{1}{2} + \frac{x}{L} \right) \)
Efficient Use of Tables

1. Yielding

\[ M_y = M_p = F_y S_y \]  \hspace{1cm} (F2-1)

where

- \( F_y \) = specified minimum yield stress of the type of steel being used, ksi (MPa)
- \( S_y \) = plastic section modulus of the type of steel being used, in.\(^3\) (mm\(^3\))

2. Lateral-Torsional Buckling

(a) When \( L_0 \leq L_p \), the limit state of lateral-torsional buckling does not apply.

(b) When \( L_p < L_0 \leq L_r \)

\[ M_n = C_0 \left[ M_p - \left( M_p - 0.7F_y S_t \left( \frac{L_0 - L_p}{L_0 - L_r} \right) \right) \right] \leq M_p \]  \hspace{1cm} (F2-2)

(c) When \( L_0 > L_r \)

\[ M_n = F_x S_t \leq M_p \]  \hspace{1cm} (F2-3)

where

- \( L_0 \) = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

\[ F_x = \frac{C_0 \pi^2 E}{L_0 \left( \frac{r_0}{S_t} \right)^2} + 0.078 \frac{J_c}{S_t b_0} \left( \frac{L_0}{r_0} \right)^2 \]  \hspace{1cm} (F2-4)

Efficient Use of Tables

- From Specification:

The limiting lengths \( L_p \) and \( L_r \) are determined as follows:

\[ L_p = 1.76r_0 \sqrt{\frac{E}{F_y}} \]  \hspace{1cm} (F2-5)

\[ L_r = 1.95r_0 \sqrt{\frac{J_c}{0.7F_y S_t b_0} \left( \frac{F_y}{E} \right)^2 + 6.76 \left( \frac{0.7 F_y}{E} \right)^2} \]  \hspace{1cm} (F2-6)

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

\[ M_n = M_p - \left( M_p - 0.7F_y S_t \right) \left( \frac{\lambda - \lambda_{eff}}{\lambda_{eff} - \lambda} \right) \]  \hspace{1cm} (F3-1)

(b) For sections with slender flanges

\[ M_n = \frac{0.9E_b S_t}{\lambda^2} \]  \hspace{1cm} (F3-2)
Efficient Use of Tables


Fig. 5.1  Beam buckling curves.

Efficient Use of Tables

- *Manual* Fig 3-1 (also see Commentary Figure C-F1.2)

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Efficient Use of Tables

Determine Strength of Composite Beam (shown)

• Given:
  – W16x26 ASTM A992, $F_y = 50$ ksi, $L = 30$ ft
  – 3” NLWT concrete ($f'_c = 4$ ksi) on 3” metal deck
  – ¾” dia. stud

[Diagram of composite beam]

Determine Effective Width of Concrete Slab (Specification I3.1a)

Each side of beam centerline, minimum of:

1) $1/8$ of span = $\frac{L_b}{8} = \frac{(30 \text{ ft})(12 \text{ in/ft})}{8} = 45$ in., controls

2) $1/2$ c.c. beam spacing = $\frac{b_o}{2} = \left(\frac{1}{2}\right)\frac{(30 \text{ ft})(12 \text{ in/ft})}{(3 \text{ spaces})} = 60$ in.

3) distance to edge of slab = N/A

For this example, $b_{eff} = (2 \text{ sides of beam centerline})(45 \text{ in}) = 90$ in.

[Diagram showing calculation of $b_{eff}$]
Efficient Use of Tables

- Load Transfer Between Steel Beam and Concrete Slab (*Specification* I3.2d(1), Positive Flexural Strength where concrete slab is in compression)

  (a) Concrete crushing (concrete fully in compression)
  \[ V'_{c} = 0.85f'_c A_c = 0.85(4 \text{ ksi})(3 \text{ in})(90 \text{ in.}) = 918 \text{ kips} \]
  *(Note: only concrete above deck considered.)*

  (b) Tensile yielding of steel section (steel fully in tension)
  \[ V'_{s} = A_s F_y = (50 \text{ ksi})(7.68 \text{ in}^2) = 384 \text{ kips} < V'_{c} \quad \therefore \text{PNA in concrete} \]

  (c) Shear strength of steel studs
  \[ V'_{q} = \sum Q_n = \min(V'_{c}, V'_{s}) \text{ for } 100\% \text{ composite} \]

