APPLIED ARCHITECTURAL STRUCTURES:
STRUCTURAL ANALYSIS AND SYSTEMS
ARCH 631
DR. ANNE NICHOLS
SPRING 2017

lecture
twenty one

steel construction
and design

http://nisee.berkeley.edu/godden
Steel

- cast iron – wrought iron - steel
- cables
- columns
- beams
- trusses
- frames

http://nisee.berkeley.edu/godden
Steel Construction

- standard rolled shapes
- open web joists
- plate girders
- decking
Steel Construction

- welding
- bolts
Steel Construction

- fire proofing
  - cementitious spray
  - encasement in gypsum
  - intumescent – expands with heat
  - sprinkler system
Steel Materials

- high strength to weight ratio
- ductile
- beam size often limited by deflection
- column size limited by slenderness

Figure 5.22  Stress-strain diagram for mild steel (A36) with key points highlighted.
Steel Beams

- types
  - manufactured shapes

http://nisee.berkeley.edu/godden

castellated
Steel Beams

- **types**
  - wide flange
Steel Beams

- types
  - open web joists (manufactured trusses)
Steel Beams

- types (more)
  - plate girder
  - decking

(a) Light-gage steel beam.
(b) Cover plated beam.
(d) Welded girder.
(e) Welded box girder.
Steel Beams

- Plate web
- Angle stiffeners
- Thicker flange in center where moment is greatest

Stiffeners:
- At end where shear is greatest
- And at support

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Steel Beams

- lateral stability - bracing
- local buckling - stiffen
Local Buckling

- steel I beams
- flange
  - buckle in direction of smaller radius of gyration
- web
  - force
  - “crippling”
Local Buckling

- **flange**
- **web**

*Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, K., 1964)*

*Figure 2-7. Web Local Buckling Limit State (SAC Project)*
Shear in Web

- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners
Steel Beams

- end conditions

- a) away from connection - full section effective

- b) high shear – only web effective
Steel Beams

• end conditions

  • c) bolt holes – less material

  • d) local web buckling
Steel Beams

• bearing
  – provide adequate area
  – prevent local yield of flange and web
Steel Beams

- connections
  - welds
  - bolts
Steel Design – Open Web Joists

- **SJI**: [www.steeljoist.com](http://www.steeljoist.com)
- **Vulcraft**: [www.vulcraft.com](http://www.vulcraft.com)
  - **K Series (Standard)**
    - 8-30” deep, spans 8-50 ft
  - **LH Series (Long span)**
    - 18-48” deep, spans 25-96 ft
  - **DLH (Deep Long Spans)**
    - 52-72” deep, spans 89-144 ft
  - **SLH (Long spans with high strength steel)**
    - pitched top chord
    - 80-120” deep, spans 111-240 ft
# Steel Design – Open Web Joists ASD

## Standard Load Table/Open Web Steel Joists, K-Series

Based on a Maximum Allowable Tensile Stress of 30 ksi

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<th>Joist Designation</th>
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**load for live load**

**deflection limit in **RED**

**total in **BLACK**
Steel Design – Open Web Joists

### Standard Load Table for Open Web Steel Joists, K-Series

Based on a 50 ksi maximum yield strength - Loads shown in pounds per linear foot (plf)

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Load for live load deflection limit (L/360) in **RED**
Total in **BLACK**
Steel Beam Design

• American Institute of Steel Construction
  – steel grades
    • ASTM A36 – carbon
      – plates, angles
      – $F_y = 36$ ksi & $F_u = 58$ ksi
    • ASTM A572 – high strength low-alloy
      – some beams
      – $F_y = 60$ ksi & $F_u = 75$ ksi
    • ASTM A992 – for building framing
      – most beams
      – $F_y = 50$ ksi & $F_u = 65$ ksi
Steel Beam Design

- AISC: 14th ed.
  - combined ASD & LRFD in one volume in 2005
Unified Steel Design

- **ASD**

\[ R_a \leq \frac{R_n}{\Omega} \]

- bending (braced) \( \Omega = 1.67 \)
- bending (unbraced*) \( \Omega = 1.67 \)
- shear \( \Omega = 1.5 \) or \( 1.67 \)
- shear (bolts & welds) \( \Omega = 2.00 \)
- shear (welds) \( \Omega = 2.00 \)

*flanges in compression can buckle*
Unified Steel Design

- braced vs. unbraced
LRFD Steel Beam Design

- limit state is yielding all across section
- outside elastic range
- load factors & resistance factors
Load Types

