Steel beams: materials & beams
Steel Beam Design

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

- smelt iron ore
- add alloying elements
- heat treatments
- iron, carbon
- microstructure

A36 steel, JOM 1998

AISC
Steel Materials

- cast into billets
- hot rolled
- cold formed
- residual stress
- corrosion-resistant "weathering" steels
- stainless
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

Strain hardening
Structural Steel

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

- welding
- bolts
Steel Construction

- fire proofing
  - cementicious spray
  - encasement in gypsum
  - intumescent – expands with heat
  - sprinkler system
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

- 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**

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

- **web**

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

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

- **bearing**
  - provide adequate area
  - prevent local yield of flange and web

*Figure 9.10* Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).
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*

* For LRFD - Steel Beams
**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
- $A_{\text{tension}} = A_{\text{compression}}$

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

- $\sigma_y = 50\text{ksi}$
- $\varepsilon_y = 0.001724$
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, $Z$

$$Z = \frac{M_p}{f_y}$$

$$k = \frac{M_p}{M_y}$$
LRFD – Shear (compact shapes)

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

- \( V_u \) - 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

$$L_p = 1.76 r_y \sqrt{\frac{F_y}{E}}$$

$$\sum \gamma_i R_i = M_u \leq \phi_b M_n$$
Compact Sections

- plastic moment can form before buckling
- criteria

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

- and \( \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}} \)

TABLE A.3 Properties of W Shapes

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Lateral Torsional Buckling

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

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

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

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

<table>
<thead>
<tr>
<th>Unbraced Length (0.5-ft increments)</th>
<th>Available Moment, $M_a/\omega$ (1 kip-ft increments)</th>
<th>ASD</th>
<th>LRFD</th>
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<tbody>
<tr>
<td>2</td>
<td>56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>48</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

W Shapes: W8x24, W8x21, W10x16, W12x22, W14x30

Legend:
- ASD: Allowable Stress Design
- LRFD: Load and Resistance Factor Design
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**
# Beam Charts by $S_x$ (Appendix A)

## Table 11

<table>
<thead>
<tr>
<th>$S_x$—US (in.³)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times$ mm³)</th>
<th>$S_x$—US (in.³)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times$ mm³)</th>
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<td>448</td>
<td>W33 × 141</td>
<td>7350</td>
<td>188</td>
<td>W18 × 97</td>
<td>3080</td>
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<td>439</td>
<td>W36 × 135</td>
<td>7200</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>411</td>
<td>W27 × 146</td>
<td>6740</td>
<td>176</td>
<td>W24 × 76</td>
<td>2890</td>
</tr>
<tr>
<td>406</td>
<td>W33 × 130</td>
<td>6660</td>
<td>175</td>
<td>W16 × 100</td>
<td>2870</td>
</tr>
<tr>
<td>380</td>
<td>W30 × 132</td>
<td>6230</td>
<td>171</td>
<td>W14 × 109</td>
<td>2840</td>
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<tr>
<td>371</td>
<td>W24 × 146</td>
<td>6080</td>
<td>166</td>
<td>W21 × 83</td>
<td>2800</td>
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<tr>
<td>359</td>
<td>W33 × 118</td>
<td>5890</td>
<td>157</td>
<td>W18 × 86</td>
<td>2720</td>
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<td>355</td>
<td>W30 × 124</td>
<td>5820</td>
<td>155</td>
<td>W14 × 99</td>
<td>2570</td>
</tr>
<tr>
<td>329</td>
<td>W30 × 116</td>
<td>5400</td>
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<td>5400</td>
<td>146</td>
<td>W18 × 76</td>
<td>2390</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>143</td>
<td>W14 × 90</td>
<td>2350</td>
</tr>
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</table>
# Beam Charts by $Z_x$

## TABLE 9.1 Load Factor Resistance Design Selection for Shapes Used as Beams

<table>
<thead>
<tr>
<th>Designation</th>
<th>$Z_x$ in.°</th>
<th>$L_p$ ft</th>
<th>$L_r$ ft</th>
<th>$M_p$ kip-ft</th>
<th>$M_r$ kip-ft</th>
<th>$F_s = 36$ ksi</th>
<th>$F_s = 50$ ksi</th>
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<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>21.3</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>
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<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>
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<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>
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<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>
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<td>W 30 × 108</td>
<td>346</td>
<td>8.96</td>
<td>26.3</td>
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<td>W 21 × 122</td>
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<td>555</td>
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<th>$b_y/2t_y$</th>
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<th>$X_1$ ksi</th>
<th>$X_2 \times 10^6 (1/ksi)^2$</th>
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<tr>
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<td>10.1</td>
<td>30.1</td>
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Lecture 18  
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ARCH 331  
F2013abn
Beam Design (revisited)

4*. Include self weight for $M_{max}$
   - it’s dead load
   - 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 \((V_u \leq \phi_v V_n)\)

- rectangles and W’s

\[
f_{v-\text{max}} = \frac{3V}{2A} \approx \frac{V}{A_{\text{web}}}
\]

\[
V_n = 0.6 F_{yw} A_w
\]

- general

\[
f_{v-\text{max}} = \frac{VQ}{Ib}
\]
Beam Design (revisited)

7. Provide adequate bearing area at supports

\[( P_a \leq \frac{P_n}{\Omega} ) \]
\[( P_u \leq \phi P_n ) \]
8. Evaluate torsion

\( (f_v \leq F_v) \)

- **circular cross section**
  \[ f_v = \frac{T\rho}{J} \]

- **rectangular**
  \[ f_v = \frac{T}{c_1ab^2} \]

<table>
<thead>
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<th>(a/b)</th>
<th>(c_1)</th>
<th>(c_2)</th>
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<td>2.0</td>
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<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”**

\[ M_{max} = \frac{w_{equivalent} L^2}{8} \]

### Load Tables for Open Web Steel Joists, K-Series

<table>
<thead>
<tr>
<th>Joint Designation</th>
<th>10K1</th>
<th>12K1</th>
<th>12K3</th>
<th>12K5</th>
<th>14K1</th>
<th>14K3</th>
<th>14K5</th>
<th>14K6</th>
<th>16K2</th>
<th>16K3</th>
<th>16K4</th>
<th>16K6</th>
<th>16K8</th>
<th>16K9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approx. Wt.</td>
<td>5.0</td>
<td>5.0</td>
<td>5.7</td>
<td>7.1</td>
<td>5.2</td>
<td>6.0</td>
<td>6.7</td>
<td>7.7</td>
<td>5.5</td>
<td>6.3</td>
<td>7.0</td>
<td>7.5</td>
<td>8.1</td>
<td>8.6</td>
</tr>
<tr>
<td>Span (ft.)</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td><strong>Load Tables</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Load for live load deflection limit in RED, total in BLACK**

---

Steel Beams 43  
Lecture 18  
Architectural Structures  
ARCH 331  
F2013abn
Sloped Beams

- stairs & roofs
- projected live load
- dead load over length

- perpendicular load to beam:
  \[ w_\perp = w \cdot \cos \alpha \]

- equivalent distributed load:
  \[ w_{adj.} = \frac{w}{\cos \alpha} \]
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</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

(Meters) 0 5 10 15 20 25 30 35 40 45 50 55

Feet 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180