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
    • $F_y = 36 \text{ ksi}$ & $F_u = 58 \text{ ksi}$
  – 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
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)

ASCE-7 (2010)
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

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

- \( M_u \) - 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*

*Note: The asterisk (*) indicates additional information or conditions that are not explicitly stated in the given equation but are typically associated with the plastic section modulus in structural design.
**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
- $A_{\text{tension}} = A_{\text{compression}}$

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

\[ \sigma_y = 50 \text{ksi} \]

$\varepsilon_y = 0.001724$
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, $k$
  
  $= 3/2$ for a rectangle

  $\approx 1.1$ for an I

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

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

- \( V_u \) - 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

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

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

- plastic moment can form before buckling
- criteria

\[- \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \leq 3.76 \frac{E}{F_y}\]
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
- \( M_{\text{max}} \) = \(|\text{max moment}|\), unbraced segment
- \( M_A \) = \(|\text{moment}|\), 1/4 point
- \( M_B \) = \(|\text{moment}|\), center point
- \( M_C \) = \(|\text{moment}|\), 3/4 point
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 $M_{max}$

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

4. Choose (economical) section from section or beam capacity charts
**Beam Charts by $S_x$ (Appendix A)**

<table>
<thead>
<tr>
<th>$S_x$—US (in.$^3$)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times \text{mm}^3$)</th>
<th>$S_x$—US (in.$^3$)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times \text{mm}^3$)</th>
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<tbody>
<tr>
<td>448</td>
<td>W33 × 141</td>
<td>7350</td>
<td>188</td>
<td>W18 × 97</td>
<td>3080</td>
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<tr>
<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></td>
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<td></td>
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<tr>
<td>406</td>
<td>W33 × 130</td>
<td>6660</td>
<td>175</td>
<td>W16 × 100</td>
<td>2870</td>
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<tr>
<td>380</td>
<td>W30 × 132</td>
<td>6230</td>
<td>173</td>
<td>W14 × 109</td>
<td>2840</td>
</tr>
<tr>
<td>371</td>
<td>W24 × 146</td>
<td>6080</td>
<td>166</td>
<td>W21 × 83</td>
<td>2800</td>
</tr>
<tr>
<td>359</td>
<td>W33 × 118</td>
<td>5890</td>
<td>157</td>
<td>W18 × 86</td>
<td>2720</td>
</tr>
<tr>
<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>
<td>154</td>
<td>W24 × 68</td>
<td>2530</td>
</tr>
<tr>
<td>329</td>
<td>W24 × 131</td>
<td>5400</td>
<td>146</td>
<td>W21 × 73</td>
<td>2480</td>
</tr>
<tr>
<td>329</td>
<td>W21 × 147</td>
<td>5400</td>
<td>143</td>
<td>W18 × 76</td>
<td>2390</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>W14 × 90</td>
<td>2350</td>
</tr>
</tbody>
</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.$^3$</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>
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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>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>
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<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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<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>
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<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>
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<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>
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<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>
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<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>
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<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>

For other shapes and sections, consult relevant structural design standards or resources.
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 \((V_u \leq \phi V_n)\)
- rectangles and W’s
  \[ f_{v\text{-max}} = \frac{3V}{2A} \approx \frac{V}{A_{web}} \]
  \[ V_n = 0.6F_{yw}A_w \]
- general
  \[ f_{v\text{-max}} = \frac{VQ}{Ib} \]
Beam Design (revisited)

7. Provide adequate bearing area at supports

\[
\begin{align*}
P_a & \leq P_n / \Omega \\
P_u & \leq \phi P_n
\end{align*}
\]
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 |
|-----------|--------|--------|
| \( \frac{a}{b} \) | \( c_1 \) | \( c_2 \) |
| 1.0 | 0.208 | 0.1406 |
| 1.2 | 0.219 | 0.1661 |
| 1.5 | 0.231 | 0.1958 |
| 2.0 | 0.246 | 0.229  |
| 2.5 | 0.258 | 0.249  |
| 3.0 | 0.267 | 0.263  |
| 4.0 | 0.282 | 0.281  |
| 5.0 | 0.291 | 0.291  |
| 10.0 | 0.312 | 0.312  |
| \( \infty \) | 0.333 | 0.333  |
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”\[
M_{max} = \frac{We_{equivalent}L^2}{8}
\]

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

<table>
<thead>
<tr>
<th>Structure</th>
<th>Span</th>
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<tbody>
<tr>
<td>Decking</td>
<td>L/30–L/50</td>
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<tr>
<td>Wide flanges</td>
<td>L/18–L/28</td>
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<tr>
<td>Plate girders</td>
<td>L/15–L/20</td>
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<tr>
<td>Open-web joists</td>
<td>L/18–L/22</td>
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<tr>
<td>Fink truss</td>
<td>L/4–L/5</td>
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<td>Howe truss</td>
<td>L/4–L/5</td>
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<td>Bowstring truss</td>
<td>L/6–L/10</td>
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<td>Special truss</td>
<td>L/4–L/15</td>
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<td>Arches</td>
<td>L/3–L/5</td>
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<td>Ribbed domes</td>
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<td>Cables</td>
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<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>