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
    - $F_y = 36 \text{ ksi} \; \& \; F_u = 58 \text{ ksi}$
  - ASTM A572 – high strength low-alloy
    - some beams
    - $F_y = 60 \text{ ksi} \; \& \; F_u = 75 \text{ ksi}$
  - ASTM A992 – for building framing
    - most beams
    - $F_y = 50 \text{ ksi} \; \& \; F_u = 65 \text{ ksi}$
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
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

- 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

\[ f_y = 50 \text{ksi} \]
\[ \varepsilon_y = 0.001724 \]
## LRFD Load Combinations

**ASCE-7 (2010)**

- **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)*
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
Shear in Web

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

[Diagrams showing shear failure and shear buckling]
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, $Z$

  $Z = \frac{M_p}{f_y}$
SRFD - Shear

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

- \( V_u \) - 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

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

and

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

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

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

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

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

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

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

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

Table 3-10 (continued)

W Shapes

Available Moment vs. Unbraced Length
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** \((Ω, φ)\)

2. **Draw V & M, finding** \(M_{\text{max}}\)

3. **Calculate** \(Z_{\text{req'd}}\) \(\left( M_a \leq M_n / Ω \right)\)

   \(\left( M_u \leq φ_b M_n \right)\)

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>
</tr>
</thead>
<tbody>
<tr>
<td>W 33 × 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>
</tr>
<tr>
<td>W 30 × 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>
</tr>
<tr>
<td>W 24 × 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>
</tr>
<tr>
<td>W 24 × 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>
</tr>
<tr>
<td>W 33 × 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>
</tr>
<tr>
<td>W 30 × 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>
</tr>
<tr>
<td>W 21 × 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>
</tr>
<tr>
<td>W 24 × 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>
</tr>
<tr>
<td>W 18 × 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>
</tr>
<tr>
<td>W 30 × 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>
</tr>
<tr>
<td>W 27 × 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>
</tr>
<tr>
<td>W 24 × 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>
</tr>
<tr>
<td>W 21 × 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>
</tr>
<tr>
<td>W 18 × 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>
</tr>
<tr>
<td>W 30 × 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>
</tr>
<tr>
<td>W 24 × 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>
</tr>
<tr>
<td>W 27 × 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>
</tr>
<tr>
<td>W 14 × 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>
</tr>
<tr>
<td>W 24 × 94</td>
<td>254</td>
<td>8.25</td>
<td>25.9</td>
<td>762</td>
<td>481</td>
<td>7.00</td>
<td>19.4</td>
</tr>
</tbody>
</table>

Where: $Z_x$: Cross-sectional area moment of inertia, $L_p$: Plastic section modulus, $L_r$: Residual section modulus, $M_p$: Plastic bending moment, $M_r$: Residual bending moment, $r_y$: Radius of gyration, $b_{/2t}$: Width to thickness ratio, $h/t$: Height to thickness ratio, $X_1$: First resistance moment of area, $X_2$: Second moment of area.
Beam Design (revisited)

4*. Include self weight for $M_{max}$
   - 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

\[ P_a \leq P_n / \Omega \]
\[ P_u \leq \phi P_n \]
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>( \frac{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>
</tr>
<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>Element</th>
<th>Approximate Depths</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>