steel construction: columns & tension members

Design Methods (revisited)
- know
  - loads or lengths
- select
  - section or load
  - adequate for strength and no buckling

Allowable Stress Design (ASD)
- AICS 9th ed
  \[ F_a = \frac{f_{critical}}{F.S.} = \frac{12\pi^2E}{23(Kl/r)^2} \]
  - slenderness ratio \( \frac{Kl}{r} \)
    - for \( kl/r \geq C_c \):
      \[ F_a = \frac{f_{critical}}{F.S.} = \frac{12\pi^2E}{23(Kl/r)^2} \]
      - with \( F_y = 36 \text{ ksi} \)
        \[ = 126.1 \]
      - with \( F_y = 50 \text{ ksi} \)
        \[ = 107.0 \]
**C_c and Euler’s Formula**

- **Kl/r < C_c**
  - short and stubby
  - parabolic transition

- **Kl/r > C_c**
  - Euler’s relationship
  - < 200 preferred

\[ C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \]

**Short / Intermediate**

- **L_e/r < C_c**

\[ F_a = 1 - \left( \frac{Kl/r}{2C_c^2} \right)^2 \frac{F_y}{F.S.} \]

- where

\[ F.S. = \frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3} \]

**Unified Design**

- limit states for failure

\[ P_a \leq \frac{P_n}{\phi_c} \]

\[ \phi_c = 0.90 \quad P_n = F_{cr} A_g \quad P_u \leq \phi_c P_n \]

1. **yielding**

\[ \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F}} \quad \text{or} \quad F_e \geq 0.44F_y \]

2. **buckling**

\[ \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F}} \quad \text{or} \quad F_e < 0.44F_y \]

\[ F_e \text{ – elastic buckling stress (Euler)} \]
Unified Design

- \( P_n = F_{cr} A_g \)
  - for \( \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} F_{cr} = \left[ \frac{0.658}{F_e} \right] F_y \)
  - for \( \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} F_{cr} = 0.877 F_e \)
  - where \( F_e = \frac{\pi^2 E}{(KL/r)^2} \)

Procedure for Analysis

1. calculate \( KL/r \)
   - biggest of \( KL/r \) with respect to \( x \) axes and \( y \) axis
2. find \( F_a \) or \( F_{cr} \) from appropriate equation
   - tables are available
3. compute \( P_{\text{allowable}} = F_a A \) or \( P_n = F_{cr} A_g \)
   - or find \( f_{\text{actual}} = P/A \)
4. is \( P \leq P_{\text{allowable}} \) (or \( P_u \leq \phi P_n \)?)
   - yes: ok
   - no: insufficient capacity and no good

Procedure for Design

1. guess a size (pick a section)
2. calculate \( KL/r \)
   - biggest of \( KL/r \) with respect to \( x \) axes and \( y \) axis
3. find \( F_a \) or \( F_{cr} \) from appropriate equations
   - or find a chart
4. compute \( P_{\text{allowable}} = F_a A \) (or \( P_n = F_{cr} A_g \))
   - or find \( f_{\text{actual}} = P/A \)
5. is \( P \leq P_{\text{allowable}} \) (or \( P_u \leq \phi P_n \)?)
   - yes: ok
   - no: pick a bigger section and go back to step 2.
6. check design efficiency
   - percentage of stress = \( \frac{P_r}{P_c} \cdot 100\% \)
   - if between 90-100%: good
   - if < 90%: pick a smaller section and go back to step 2.
Column Charts, $F_a$ (pg. 361-364)

Table 10.1 Allowable stress for compression members ($F_y = 36$ ksi and $F_v = 250$ ksi).

<table>
<thead>
<tr>
<th>$KL$</th>
<th>$r$</th>
<th>$F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>9</td>
<td>0</td>
<td>312</td>
</tr>
</tbody>
</table>

Column Charts, $\phi F_{cr}$

Available Critical Stress, $\phi F_{cr}$, for Compression Members, ksi ($F_y = 60$ ksi and $\phi = 0.90$).

<table>
<thead>
<tr>
<th>$KL/r$</th>
<th>$\phi F_{cr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45.0</td>
</tr>
<tr>
<td>2</td>
<td>45.0</td>
</tr>
<tr>
<td>3</td>
<td>45.0</td>
</tr>
<tr>
<td>4</td>
<td>44.9</td>
</tr>
<tr>
<td>5</td>
<td>44.9</td>
</tr>
<tr>
<td>6</td>
<td>44.8</td>
</tr>
<tr>
<td>7</td>
<td>44.8</td>
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<td>8</td>
<td>44.8</td>
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<tr>
<td>9</td>
<td>44.7</td>
</tr>
<tr>
<td>10</td>
<td>44.7</td>
</tr>
</tbody>
</table>

Beam-Column Design

- moment magnification ($P-\Delta$)

$$M_u = B_1 M_{max} - \text{factored} \quad B_1 = \frac{C_m}{1 - (P_u / P_{el})}$$

$C_m$ – modification factor for end conditions

$P_{el} = \frac{\pi^2 EA}{(KL/r)^2}$

Steel Columns & Tension 11
Foundations Structures
ARCH 331
Lecture 20
F2008abn

Steel Columns & Tension 14
Foundations Structures
ARCH 331
Lecture 20
S2012abn
Beam-Column Design

• LRFD (Unified) Steel
  – for \( \frac{P_r}{P_c} \geq 0.2 : \quad \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \)
  – for \( \frac{P_r}{P_c} < 0.2 : \quad \frac{P_u}{2\phi_c P_n} + \frac{9}{8} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \)

\( P_r \) is required, \( P_c \) is capacity

\( \phi_c \) - resistance factor for compression = 0.9

\( \phi_b \) - resistance factor for bending = 0.9

Design Steps Knowing Loads (revisited)

1. assume limiting stress
   • buckling, axial stress, combined stress
2. solve for \( r, A \) or \( S \)
3. pick trial section
4. analyze stresses
5. section ok?
6. stop when section is ok

Rigid Frame Design (revisited)

• columns in frames
  – ends can be “flexible”
  – stiffness affected by beams and column = \( EI/L \)
  \[ G = \Psi = \frac{\Sigma EI}{l_c} \]
  – for the joint
    • \( l_c \) is the column length of each column
    • \( l_b \) is the beam length of each beam
    • measured center to center
**Steel Columns & Tension**

**Lecture 20**

**Foundations Structures**

**ARCH 331**

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**Tension Members**

- steel members can have holes
- reduced area
  \[ A_n = A_g - A_{of \ all \ holes} + t \sum \frac{s^2}{4g} \]
- increased stress

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**Effective Net Area**

- likely path to “rip” across
- bolts divide transferred force too
- shear lag
  \[ A_e \leq A_n U \]

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**Tension Members**

- limit states for failure

1. yielding \( \phi_t = 0.90 \) \( P_n = F_y A_g \)
2. rupture* \( \phi_t = 0.75 \) \( P_n = F_u A_e \)

- \( A_g \) - gross area
- \( A_e \) - effective net area
- (holes 3/16” + d)
- \( F_u \) = the tensile strength of the steel (ultimate)