Elements of Architectural Structures: Form, Behavior, and Design
ARCH 614
Dr. Anne Nichols
Spring 2013

Steel construction: materials & beams
Steel Beam Design

- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - combined in 2005
Steel Materials

• 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 Properties

- high strength to weight ratio
- elastic limit – yield ($F_y$)
- inelastic – plastic
- ultimate strength ($F_u$)
- ductile
- strength sensitive to temperature
- can corrode
- fatigue
Structural Steel

- standard rolled shapes (W, C, L, T)
- open web joists
- plate girders
- decking
Steel Construction

- welding
- bolts

connection at web only (flanges not connected)
flanges connected (bolted web connection to facilitate erection only)

SHEAR CONNECTION
MOMENT CONNECTION

http://courses.civil.ualberta.ca
Unified Steel Design

- **ASD**

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

- bending (braced) \[ \Omega = 1.67 \]
- bending (unbraced\(^*\)) \[ \Omega = 1.67 \]
- shear \[ \Omega = 1.5 \text{ or } 1.67 \]
- shear (bolts & welds) \[ \Omega = 2.00 \]
- shear (welds) \[ \Omega = 2.00 \]

\(^*\) flanges in compression can buckle
Unified Steel Design

- braced vs. unbraced
LRFD

• loads on structures are
  – not constant
  – can be more influential on failure
  – happen more or less often
  – UNCERTAINTY

\[ R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n \]

\( \phi \) - resistance factor
\( \gamma \) - load factor for (D)ead & (L)ive load
LRFD Steel Beam Design

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

\[
f_y = 50 \text{ksi} \quad \varepsilon_y = 0.001724 \]

\[\text{f} \quad \varepsilon\]

\[
f_y = 50 \text{ksi} \quad \varepsilon_y = 0.001724 \]

\[\text{f} \quad \varepsilon\]
LRFD Load Combinations

- $1.4D$
- $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
- $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S$
- $0.9D + 1.0W$
- $0.9D + 1.0E$
  - $F$ has same factor as $D$ in 1-5 and 7
  - $H$ adds with 1.6 and resists with 0.9 (permanent)
Beam Design Criteria (revisited)

• strength design
  – bending stresses predominate
  – shear stresses occur

• serviceability
  – limit deflection
  – stability

• superpositioning
  – use of beam charts
  – elastic range only!
  – “add” moment diagrams
  – “add” deflection CURVES (not maximums)
Steel Beams

- lateral stability - bracing
- local buckling – stiffen, or bigger $I_y$
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, R., 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
Shear in Web

- plate girders and stiffeners
Steel Beams

• bearing
  – provide adequate area
  – prevent local yield of flange and web
LRFD - 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*
Internal Moments - at yield

- material hasn’t failed

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

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

- All parts reach yield
- Plastic hinge forms
- Ultimate moment
- \( A_{\text{tension}} = A_{\text{compression}} \)

\[
M_p = b c^2 f_y = \frac{3}{2} M_y
\]
n.a. of Section at Plastic Hinge

- cannot guarantee at centroid
- \( f_y A_1 = f_y A_2 \)
- 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,** $k$
  
  $= \frac{3}{2}$ for a rectangle

  $\approx 1.1$ for an $I$

- **plastic modulus,** $Z$
  
  $Z = \frac{M_p}{f_y}$

\[ k = \frac{M_p}{M_y} \]

\[ k = \frac{Z}{S} \]
LRFD - Shear

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

- \( V_u \) - 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 = \( t_w d \)
LRFD - 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

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

- plastic moment can form before any buckling
- criteria

\[
- \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \\
- \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}
\]
Lateral Torsional Buckling

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

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

- \( C_b \) = modification factor
- \( M_{\text{max}} \) - |max moment|, unbraced segment
- \( M_A \) - |moment|, 1/4 point
- \( M_B \) = |moment|, center point
- \( M_C \) = |moment|, 3/4 point
# Beam Design Charts

**Table 3-10 (continued)**

## W Shapes

### Available Moment vs. Unbraced Length

<table>
<thead>
<tr>
<th>Available Moment, $M_p/\Omega$ (1 kip-ft increments)</th>
<th>$\phi M_p$ (1.5 kip-ft increments)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASD</td>
<td>LRFD</td>
</tr>
<tr>
<td>56</td>
<td>84</td>
</tr>
<tr>
<td>52</td>
<td>78</td>
</tr>
<tr>
<td>48</td>
<td>72</td>
</tr>
<tr>
<td>44</td>
<td>66</td>
</tr>
<tr>
<td>40</td>
<td>60</td>
</tr>
</tbody>
</table>

Unbraced Length (0.5-ft increments)
Charts & Deflections

• beam charts
  – solid line is most economical
  – dashed indicates there is another more economical section
  – self weight is NOT included in $M_n$

• deflections
  – no factors are applied to the loads
  – often governs the design
Design Procedure (revisited)

