CH. 10 – STEEL CONSTRUCTION

PROPERTIES OF STRUCTURAL STEEL

• Most widely used
• **Ductility**: property that allows steel to withstand excessive deformation due to high tensile stresses w/o failure
  - Good for earthquake resistant structures
• Due to carefully controlled conditions, composition, size and strength can be uniformly predicted ∴ structures do not need to be over designed to compensate for manufacturing or erection variables as do concrete or timber

**Advantages:**
- High strength
- Ductility
- Uniformity of manufacture
- Variety of shapes and sizes
- Ease & speed of erection

**Disadvantages:**
- Reduction in strength when subject to fire
- Does not burn but deforms when exposed to high temps ∴ must protect with fire-resistant material
  - Sprayed on cementitious material
  - Gypsum board
  - Embedded in concrete
- Corrode in presence of moisture
  - Prevented by including elements in steel to resist (stainless)
  - Prevent by covering (paint)

Types and Composition of Steel

• Composed primarily of iron w/ small amounts of carbon & other (impurities or added)
• **Medium carbon steel**: typical for construction
  - Other elements: manganese: .5% to 1.0%
  - Silicon: .25% - .75%
  - Phosphorous
  - Sulfur
• Percentage of carbon content in steel affects strength and ductility, as carbon is added the strength increases while ductility and weldability decreases
  - **Low carbon steel**: .06% - .3% carbon
  - **Medium carbon steel**: .3% - .5%
  - **High carbon steel**: .5% - .8%
  - **Standard structural steel**: .2% - .5%
• **ASTM A572 grade 50**: Most common type of steel for structural use
  - Means was manufactured according to American Society of Testing and Materials specification #A572
  - Yield point of steel is 50ksi
• Other high strength steel A242, A440, A441 which have yield points of 46ksi or 50ksi

Shapes and Sizes of Structural Steel

• **Wide flange members (H)**: called this b/c width of flange is deeper that standard I-beams
  - Suitable for columns b/c width of flange nearly equal to depth of section so similar rigidity in both directions
• **Wide flange sections (W)**: nominal depth in inches 7 weight in lbm/ft
• **American standard I-beams (S)**: narrow flange width in relation to depth
  - Inside faces of flanges have slope of 1:6 or 16 2/3%
  - Unlike wide flanges, actual depth of I-beam in any size group is also the nominal depth
  - Beams only
  - Heavier sections are made by adding thickness to the flanges on the inside face only
• **American standard channel sections (C):** designated by ‘C’ followed by depth and weight per foot
  - Depth is constant for any size group
  - Extra weight is added by increasing thickness of web and inside face of flanges
  - **Applications:**
    - Frame openings
    - Stair stringers
    - Applications where flush side is required
    - Seldom used as beams b/c buckle due to asymmetrical shape
• **Structural tees:** made by cutting a wide flange section (WT) or an I-beam (ST)
  - A WT9x57 is cut from a W18x114
  - B/c symmetrical about one axis and open flange, often used for chords of steel trusses
• **Steel angles (L):** equal or unequal legs
  - Designated by L followed by lengths of angles and then followed by thickness of the legs
  - **Applications:**
    - Used in pairs as members for steel trusses
    - Single ‘T’ as lintels
    - Misc bracing
• **Structural tubing:** Square (TS), rectangular or round pipes
  - Used as light columns and in large trusses or space frames
  - Structural pipe available in standard weight, extra strong and double extra strong
  - Pipe is designated by nominal diameter although outside dimension is slightly larger
  - Square/rectangular tubing refers to actual outside dimensions
• **Bars** are any rectangular section 6” or less in width w/ a thickness of .203” and greater or sections 6” to 8” in width with a thickness of .230” and greater
• **Plates:** any section over 8” in width w/ thickness of .230” and greater or sections 48” in width w/ a thickness of .180” and over

**Allowable Stresses**
- Allowable unit stresses for structural steel are expressed as percentages of the minimum specified yield point
  - For A36 steel the yield point is 36ksi
  - Percentages used depends on the type of stress
  - Established by AISC and adopted by reference into model codes

**STEEL BEAMS**
- Design involves finding lightest section : least expensive that will resist bending and shear forces within allowable limits of stress and that will not have excessive deflection for the condition of use