---

Efficient Use of Tables

- Stud Strength (*Specification* Eq. I8-1)

  \[ Q_a = 0.5 A_{a_s} \sqrt{f'_c E_c} \leq R_{g} R_{p} A_{a_s} F_u \]
  \[ A_{a_s} = \text{area of stud, in}^2 \]
  \[ E_c = \text{concrete modulus of elasticity} = w_{c}^{1/2} \sqrt{f'_c}, \text{ksi} \]
  \[ F_u = \text{specified minimum tensile strength of stud} = 65 \text{ ksi} \]

  \[ Q_a = 0.5 \left(0.4418 \text{ in}^2\right) \sqrt{(4 \text{ ksi}) \left[(145 \text{ pcf})^{1/2} (4 \text{ ksi})\right]^{1/2}} \]
  \[ = 26.1 \text{ kips/stud} \]

  \[ R_{g} R_{p} A_{a_s} F_u = 1.0(0.6)(0.4418 \text{ in}^2)(65 \text{ ksi}) \]
  \[ = 17.2 \text{ kips/stud, controls} \]
Efficient Use of Tables

• Use Table 3-21 for Stud Strength

![Table 3-21 Shear Stud Anchor](image)

Efficient Use of Tables

• Determine Number of Studs

For 100% composite, studs required between points of maximum and zero bending moment (half span of beam):

\[
\text{# of studs} = \left\lceil \frac{V'_{q} = \min(V'_{c}, V'_{s})}{Q_{n}} \right\rceil = \frac{384 \text{ kips}}{17.2 \text{ kips/stud}} = 22.3 \quad \therefore 46 \text{ studs for beam total length}
\]

However, for 46 studs, there will be (2) studs per rib. For (2) studs per rib,

\[
R_{g} = 0.85 \quad Q_{n} = 14.6 \text{ kips/stud}
\]

bending moment (half span of beam):

\[
\text{# of studs} = \left\lceil \frac{V'_{q} = \min(V'_{c}, V'_{s})}{Q_{n}} \right\rceil = \frac{384 \text{ kips}}{14.6 \text{ kips/stud}} = 26.3 \quad \therefore 54 \text{ studs for beam total length}
\]

*See Design Example I.1 on page I-15 for more information*
**Efficient Use of Tables**

- Determine 100% Composite Strength

\[
a = \frac{\sum q_n}{0.85 f'_c b_{eff}} = \frac{384 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 1.25 \text{ in.}
\]

\[
Y_2 = (3 \text{ in. conc.} + 3 \text{ in. deck}) - \frac{1.25 \text{ in.}}{2} = 5.38 \text{ in.}
\]

- Sum moments about \( a/2 \):

\[
\phi_b M_n = \phi A_y f_y \left( Y_2 + \frac{d_{bm}}{2} \right) = 0.9 \left[ \frac{384 \text{ kips}}{12 \text{ in./ft}} \left( 5.38 \text{ in.} + \frac{15.7 \text{ in.}}{2} \right) \right] = 381 \text{ kip-ft}
\]

---

**Efficient Use of Tables**

- For Use with Table 3-19 (see also Figure 3-3 in *Manual*)

\[
Y_1 = \text{distance from top of steel flange to any of the PNA locations listed}
\]

PNA locations:
- TFL = Top of Flange (beam top flange), also 1
- BFL = Bottom of Flange (beam top flange), also 5
- 2,3,4 = 4 equal spaces within flange

\[
6 = \frac{\sum q_n \text{ at BFL} + \sum q_n \text{ at 7}}{2}
\]

\[
7 = 0.25 F_y A_y
\]

\[
Y_2 = Y_{con} = \frac{a}{2} \quad a = \frac{\sum q_n}{0.85 f'_c b_{eff}}
\]

and \( Y_{con} = \text{top of steel to top of concrete} \)
Efficient Use of Tables

- Table 3-19

From above,

\[ V'_{c} = 918 \text{ kips} \]
\[ V'_{s} = 384 \text{ kips} \]

Enter Table with PNA at TFL and

\[ \sum Q_n = 384 \text{ kips (100\% composite)} \]

\[ a = \frac{\sum Q_n}{0.85 f'_c b_{ef}} = \frac{384 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 1.25 \text{ in.} \]

\[ Y2 = (3 \text{ in. conc. + 3 in. deck}) - \frac{1.25 \text{ in.}}{2} = 5.38 \text{ in.} \]

Efficient Use of Tables

- Moment Capacity based on Table 3-19 (100\% composite)

\[ \phi_b M_n = 384 \text{ kft} - (384 \text{ kft} - 370 \text{ kft}) \left( \frac{5.5 \text{ in.} - 5.38 \text{ in.}}{5.5 \text{ in.} - 5 \text{ in.}} \right) = 381 \text{ kip-ft} \]