1. Know unbraced length, material, design method ($\Omega, \phi$)

2. Draw $V$ & $M$, finding $M_{\text{max}}$

3. Calculate $Z_{\text{req'd}}$ \( (M_a \leq M_n / \Omega) \)
   \( (M_u \leq \phi_b M_n) \)

4. Choose (economical) section from section or beam capacity charts
### TABLE 9.1 Load Factor Resistance Design Selection for Shapes Used as Beams

<table>
<thead>
<tr>
<th>Designation</th>
<th>$Z_x$ in.$^3$</th>
<th>$L_p$ ft</th>
<th>$L_r$ ft</th>
<th>$M_p$ kip-ft</th>
<th>$M_r$ kip-ft</th>
<th>$F_y = 36$ ksi</th>
<th>$F_y = 50$ ksi</th>
<th>$r_y$ in.</th>
<th>$b_y/2t_y$</th>
<th>$h_i t_w$</th>
<th>$X_i$ ksi</th>
<th>$X_s \times 10^6$ (1/ksi)$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W 33 $\times$ 141</td>
<td>514</td>
<td>10.1</td>
<td>30.1</td>
<td>1,542</td>
<td>971</td>
<td>8.59</td>
<td>23.1</td>
<td>2,142</td>
<td>1,493</td>
<td>2.43</td>
<td>6.01</td>
<td>49.6</td>
</tr>
<tr>
<td>W 30 $\times$ 148</td>
<td>500</td>
<td>9.50</td>
<td>30.6</td>
<td>1,500</td>
<td>945</td>
<td>8.06</td>
<td>22.8</td>
<td>2,083</td>
<td>1,453</td>
<td>2.28</td>
<td>4.44</td>
<td>41.6</td>
</tr>
<tr>
<td>W 24 $\times$ 162</td>
<td>468</td>
<td>12.7</td>
<td>45.2</td>
<td>1,404</td>
<td>897</td>
<td>10.8</td>
<td>32.4</td>
<td>1,950</td>
<td>1,380</td>
<td>3.05</td>
<td>5.51</td>
<td>30.6</td>
</tr>
<tr>
<td>W 24 $\times$ 146</td>
<td>418</td>
<td>12.5</td>
<td>42.0</td>
<td>1,254</td>
<td>804</td>
<td>10.6</td>
<td>30.6</td>
<td>1,742</td>
<td>1,237</td>
<td>3.01</td>
<td>5.92</td>
<td>33.2</td>
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<tr>
<td>W 33 $\times$ 118</td>
<td>415</td>
<td>9.67</td>
<td>27.8</td>
<td>1,245</td>
<td>778</td>
<td>8.20</td>
<td>21.7</td>
<td>1,729</td>
<td>1,197</td>
<td>2.32</td>
<td>7.76</td>
<td>54.5</td>
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<tr>
<td>W 30 $\times$ 124</td>
<td>408</td>
<td>9.29</td>
<td>28.2</td>
<td>1,224</td>
<td>769</td>
<td>7.88</td>
<td>21.5</td>
<td>1,700</td>
<td>1,183</td>
<td>2.23</td>
<td>5.65</td>
<td>46.2</td>
</tr>
<tr>
<td>W 21 $\times$ 147</td>
<td>373</td>
<td>12.3</td>
<td>46.4</td>
<td>1,119</td>
<td>713</td>
<td>10.4</td>
<td>32.8</td>
<td>1,554</td>
<td>1,097</td>
<td>2.95</td>
<td>5.44</td>
<td>26.1</td>
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<tr>
<td>W 24 $\times$ 131</td>
<td>370</td>
<td>12.4</td>
<td>39.3</td>
<td>1,110</td>
<td>713</td>
<td>10.5</td>
<td>29.1</td>
<td>1,542</td>
<td>1,097</td>
<td>2.97</td>
<td>6.70</td>
<td>35.6</td>
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<tr>
<td>W 18 $\times$ 158</td>
<td>356</td>
<td>11.4</td>
<td>56.5</td>
<td>1,068</td>
<td>672</td>
<td>9.69</td>
<td>38.0</td>
<td>1,483</td>
<td>1,033</td>
<td>2.74</td>
<td>3.92</td>
<td>19.8</td>
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<tr>
<td>W 30 $\times$ 108</td>
<td>346</td>
<td>8.96</td>
<td>26.3</td>
<td>1,038</td>
<td>648</td>
<td>7.60</td>
<td>20.3</td>
<td>1,442</td>
<td>997</td>
<td>2.15</td>
<td>6.89</td>
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<td>W 27 $\times$ 114</td>
<td>343</td>
<td>9.08</td>
<td>28.2</td>
<td>1,029</td>
<td>648</td>
<td>7.71</td>
<td>21.3</td>
<td>1,429</td>
<td>997</td>
<td>2.18</td>
<td>5.41</td>
<td>42.5</td>
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<tr>
<td>W 24 $\times$ 117</td>
<td>327</td>
<td>12.3</td>
<td>37.1</td>
<td>981</td>
<td>631</td>
<td>10.4</td>
<td>27.9</td>
<td>1,363</td>
<td>970</td>
<td>2.94</td>
<td>7.53</td>
<td>39.2</td>
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<tr>
<td>W 21 $\times$ 122</td>
<td>307</td>
<td>12.2</td>
<td>41.0</td>
<td>921</td>
<td>592</td>
<td>10.3</td>
<td>29.8</td>
<td>1,279</td>
<td>910</td>
<td>2.92</td>
<td>6.45</td>
<td>31.3</td>
</tr>
<tr>
<td>W 18 $\times$ 130</td>
<td>290</td>
<td>11.3</td>
<td>47.7</td>
<td>870</td>
<td>555</td>
<td>9.55</td>
<td>32.8</td>
<td>1,208</td>
<td>853</td>
<td>2.7</td>
<td>4.65</td>
<td>23.9</td>
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<tr>
<td>W 30 $\times$ 90</td>
<td>283</td>
<td>8.71</td>
<td>24.8</td>
<td>849</td>
<td>531</td>
<td>7.39</td>
<td>19.4</td>
<td>1,179</td>
<td>817</td>
<td>2.09</td>
<td>8.52</td>
<td>57.5</td>
</tr>
<tr>
<td>W 24 $\times$ 103</td>
<td>280</td>
<td>8.29</td>
<td>27.0</td>
<td>840</td>
<td>531</td>
<td>7.04</td>
<td>20.0</td>
<td>1,167</td>
<td>817</td>
<td>1.99</td>
<td>4.59</td>
<td>39.2</td>
</tr>
<tr>
<td>W 27 $\times$ 94</td>
<td>278</td>
<td>8.83</td>
<td>25.9</td>
<td>834</td>
<td>527</td>
<td>7.50</td>
<td>19.9</td>
<td>1,158</td>
<td>810</td>
<td>2.12</td>
<td>6.70</td>
<td>49.5</td>
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<tr>
<td>W 14 $\times$ 145</td>
<td>260</td>
<td>16.6</td>
<td>81.6</td>
<td>780</td>
<td>503</td>
<td>14.1</td>
<td>54.7</td>
<td>1,083</td>
<td>773</td>
<td>3.98</td>
<td>7.11</td>
<td>16.8</td>
</tr>
<tr>
<td>W 24 $\times$ 94</td>
<td>254</td>
<td>8.25</td>
<td>25.9</td>
<td>762</td>
<td>481</td>
<td>7.00</td>
<td>10.4</td>
<td>1,058</td>
<td>740</td>
<td>1.98</td>
<td>5.18</td>
<td>41.9</td>
</tr>
</tbody>
</table>
Beam Design (revisited)

