Steel construction: 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

[Diagram of stress-strain curve]
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
- web
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

*Figure 9.10* Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).
\[ \sum \gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z \]

- \( M_u \) - 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
- $A_{\text{tension}} = A_{\text{compression}}$

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

\[
M_p = f_{y}A_1d = f_{y} \sum_{n.a} A_i d_i
\]
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, \( k \)**

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

  = 3/2 for a rectangle

  ≈ 1.1 for an \( I \)

- **plastic modulus, \( Z \)**

  \[ Z = \frac{M_p}{f_y} \]
LRFD – Shear (compact shapes)

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

- \( V_u \) - 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 = \( t_wd \)
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

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

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

- plastic moment can form before any buckling
- criteria

\[
b \leq \frac{bf}{2tf} \leq 0.38 \sqrt{\frac{E}{Fy}}
\]

- and \( \frac{hc}{tw} \leq 3.76 \sqrt{\frac{E}{Fy}} \)
Lateral Torsional Buckling

\[ M_n = C_b \left[ \frac{\text{moment based on lateral buckling}}{\text{max|moment|, unbraced segment}} \right] \leq M_p \]

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

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

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Lecture 15

Architectural Structures
ARCH 331

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Beam Design Charts

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

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
### Beam Charts by $S_x$ (Appendix)

**Table A9 Elastic Section Modulus—U.S. and S.I. Metric.**

<table>
<thead>
<tr>
<th>Allowable Stress Design—Selected beam shapes</th>
<th>$S_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_x$—U.S. (in.$^3$)</td>
<td>Section</td>
</tr>
<tr>
<td>448</td>
<td>W33 × 141</td>
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<tr>
<td>439</td>
<td>W36 × 135</td>
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<td>411</td>
<td>W27 × 146</td>
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<td>406</td>
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<td>380</td>
<td>W30 × 132</td>
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<td>371</td>
<td>W24 × 146</td>
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<tr>
<td>359</td>
<td>W33 × 118</td>
</tr>
<tr>
<td>355</td>
<td>W30 × 124</td>
</tr>
<tr>
<td>329</td>
<td>W30 × 116</td>
</tr>
<tr>
<td>329</td>
<td>W24 × 131</td>
</tr>
<tr>
<td>329</td>
<td>W21 × 147</td>
</tr>
</tbody>
</table>
### Beam Charts by $Z_x$ (Appendix)