**Lateral Support and Compact Sections**
- When subject to a load the top flange is in compression there is a tendency for beam to buckle under load similar to column under axial load
- To resist, compression flange needs to be supported or beam made larger
  - Steel beams are typically laterally supported b/c of standard construction methods
    - Steel deck welded to beam
    - Top flange embedded in concrete
    - Composite construction
  - If beam is continuously supported or supported at intervals no greater than Lc, the full allowable stress of .66Fy may be used
    - **Lc:** Max unbraced length of compression flange where allowable bending stress may be taken at .66Fy
  - If greater than Lc but not greater than Lu then allowable stress must be reduced to .60Fy
    - **Lu:** Max unbraced length of compression flange where allowable bending stress may be taken at .60Fy
- Sections are determined to be compact or non-compact based on yield strength and width-to-thickness ratio of web and flanges
  - **Non-compact:** lower allowable bending stress
Design for Bending

- Two approaches to designing steel beams
  - Flexural formula
  - AISC manual tables

- **Basic flexural formula** is

  \[ S = \frac{M}{F_b} \]

  - The formula is the same for steel except that AISC uses nomenclature \( F_b \) for the allowable bending stress instead of \( f_b \) which it reserves for computed bending stress
  - Basic formula is used in two forms:
    - To select beam by finding required **section modulus, \( S \)**
    - To calculate the **maximum resisting moment, \( M \)** when a beam is being analyzed
  - Two forms are:
    \[ S = \frac{M}{F_b} \]
    \[ M = SF_b \]

  - Keep units consistent!!!!!!!!!
  - Typical steel design uses kips instead of lbf and feet instead of inches
    - **However**: allowable stresses listed in kips per square inch so convert kip=in from kip-ft
  - Tables in AISC manual are kips while problems are often in lbf
    - If a problem is stated in lbf or inches, first step should be to convert the units

- **Example 10.1**: An A36 steel beam that is laterally supported is to span 26’ supporting a uniform load of 1500plf not including its own weight. What is the most economical wide flange section that can be used?

  **Calculation method of design using flexural formula**:
  - Weight of a beam accounted for in one of two ways
    - Assume weight and add to the load and then calculate moment or . . .
    - Ignore it, solve the problem to find actual weight of the beam and then recheck the work
  - Since weight of steel is usually small percentage of total weight, it can usually be ignored
    - **Example**: heaviest wide flange section used for a beam is a W44x335 so the most additional weight possible is .335klf
  - Finding **maximum moment for uniformly loaded beam**

    \[ M = \frac{wL^2}{8} \]

  - Converting 1500plf to 1.5klf

    \[ M = (1.5)(26)^2/8 \]
    \[ M = 127ft-kips \]

  - For A36 steel the allowable bending stress is 24ksi from Table 10.2.
  - Finding **required section modulus**

    \[ S = \frac{M}{F_b} \]
    \[ S = (127)(12/24) \]
    \[ S = 63.5in^3 \]

    - **Note**: value of 127ft-kips had to be converted to in-kips to work with the value of 24ksi and yield an answer in cubic inches for the **section modulus**
  - To find the most economical section (lightest weight) look at AISC tables showing **properties of sections Table** 10.3 or look in the **section modulus Table** 10.4 in the AISC manual
    - The lightest weight section that satisfies the required section modulus is a **W16x40** with an \( S \) of 64.7in³
- Table 10.4 also lists the maximum resisting moment that the beam can carry and the length limits, \( L_c \) and \( L_u \) for unsupported sections
  - Maximum moment the beam can resist for A36 steel is 129ft-kips
  - If 40lbf (.40kips) of beam weight is added to the load and the moment recalculated, the resulting moment is **130.1ft-kips** which is over the maximum allowable of 129 so another size should be selected

**Design method based on Tabulated values in the AISC manual:**
- Use the *allowable loads on beams tables* in the AISC manual
- Table 10.5 which makes it easy to calculate allowable loads for a given size beam, select for a given span and loading, find deflections and shear values and check for unbraced lengths
  - Tables take into account the weight of the beams but should be deducted to determine the net load that the beam will support
  - Tables are for uniformly loaded beams but can be used for concentrated loads by using the table of concentrated load equivalents in the AISC manual which five factors for converting concentrated loads to uniform loads
  - **Note:** for short spans the allowable loads for beams may be limited by shear stress in the web instead of maximum bending stress
    -Loads above the heavy line in the tables are limited by the maximum allowable web shear
    - When spacing of lateral bracing exceeds \( L_c \) but is less than \( L_u \) the tabulated loads must be reduced by a ratio of \( F_b \) for non-compact sections to \( F_b \) for compact sections or 21.6/23.8 which is .91
    - Use the tables, take the tabulated load and multiply by .91