4*. Include self weight for $M_{\text{max}}$
   - and repeat 3 & 4 if necessary

5. Consider lateral stability
   Unbraced roof trusses were blown down in 1999 at this project in Moscow, Idaho.
   Photo: Ken Carper
Beam Design (revisited)

6. Evaluate shear stresses - horizontal

- \( V_a \leq V_n / \Omega \) or \( V_u \leq \phi_v V_n \)

- \( W \) and rectangles
  \[
  f_{v-\text{max}} = \frac{3V}{2A} \approx \frac{V}{A_{\text{web}}}
  \]

- General
  \[
  f_{v-\text{max}} = \frac{VQ}{Ib}
  \]

\[ V_n = 0.6 \, F_{yw} A_w \]
Beam Design (revisited)

7. Provide adequate bearing area at supports

\[ P_a \leq \frac{P_n}{\Omega} \]
\[ P_u \leq \phi P_n \]
Beam Design (revisited)

8. Evaluate torsion

\( f_v \leq F_v \)

- circular cross section
  \[ f_v = \frac{T \rho}{J} \]
- rectangular
  \[ f_v = \frac{T}{c_1 ab^2} \]

<table>
<thead>
<tr>
<th>a/b</th>
<th>c_1</th>
<th>c_2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.208</td>
<td>0.1406</td>
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<tr>
<td>1.2</td>
<td>0.219</td>
<td>0.1661</td>
</tr>
<tr>
<td>1.5</td>
<td>0.231</td>
<td>0.1958</td>
</tr>
<tr>
<td>2.0</td>
<td>0.246</td>
<td>0.229</td>
</tr>
<tr>
<td>2.5</td>
<td>0.258</td>
<td>0.249</td>
</tr>
<tr>
<td>3.0</td>
<td>0.267</td>
<td>0.263</td>
</tr>
<tr>
<td>4.0</td>
<td>0.282</td>
<td>0.281</td>
</tr>
<tr>
<td>5.0</td>
<td>0.291</td>
<td>0.291</td>
</tr>
<tr>
<td>10.0</td>
<td>0.312</td>
<td>0.312</td>
</tr>
<tr>
<td>( \infty )</td>
<td>0.333</td>
<td>0.333</td>
</tr>
</tbody>
</table>
Beam Design (revisited)

9. Evaluate deflections – NO LOAD FACTORS

\[ y_{\text{max}}(x) = \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \]
Load Tables & Equivalent Load

- uniformly distributed loads
- equivalent “w” \[ M_{\text{max}} = \frac{W_{\text{equivalent}} L^2}{8} \]

load for live load deflection limit in RED, total in BLACK
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>Typical Depth (L)</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