**Table A11  Plastic Section Modulus—Selected Beam Shapes.**

<table>
<thead>
<tr>
<th>$Z_x$ (in.(^3))</th>
<th>Section</th>
<th>$A$ (in.(^2))</th>
<th>$d$ (in.)</th>
<th>$h$ (in.)</th>
<th>$b_1$ (in.)</th>
<th>$t_{\text{f}}$ (in.)</th>
<th>$t_{\text{w}}$ (in.)</th>
<th>$\phi_{0}M_p$ (k-ft.)</th>
<th>$\phi_{0}M_s$ (k-ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>378</td>
<td>W30 x 116</td>
<td>34.2</td>
<td>30.01</td>
<td>26.75</td>
<td>10.50</td>
<td>0.850</td>
<td>0.565</td>
<td>1,420</td>
<td>987</td>
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<td>373</td>
<td>W21 x 147</td>
<td>43.2</td>
<td>22.06</td>
<td>18.25</td>
<td>12.51</td>
<td>1.150</td>
<td>0.720</td>
<td>1,400</td>
<td>987</td>
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<tr>
<td>370</td>
<td>W24 x 131</td>
<td>38.5</td>
<td>24.48</td>
<td>21.00</td>
<td>12.86</td>
<td>0.960</td>
<td>0.605</td>
<td>1,390</td>
<td>987</td>
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<td>346</td>
<td>W30 x 108</td>
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<td>29.83</td>
<td>26.75</td>
<td>10.48</td>
<td>0.760</td>
<td>0.545</td>
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<td>343</td>
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<td>33.5</td>
<td>27.29</td>
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<td>10.07</td>
<td>0.930</td>
<td>0.570</td>
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<td>897</td>
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<td>333</td>
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<td>38.8</td>
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<td>12.44</td>
<td>1.035</td>
<td>0.650</td>
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<td>327</td>
<td>W24 x 117</td>
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<td>21.00</td>
<td>12.80</td>
<td>0.850</td>
<td>0.550</td>
<td>1,230</td>
<td>873</td>
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<tr>
<td>322</td>
<td>W18 x 143</td>
<td>42.1</td>
<td>19.49</td>
<td>15.5</td>
<td>11.22</td>
<td>1.320</td>
<td>0.730</td>
<td>1,210</td>
<td>846</td>
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<td>312</td>
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<td>29.65</td>
<td>26.75</td>
<td>10.45</td>
<td>0.670</td>
<td>0.520</td>
<td>1,170</td>
<td>807</td>
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<tr>
<td>307</td>
<td>W21 x 122</td>
<td>35.9</td>
<td>21.68</td>
<td>18.25</td>
<td>12.39</td>
<td>0.960</td>
<td>0.600</td>
<td>1,150</td>
<td>819</td>
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<td>305</td>
<td>W27 x 102</td>
<td>30.0</td>
<td>27.09</td>
<td>24.00</td>
<td>10.02</td>
<td>0.830</td>
<td>0.515</td>
<td>1,140</td>
<td>801</td>
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<tr>
<td>289</td>
<td>W24 x 104</td>
<td>30.6</td>
<td>24.06</td>
<td>21.00</td>
<td>12.75</td>
<td>0.750</td>
<td>0.500</td>
<td>1,080</td>
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<tr>
<td>279</td>
<td>W21 x 111</td>
<td>32.7</td>
<td>21.51</td>
<td>18.25</td>
<td>12.34</td>
<td>0.875</td>
<td>0.550</td>
<td>1,050</td>
<td>747</td>
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<td>278</td>
<td>W27 x 94</td>
<td>27.7</td>
<td>26.92</td>
<td>24.00</td>
<td>9.90</td>
<td>0.745</td>
<td>0.490</td>
<td>1,040</td>
<td>729</td>
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<tr>
<td>261</td>
<td>W18 x 119</td>
<td>35.1</td>
<td>18.92</td>
<td>15.50</td>
<td>11.27</td>
<td>1.060</td>
<td>0.655</td>
<td>979</td>
<td>693</td>
</tr>
</tbody>
</table>

*Steel Beams 37*  
*Lecture 18*  
*Architectural Structures*  
*ARCH 331*  
*Su2016abn*
Beam Design (revisited)

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

- 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 - horizontal

- \((V_a \leq V_{nt}/\Omega)\) or \((V_u \leq \phi \, V_n)\)

- rectangles and W’s

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

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

- general

\[ f_{v-max} = \frac{VQ}{I_b} \]
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 3.1.** Coefficients for Rectangular Bars in Torsion

<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>
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<td>1.5</td>
<td>0.231</td>
<td>0.1988</td>
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<td>2.0</td>
<td>0.246</td>
<td>0.229</td>
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<tr>
<td>2.5</td>
<td>0.258</td>
<td>0.249</td>
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<tr>
<td>3.0</td>
<td>0.267</td>
<td>0.263</td>
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<tr>
<td>4.0</td>
<td>0.282</td>
<td>0.281</td>
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<td>5.0</td>
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<td>10.0</td>
<td>0.312</td>
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<tr>
<td>(\infty)</td>
<td>0.333</td>
<td>0.333</td>
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</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 Table 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>14K4</th>
<th>14K6</th>
<th>16K2</th>
<th>16K3</th>
<th>16K4</th>
<th>16K6</th>
<th>16K8</th>
<th>16K9</th>
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<tbody>
<tr>
<td>Depth (in.)</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>
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<td>16</td>
<td>16</td>
<td>16</td>
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<tr>
<td>Approx. Wt. (lbs./ft.)</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.7</td>
<td>5.7</td>
<td>5.7</td>
<td>5.2</td>
<td>6.0</td>
<td>6.7</td>
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<td>Span (ft.)</td>
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**Load for live load deflection limit in RED, total in BLACK**
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

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