- $D =$ dead load
- $L =$ live load
- $L_r =$ live roof load
- $W =$ wind load
- $S =$ snow load
- $E =$ earthquake load
- $R =$ rainwater load or ice water load
- $T =$ effect of material & temperature
LRFD Load Combinations

- 1.4D
- 1.2D + 1.6L + 0.5(L_r or S or R)
- 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
- 1.2D + 1.0W + L + 0.5(L_r or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E

- F has same factor as D in 1-5 and 7
- H adds with 1.6 and resists with 0.9 (permanent)
Pure Flexure

\[ \sum \gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z \]

- \( M_u \) - maximum moment
- \( \phi_b \) - resistance factor for bending = 0.9
- \( M_n \) - nominal moment (ultimate capacity)
- \( F_y \) - yield strength of the steel
- \( Z \) - plastic section modulus*

*Plastic section modulus is a measure of the section's ability to resist plastic deformation under load.
Internal Moments - at yield

\[ M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y \]

\[ = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y \]
Internal Moments - ALL at yield

- all parts reach yield
- plastic hinge forms
- ultimate moment
- $A_{\text{tension}} = A_{\text{compression}}$

$$M_p = bc^2 f_y = \frac{3}{2} M_y$$

\[\sigma_y = 50\text{ksi} \quad \varepsilon_y = 0.001724\]
n.a. of Section at Plastic Hinge

- cannot guarantee at centroid
- \( f_y \cdot A_1 = f_y \cdot A_2 \)
- moment found from yield stress times moment area

\[
M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i
\]
Plastic Hinge Development

(a) $M < M_Y$

(b) $M = M_Y$

(c) $M > M_Y$

(d) $M = M_p$
Plastic Hinge Examples

- Stability can be effected
Plastic Section Modulus

- shape factor, $k$
  
  $k = \frac{M_p}{M_y}$
  
  $k = \frac{Z}{S}$

  $= 3/2$ for a rectangle

  $\approx 1.1$ for an I

- plastic modulus, $Z$
  
  $Z = \frac{M_p}{f_y}$
LRFD – Shear (compact shapes)

\[ \sum \gamma_i R_i = V_u \leq \phi_v V_n = 1.0(0.6 F_{yw} A_w) \]

- $V_u$ - maximum shear
- $\phi_v$ - resistance factor for shear = 1.0
- $V_n$ - nominal shear
- $F_{yw}$ - yield strength of the steel in the web
- $A_w$ - area of the web = $t_w d$
Flexure Design

- limit states for beam failure
  1. yielding
  2. lateral-torsional buckling
  3. flange local buckling
  4. web local buckling

- minimum $M_n$ governs

\[ L_p = 1.76 r_y \sqrt{\frac{F_y}{E}} \]

\[ \sum \gamma_i R_i = M_u \leq \phi_b M_n \]
Compact Sections

- plastic moment can form before any buckling
- criteria

\[- \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}}\]

- and \[\frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}\]
Lateral Torsional Buckling

\[ M_n = C_b \left[ \text{moment based on lateral buckling} \right] \leq M_p \]

\[ C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 2M_A + 4M_B + 3M_C} \]

\( C_b = \text{modification factor} \)

\( M_{\text{max}} = |\text{max moment}|, \text{ unbraced segment} \)

\( M_A = |\text{moment}|, 1/4 \text{ point} \)

\( M_B = |\text{moment}|, \text{ center point} \)

\( M_C = |\text{moment}|, 3/4 \text{ point} \)
Beam Design Charts

Table 3-10 (continued)

W Shapes

Available Moment vs. Unbraced Length

Available Moment, $\frac{M_e}{\Omega}$ (1 kip-ft increments) vs. $\phi M_e$ (1.5 kip-ft increments)

Unbraced Length (0.5-ft increments)
Deflection Limits

- based on service condition
- no “impairment” to serviceability
- avoid ponding
- $L/360$ due to live load for beams & girders supporting plaster (service)
Steel Arches and Frames

- solid sections
- or open web

http://nisee.berkeley.edu/godden
Steel Shell and Cable Structures
Approximate Depths

<table>
<thead>
<tr>
<th>Structure</th>
<th>Span (Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Decking</td>
<td>L/30-L/50</td>
</tr>
<tr>
<td>Wide flanges</td>
<td>L/18-L/28</td>
</tr>
<tr>
<td>Plate girders</td>
<td>L/15-L/20</td>
</tr>
<tr>
<td>Open-web joists</td>
<td>L/18-L/22</td>
</tr>
<tr>
<td>Fink truss</td>
<td>L/4-L/5</td>
</tr>
<tr>
<td>Howe truss</td>
<td>L/4-L/5</td>
</tr>
<tr>
<td>Bowstring truss</td>
<td>L/6-L/10</td>
</tr>
<tr>
<td>Special truss</td>
<td>L/4-L/15</td>
</tr>
<tr>
<td>Arches</td>
<td>L/3-L/5</td>
</tr>
<tr>
<td>Ribbed domes</td>
<td>L/3-L/5</td>
</tr>
<tr>
<td>Cables</td>
<td>L/5-L/11</td>
</tr>
<tr>
<td>Space frame (column-supported)</td>
<td>L/12-L/20</td>
</tr>
<tr>
<td>Space frame (wall-supported)</td>
<td>L/12-L/20</td>
</tr>
</tbody>
</table>

