Steel Construction

- standard rolled shapes
- open web joists
- plate girders
- decking

Steel

- cast iron – wrought iron - steel
- cables
- columns
- beams
- trusses
- frames

Steel Construction

- welding
- bolts
Steel Construction

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

Steel Materials

- high strength to weight ratio
- ductile
- beam size often limited by deflection
- column size limited by slenderness

Steel Beams

- types
  - manufactured shapes

Steel Beams

- types
  - wide flange

Figure 5.22: Stress-strain diagram for mild steel (A36) with key points highlighted.
Steel Beams

- types
  - open web joists (manufactured trusses)

Steel Beams

- types (more)
  - plate girder
  - decking

Steel Beams

- lateral stability - bracing
- local buckling - stiffen
Local Buckling

- steel I beams
- flange
  - buckle in direction of smaller radius of gyration
- web
  - force
  - “crippling”

Shear in Web

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

Steel Beams

- end conditions
  - a) away from connection - full section effective
  - b) high shear – only web effective
Steel Beams

- end conditions
  - c) bolt holes – less material
  - d) local web buckling

Steel Beams

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

Steel Beams

- connections
  - welds
  - bolts

Steel Design – Open Web Joists

- SJI: [www.steeljoist.com](http://www.steeljoist.com)
- Vulcraft: [www.vulcraft.com](http://www.vulcraft.com)
  - K Series (Standard)
    - 8-30” deep, spans 8-50 ft
  - LH Series (Long span)
    - 18-48” deep, spans 25-96 ft
  - DLH (Deep Long Spans)
    - 52-72” deep, spans 89-144 ft
  - SLH (Long spans with high strength steel)
    - pitched top chord
    - 80-120” deep, spans 111-240 ft
Steel Design – Open Web Joists

<table>
<thead>
<tr>
<th>Steel Construction 22 Lecture 20</th>
<th>Architectural Structures III</th>
<th>ARCH 631</th>
</tr>
</thead>
</table>

**Steel Design – Open Web Joists**

**Standards Table: Open Web Steel Joists, K-Series**

<table>
<thead>
<tr>
<th>Load for live load deflection limit in RED</th>
<th>Total in BLACK</th>
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<tbody>
<tr>
<td>8.0</td>
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<tr>
<td>8.0</td>
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<tr>
<td>8.7</td>
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<tr>
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<td>7.5</td>
</tr>
<tr>
<td>6.3</td>
<td>6.3</td>
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<tr>
<td>8.5</td>
<td>8.5</td>
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<tr>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>10.0</td>
<td>10.0</td>
</tr>
</tbody>
</table>

**Steel Beam Design**

- **American Institute of Steel Construction**
  - **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 Beam Design**

- **AISC: 14th ed.**
  - combined ASD & LRFD in one volume in 2005

**Steel Construction 24 Lecture 21 | Applied Architectural Structures | ARCH 631 | F2012abn**
**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

---

**LRFD Steel Beam Design**

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

---

**Load Types**

- **D** = dead load
- **L** = live load
- **L_r** = live roof load
- **W** = wind load
- **S** = snow load
- **E** = earthquake load
- **R** = rainwater load or ice water load
- **T** = effect of material & temperature
<table>
<thead>
<tr>
<th>LRFD Load Combinations</th>
<th>Pure Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE-7 (2010)</td>
<td></td>
</tr>
</tbody>
</table>

- **1.4D**
- **1.2D + 1.6L + 0.5(L<sub>r</sub> or S or R)**
- **1.2D + 1.6(L<sub>r</sub> or S or R) + (L or 0.5W)**
- **1.2D + 1.0W + L + 0.5(L<sub>r</sub> 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)

**Pure Flexure**

\[ \sum \gamma R_i = M_u \leq \phi_b M_n = 0.9 F_y Z \]

- \( M_u \) - 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
- \( f_y \cdot A_1 = f_y \cdot A_2 \)
- 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

Plastic Hinge Examples

- stability can be effected

Plastic Section Modulus

- shape factor, \( k \)
  \[
k = \frac{M_p}{M_y}
\]
  \approx 3/2 for a rectangle
  \approx 1.1 for an I