- **Example 10.2:** An A36 beam fully laterally supported spans 15’ and carries a uniform load of 1700plf. If there is only space for a 12” deep beam, what size section should be used? If the beam is only laterally supported at its third points could the same beam be used?
  - 1700plf is 1.7klf
  - Total load on beam is \((1.7)(15') = 25.5kips\)
  - In Table 10.5 the lightest weight 12” section that can support 25.5 kips is a W12x22 w/ a uniform load value of 27kips
    - If only supported at its third points (every 5’) this distance would exceed the \( L_c \) value for a W12x22 (4.3’ from table) b/c unsupported length is more than 4.3’ but less than the \( L_u \) of the same beam (6.4’) **allowable load must be reduced by the ratio of .91**
    - The new allowable load is now \((27)(.91) = 24.57kips\) which is less than 25.5kips required so try the next larger beam that can support 35kips but more importantly its \( L_c \) is 6.9 which is more than the 5’ distance of lateral bracing : this beam is acceptable

- **Example 10.3:** A W12x45 beam of A36 steel spans 21’. What is the maximum load per foot this beam can carry?
  - In Table 10.5, for a W12x45 beam the **total allowable load** for a 21’ span is 44kips
    - Dividing by 21 the allowable load per foot is:
      
      \[
      \begin{align*}
      P &= WL \\
      W &= P/L \\
      W &= 44/21 \\
      W &= 2.1kips/t
      \end{align*}
      \]

**Design for Shear**
- In most cases shear is not a factor for steel b/c sections selected to resist required bending stress is typically more than adequate to resist shear
  - **However:** should be checked especially for short heavily loaded beams or with heavy loads near the supports which may govern the design
- B/c shearing stresses are not distributed evenly over the cross section and are **zero at the extreme fibers**, flanges are discounted in calculating resistance to shear, only the area of the web is used
- **Unit shearing stress** is

\[ F_v = \frac{V}{dt_w} \]

\[ d = \text{actual depth of beam} \]
\[ t_w = \text{thickness of web} \]

- **Example 10.4:** Check the shear in beam in Ex 10.2
  - Total load on beam is **25.5kips**
  - Maximum vertical shear is one-half of this or **12.75kips**
  - In Table 10.3 for a W12x22, actual depth is **12.31”** and web thickness is .260
    - **Actual shear stress** is:

\[ F_v = \frac{V}{dt_w} \]
\[ F_v = 12.75/(12.31)(.26) \]
\[ F_v = 3.98\text{ksi} \]

- From Table 10.2 the allowable shear on gross sections is **14.5ksi**
- The area of the web is \((12.31)(.26) = 3.2\text{in}^2\)
- **Allowable shear force** is:

\[ V = (14.5)(3.2) \]
\[ V = 46.4\text{kips} \]

  - This allowable shear can also be found at the bottom of Table 10.5 under the row labeled \(V\). In this case it is rounded to 46kips

**Designing for Deflection**

- Although a beam may be sufficient to resist bending stress, it may sag or create problems such as cracking of finished ceiling or ponding on roof
- Maximum allowable deflection is determined partly by codes and partly by design judgment
- Deflection can be calculated in two ways
  - Deflection formulas for static loads
  - Tabulated values in the AISC manual

- **Example 10.5:** find the actual deflection of the W12x22 beam used in Ex10.2

**Design method using deflection formulas for static loads**

- Using the formula for maximum deflection of a uniformly loaded beam

\[ \Delta = \frac{5wl^4}{384EI} \]

  - **Note:** units must be consistent!!!!!!!
    - Units are converted to inches
    - Since the load in Ex 10.2 was 1700plf and weight of the beam is 22plf the total load is 1722plf or **143.5lb/in** \((1722/12)\)
    - Span must also be converted to inches or **\(15’\)(12”)**
      - Table 10.3 **moment of inertia** for a W12x22 beam is **156in^4**
      - The **modulus of elasticity** for steel is **29000000psi**

\[ \Delta = \frac{5wl^4}{384EI} \]
\[ \Delta = (5)(143.5)((15)(12))^4/(384)(29,000,000)(156) \]
\[ \Delta = .434\text{in} \]