Key:
- Minimum span
- Possible span range
- Maximum span
- Typical span for member
- Typical member length
Unified Column Design

- limit states for failure

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

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

1. yielding \( \frac{Kl}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \) or \( F_e \geq 0.44F_y \)

2. buckling \( \frac{Kl}{r} > 4.71 \sqrt{\frac{E}{F_y}} \) or \( F_e < 0.44F_y \)

\( F_e \) – elastic buckling stress (Euler)
Unified Column Design

- \( P_n = F_{cr} A_g \)
  - for \( \frac{Kl}{r} \leq 4.71 \) \( \sqrt{\frac{E}{F_{y}}} \) \( F_{cr} = \left[ 0.658 \frac{F_{y}}{F_{e}} \right] F_{y} \)
  - for \( \frac{Kl}{r} > 4.71 \) \( \sqrt{\frac{E}{F_{y}}} \) \( F_{cr} = 0.877 F_{e} \)
  - where
    \( F_{e} = \frac{\pi^2 E}{\left( \frac{KL}{r} \right)^2} \)
Procedure for Analysis

1. calculate $KL/r$
   - biggest of $KL/r$ with respect to $x$ axes and $y$ axis

2. find $F_{cr}$ from appropriate equation
   - tables are available

3. compute $P_n = F_{cr}A_g$
   - or find $f_c = P_a/A$ or $P_u/A$

4. is $P_a \leq P_n/\Omega$ 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 $F_{cr}$ from appropriate equations
   • or find a table
4. compute $P_n = F_{cr}A_g$
   • or find $f_c = P_a/A$ or $P_u/A$
Procedure for Design (cont’d)

5. is $P_a \leq P_r/\Omega$ 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 Tables**

**Table 4–1 (continued)**

Available Strength in Axial Compression, kips

W Shapes

$F_y = 50 \text{ ksi}$

<table>
<thead>
<tr>
<th>Shape</th>
<th>Wt/ft</th>
<th>96</th>
<th>87</th>
<th>79</th>
<th>72</th>
<th>65</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Design</td>
<td>$P_n/\Omega_c$</td>
<td>$\phi_c P_n$</td>
<td>$P_n/\Omega_c$</td>
<td>$\phi_c P_n$</td>
<td>$P_n/\Omega_c$</td>
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<tr>
<td></td>
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<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
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<td>6</td>
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<td>1220</td>
<td>735</td>
<td>1110</td>
<td>667</td>
<td>1000</td>
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<tr>
<td>7</td>
<td>800</td>
<td>1200</td>
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<td>1090</td>
<td>657</td>
<td>987</td>
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<tr>
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<td>9</td>
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<td>699</td>
<td>1050</td>
<td>634</td>
<td>952</td>
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<tr>
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<td>685</td>
<td>1030</td>
<td>620</td>
<td>932</td>
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<tr>
<td>11</td>
<td>739</td>
<td>1110</td>
<td>669</td>
<td>1010</td>
<td>606</td>
<td>910</td>
</tr>
</tbody>
</table>
Beam-Column Design

- **moment magnification (P-Δ)**

\[
M_u = B_1 M_{\text{max - factored}} \quad B_1 = \frac{C_m}{1 - (P_u / P_{e1})}
\]

- **C_m** – modification factor for end conditions
  - \( = 0.6 - 0.4(M_1/M_2) \) or
  - 0.85 restrained, 1.00 unrestrained

- **P_{e1}** – Euler buckling strength
  \[
P_{e1} = \frac{\pi^2 EA}{(Kl/r)^2}
\]
Beam-Column Design

- LRFD (Unified) Steel

  - for \( \frac{P_r}{P_c} \geq 0.2 \): \[
  \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0
  \]

  - for \( \frac{P_r}{P_c} < 0.2 \): \[
  \frac{P_u}{2\phi_c P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0
  \]

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

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

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

• proper grade material
  – high strength bolts
• quality welds
• proper bolted conditions (ex. sc)
• fabrication and erection of steel frame connection details