- plastic modulus, \( Z \)
  \[
Z = \frac{M_p}{f_y}
\]
Shear

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

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

Flexure Design

- limit states for beam failure
  1. yielding \( L_p = 1.76 r_y \sqrt{\frac{F_y}{E}} \)
  2. lateral-torsional buckling
  3. flange local buckling
  4. web local buckling
- minimum \( M_n \) governs

\[ \Sigma \gamma_i R_i = M_u \leq \phi_b M_n \]

Compact Sections

- plastic moment can form before any buckling
- criteria
  \[ - \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \]
  \[ - \text{and} \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}} \]

<table>
<thead>
<tr>
<th>TABLE A.3</th>
<th>Properties of W Shapes</th>
</tr>
</thead>
</table>

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

Deflection Limits

- based on service condition
- no “impairment” to serviceability
- avoid ponding
- L/360 due to live load for beams & girders supporting plaster (service)

Steel Arches and Frames

- solid sections
  or open web

Steel Shell and Cable Structures
Approximate Depths

Unified Column Design

- limit states for failure

\[ P_a \leq \frac{P_n}{\Omega} \]

\[ \phi_c = 0.90 \quad P_n = F_{cr} A_g \]

\[ P_u \leq \phi_c P_n \]

1. yielding \( \frac{K L}{r} \leq 4.71 \left( \frac{E}{F_y} \right) \left( \frac{F_x}{F_{cr}} \right) \)

2. buckling \( \frac{K L}{r} > 4.71 \left( \frac{E}{F_y} \right) \left( \frac{F_y}{F_x} \right) \)

\( F_e \) – elastic buckling stress (Euler)

Procedure for Analysis

1. calculate \( KL/r \)
   - biggest of \( KL/r \) with respect to x axes and y axis

2. find \( F_{cr} \) from appropriate equation
   - tables are available

3. compute \( P_n = F_{cr} A_g \)
   - or find \( f_c = \frac{P_a}{A} \) or \( P_u/A \)

4. is \( P_a \leq \frac{P_n}{\Omega} \) or \( P_u \leq \phi_c P_n \)?
   - yes: ok
   - no: insufficient capacity and no good
**Procedure for Design**

1. guess a size (pick a section)
2. calculate KL/r
   - biggest of KL/r with respect to x axes and y axis
3. find $F_{cr}$ from appropriate equations
   - or find a table
4. compute $P_n = F_{cr}A_g$
   - or find $f_c = P_a/A$ or $P_u/A$

**Procedure for Design (cont’d)**

5. is $P_a \leq P_n/\Omega$ or $P_u \leq \phi P_n$?
   - yes: ok
   - no: pick a bigger section and go back to step 2.

6. check design efficiency
   - percentage of stress $= \frac{P_r}{P_c} \cdot 100\%$
   - if between 90-100%: good
   - if < 90%: pick a smaller section and go back to step 2.

**Beam-Column Design**

- moment magnification ($P$-$\Delta$)

$$M_u = B_1 M_{\text{max-factor}}$$

$$B_1 = \frac{C_m}{1 - (P_u/P_{e1})}$$

$C_m$ – modification factor for end conditions

- 0.6 – 0.4($M_y/M_o$) or
- 0.85 restrained, 1.00 unrestrained

$P_{e1}$ – Euler buckling strength

$$P_{e1} = \frac{\pi^2 EA}{(KL/r)^2}$$
Beam-Column Design

- LRFD (Unified) Steel
  - for $\frac{P_r}{P_c} \geq 0.2$:
  $$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$
  - for $\frac{P_r}{P_c} < 0.2$:
  $$\frac{P_u}{2\phi_c P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$

$P_r$ is required, $P_c$ is capacity

$\phi_c$ - resistance factor for compression = 0.9

$\phi_b$ - resistance factor for bending = 0.9

Construction Supervision

- proper grade material
  - high strength bolts
- quality welds
- proper bolted conditions (ex. sc)
- fabrication and erection of steel frame connection details