- Compare to

\[ \phi_b M_n = \phi A_y F_y \left( Y2 + \frac{d_{bm}}{2} \right) = 0.9 \left( \frac{384 \text{ kips}}{12 \text{ in./ft}} \left( \frac{5.38 \text{ in.} + 15.7 \text{ in.}}{2} \right) \right) = 381 \text{ kip-ft} \]
Efficient Use of Tables

Using Table 3-19 for 63% composite

For 63% composite, enter Table with

\[ \sum Q_{n,63\%} = (0.63)(384 \text{ kips}) = 242 \text{ kips} \]

\[ a = \frac{\sum Q_{n,63\%}}{0.85 f'c b_{\text{eff}}} = \frac{242 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 0.791 \text{ in.} \]

\[ Y2 = 6 \text{ in.} - \frac{0.791 \text{ in.}}{2} = 5.60 \text{ in.} \]

---

Efficient Use of Tables

Using Table 3-19 for 63% composite

For 63% composite, enter Table with

\[ \sum Q_{n,63\%} = (0.63)(384 \text{ kips}) = 242 \text{ kips} \]

Note: PNA at 4.

\[ \phi_b M_{n,63\%} = 333 \text{ kft} - (333 \text{ kft} - 324 \text{ kft}) \left( \frac{6 \text{ in.} - 5.6 \text{ in.}}{6 \text{ in.} - 5.5 \text{ in.}} \right) = 326 \text{ kip-ft} \]
Efficient Use of Tables

- Determine Moment Capacity using 63% Composite
  From AISC Commentary Section C-I3.2a and Equation C-I3-10,

\[ M_u = C(d_1 + d_2) + P_y (d_3 - d_2) \]

AISC Commentary Eq. C-I3-10

For partial composite action, there is a neutral axis in the concrete and a neutral axis in the steel.

Efficient Use of Tables

- Determine Moment Capacity using 63% Composite (cont’d)

where:

\[ C = \sum Q_n \]
\[ P_y = A_y F_y \]
\[ d_1 = \text{distance from centroid of compression force, } C, \text{ in concrete to top of steel, in.} \]
\[ d_2 = \text{distance from centroid of the compression force in steel section to top of steel section, in.} \]
\[ d_3 = \text{distance from } P_y \text{ to top of steel section, in.} \]

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Efficient Use of Tables

- Determine Moment Capacity using 63% Composite (cont’d)

\[ P_y = 384 \text{ kips} \]

\[ C_{63\%} = \sum Q_n \cdot 63\% = 0.63(384 \text{ kips}) = 242 \text{ kips} \]

\[ a = \frac{C_{63\%}}{0.85 f_c' \cdot b_{eff}} = \frac{242 \text{ kips}}{0.85(4 \text{ ksi})(90 \text{ in.})} = 0.791 \text{ in.} \]

\[ d_1 = t_{slab} - \frac{a}{2} = 6 \text{ in.} - \frac{0.791 \text{ in.}}{2} = 5.60 \text{ in.} \]

\[ d_3 = \frac{d_{hm}}{2} = \frac{15.7 \text{ in.}}{2} = 7.85 \text{ in.} \]

\[ d_2 = \frac{x}{2} = \frac{1}{2} \left( \frac{A_x F_y - C_{63\%}}{2 b_j F_y} \right) = \frac{1}{2} \left( \frac{384 \text{ kips} - 242 \text{ kips}}{2(5.50 \text{ in.})(50 \text{ ksi})} \right) = 0.129 \text{ in.} \]

\[ M_{n \cdot 63\%} = C_{63\%}(d_1 + d_2) + P_y(d_3 - d_2) \]

\[ = (242 \text{ kips})(5.60 \text{ in.} + 0.129 \text{ in.}) + (384 \text{ kips})(7.85 \text{ in.} - 0.129 \text{ in.}) \]

\[ = 4351 \text{ k-in.} \]

\[ \phi M_{n \cdot 63\%} = \frac{0.9(4351 \text{ kip-in})}{12} = 326 \text{ kip-ft} \]
Efficient Use of Tables

- Determine Number of Studs

For 63% composite,

\[ \text{# of studs} = \frac{0.63 \times 384 \text{ kips}}{17.2 \text{ kips/stud}} = 14.1 \]

\[ \therefore 30 \text{ studs for beam total length} \]

*Note:* Portion of steel beam will be compression.

Summary

- Be familiar with the information contained in the *Manual*

- Use the Database when possible

- Read descriptions of the tables

- Include all design information on details to facilitate review and checking

- *Manual* can facilitate efficient design.
Polling Question

PDH Certificates

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• You will receive an email on how to report attendance from: registration@aisc.org.
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PDH Certificates

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