**Design method using tabulated values in the AISC manual**

- **Maximum deflection** under the maximum allowable load is shown in the last column in Table 10.5 as .46 for a 15’ span
  - **However:** remember this formula gives the deflection under the maximum loading based on the uniform load constant and not on the actual load which is usually less
- **Actual deflection** can be found by multiplying maximum deflection by ratio of the total design load to the total allowable load
  - The design load is 25.5kips
  - Dividing design load by maximum allowable load of 27kips found in table, a ratio of .944 is obtained
    - \((.944)(.46)\) (deflection from table) = 0.434in which is the same as found by using the standard deflection formula
  - If deflection was limited to 1/360 of the span, the **maximum allowable deflection** would be
    \[
    \Delta = \frac{L}{360} \\
    \Delta = \frac{(15)(12)}{360} \\
    \Delta = .5”
    \]
  - Actual deflection is less than the maximum so selected beam works in deflection
  - **Note:** beam length of 15’ had to be converted to inches

**STEEL COLUMNS**

- Amount of load a steel column can support depends on area and allowable unit stress & unbraced length of column
- Properties of a column that resist buckling are the area and the moment of inertia
  - These are mathematically combined into the **radius of gyration**, \(r\)
- Effect of a columns unbraced length and radius of gyration is combined in the slenderness ratio
  - **Slenderness ratio:** for steel columns is the ratio of a columns length in inches to the radius of gyration
    \[
    \text{Slenderness ratio} = \frac{l}{r}
    \]
- In general, the greater the slenderness ratio, the greater the tendency to fail under buckling \(\because\) less load \(\because\) most steel columns are not symmetrical about both axes (wide flange) & the least radius of gyration governs for design purposes \(\because\) about this axis the column will fail first
  - Radii of gyration, \(r\) about both axes are given in the AISC manual
- For most efficient column, the radius of gyration should be the same in each direction
  - Typical for light to moderate loads
  - **However:** not appropriate for heavy loads and where many beam connections are made
  - Wide flange sections are most often used \(\because\) radius of gyration in Y-Y axis is close to that of the X-X axis
    - Special wide flange sections manufactured to provide nearly symmetrical columns with large load-carrying capacities
    - Typically 12” to 14” nominal depth
- Allowable axial compressive stress, \(F_a\) in steel columns depends on the slenderness ratio and the allowable yield stress of steel
  - Exact values of allowable stress is calculated with several complex equations based on the **Euler equation**
    - Specific equations to be used depends on slenderness ratio of column
- Once allowable stress is determined the basic equation for axial loading can be used
  \[
  P = F_aA
  \]

**End Conditions**

- **Four states:**
  - Fixed against rotation and translation (side-to-side)
    - Column embedded in concrete
    - Moment resisting connection
  - Fixed in rotation but free in translation
  - Fixed in translation but free in rotation
  - Free to both rotate and move side-to-side
    - Top of flag pole
• How the ends are fixed affects the ability of a column to resist axial loads so AISC introduces a value K to modify the unbraced length when calculating the slenderness ratio:
  - Multiplying the (K) value by the actual unbraced length (l) gives the effective length, $Kl$
  - Entire formula for slenderness ratio then becomes

\[
\text{Slenderness ratio} = \frac{Kl}{r}
\]

• Values for the various end conditions

Fig 10.2
  - For most building conditions the value for K is taken as 1.0

• Example 10.6: a W12x120 column 13’ high is fixed at the top and bottom in both rotation and translation. What is the effective slenderness ratio?

  - In Table 10.3 the radius of gyration for a W12x120 section is 5.51 in the X-axis and 3.13 in the Y-axis
  - Diagram above gives the recommended K value for fixed top and bottom ends as .65
  - The slenderness ratio is

\[
\text{Slenderness ratio} = \frac{Kl}{r}
\]

\[
\text{Slenderness ratio} = \left(0.65\right)\left(13\right)\left(12\right)/3.13
\]

\[
\text{Slenderness ratio} = 32.4
\]

- Note: length must be converted to inches and that the least radius of gyration must be used

Design for Axial Compression

• Calculation can be done with formulas or with tables
  - Column formulas to determine allowable axial stress are complicated so AISC has tabulated allowable stress values based on the slenderness ratio and the allowable yield stress of the steel being used
  - Table 10.6 for Kl/r values of 1 to 200
    - Kl/r values over 200 are not allowed

• Example 10.7: A W12x26 column of A36 steel has an unsupported length of 18’. If it is free to rotate but fixed in translation at both ends what is the columns maximum load-carrying capacity?

Design method using calculations

  - In Table 10.3 a W12x26 section has an area of 7.65in² and a least radius of gyration of 1.51”
  - The K value for this type of column is 1.0 so the Kl/r is

\[
\text{Slenderness ratio} = \frac{Kl}{r}
\]

\[
\text{Slenderness ratio} = \left(1\right)\left(18\right)\left(12\right)/1.51
\]

\[
\text{Slenderness ratio} = 143
\]

  - In Table 10.6 the allowable axial stress, $F_a$ is 7.30ksi
  - Using equation for maximum axial load is then

\[
P = F_aA
\]

\[
P = (7.3)\left(7.65\right)
\]

\[
P = 55.85\text{kips}
\]

Design method using AISC manual tables

  - Design suing tables of allowable column loads is found in the AISC manual and gives allowable axial loads in kips for various wide flange shapes, pipe columns and square tubing for 36ksi and 50ksi steel
  - Table 10.7 the loads are arranged by effective length, Kl
  - Load values are omitted when Kl/r exceeds 200
**Example 10.8:** Select the lightest 12” wide flange shape of A36 steel to support a concentric axial load of 225 kips if the unbraced length is 12’ and the K value is 1.0

- In Table 10.7 in the rows designated as KL equal to 12, look across under the Fy columns marked 36 to find the sections with an allowable load of 225 or greater
- Some of the possible selections include:
  - W12x58: 301 kips
  - W12x53: 275 kips
  - W12x50: 236 kips
- Since W12x50 is the lightest, this is the best choice
  - **Note:** same design procedures apply for pipe and tube columns

Build Up Sections
- **Plate girders:** steel plate as well and steel plates welded to it for flanges
  - Similar to wide flange but much heavier
  - Easily fabricated deeper than the maximum 44 in the USA
  - B/c web is thin relative to depth, it must be reinforced with vertical stiffeners to prevent bucking
  - Typically done with angles welded to web

Open-Web Steel Joists
- K-series depth increases in 2” increments
- LH & DLH series depths increase in 4” increments
- Designation: depth, series designation and particular type of chord used
  - Example: 36LH13 joint is 36” deep and of the LH series with a #13 chord type
- Chord types within any size group increases with load carrying capacity
- **Advantages:**
  - Spanning medium to long distances
  - Lightweight and efficient structural members
  - Allows for ductwork

<table>
<thead>
<tr>
<th>Series</th>
<th>Name</th>
<th>Span in ft</th>
<th>Depth in inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-series:</td>
<td>standard</td>
<td>8’-60’</td>
<td>8”-30”</td>
</tr>
<tr>
<td>LH-series</td>
<td>long span</td>
<td>25’-96’</td>
<td>18”-48”</td>
</tr>
<tr>
<td>DLH-series</td>
<td>deep long span</td>
<td>89’-144’</td>
<td>52”-96”</td>
</tr>
</tbody>
</table>

- Configurations of open web joists vary with manufacturer but Steel Joist Institute has established standard load tables for design purposes
  - Give load carrying capacities in lbf per linear foot of the joists designs based on span
  - Tables have two numbers
    - **Top:** total safe uniformly distributed load carrying capacity
    - **Bottom:** live load per linear foot of joist which will produce an approximate deflection of 1/360 of the span
      - Live loads that will produce a deflection f 1/240 of the span may be obtained by multiplying the bottom number by 1.5
- **Example 10.9:** open web joists spaced 2’ oc span 30’. They support a dead load of 50psf including an allowance for their own weight and a live load of 60psf. If the maximum allowable deflection due to live load is 1/360 of the span, what is the most economical section to use?
  - **First:** convert to load per liner foot of joist
    - Since joists are 2’oc, the dead load is (50)(2) or 100plf and the live load is (60)(2) or 120plf
  - **Second:** looking at Table 10.9 across the row of 30’ spans there is an
    - 18K4 that will support 245lbf total load and 144lbf live load
    - 20K3 that will support 227lbf total load and 153lbf live load
  - Both will work, but looking at weight per foot the 18K4 weighs 7.2plf and the 20K3 weighs 6.7plf. :: **20K3 is the more economical section**