Overview of Structural Design
Structural design consists of selecting the “best” structural system to support the expected loads. This includes:

- determining the expected use of the building and therefore the type of loads,
- selecting a material and determining the arrangement of the structural members (the layout),
- determining the shape and size of each member

The first step is performed by the architect, with input from the owner. The second step is also performed by the architect, often with input from the structural engineer. For example, the structural engineer can provide the architect estimates of the maximum practical span lengths (and therefore column spacings) of a steel-frame and concrete-frame building, and the expected construction cost of each. The last step is performed by the structural engineer, often with input from vendors of pre-fabricated building components such as steel bar joists or concrete double-T beams. (See Vulcraft bar joist example.)

The “best” structural system is the one that best meets criteria that include the following:

- Cost to owner
  - Material costs—the smallest size members
  - Labor cost
  - Construction time
  - Maintenance costs
- Benefit to owner
  - Desirable column arrangement
  - Max. usable vertical space (e.g. minimum beam depth)
- Safety—members must have sufficient strength
- Servicability—members must not deflect excessively

Design Procedure for this Class
Design of steel members for this class will primarily involve selecting the lightest member of the specified shape that meets the appropriate strength and serviceability criteria. We will construct spreadsheets to quickly analyze if a selected member size meets these criteria. The spreadsheet will cycle through all possible sizes to help us select the lightest size. The heart of the spreadsheet will consist of an analysis of the member against the relevant criteria specified by AISC. Each spreadsheet will be documented with a complete set of hand-calculations including sketches and references to the AISC specification.

The first and most important step in learning to analyze a steel member for a particular type of loading is to understand the mechanics of the loaded member at failure. The second step is to apply the relevant text and equations from the AISC Specifications to calculate if the available strength meets or exceeds the required strength. Finally the use of select design aids to speed the design process should be mastered.
Tension Member Load-Deformation Behavior

A tension member behaves similarly to a tensile test specimen. As the tensile load (P) is gradually increased, the member elongates (δ) proportionately according to the formula

\[ \delta = \frac{P}{AE} \]

The load deformation relationship for a particular sample can be normalized to represent the stress-strain relationship for all samples of that material

\[ \frac{P}{\delta} = \frac{\sigma}{\varepsilon} = E \]

A typical stress-strain curve for steel is shown in Figure 1 below. The limit of proportional load to deformation behavior occurs at the yield stress, \( F_y \). As additional load is applied, the stress increases due to strain hardening of the steel up to the ultimate tensile strength, \( F_u \). The yield stress, strength, and modulus of elasticity (E) are specified in the Manual for typical steels in Tables 2-3 and 2-4 (see Table 1 below). The preferred grade of steel for various shapes is also indicated.

![Stress vs. Strain Diagram]

**Figure 1.** Tensile stress vs. tensile strain

| Table 1. Material properties for various steel grades (from Tables 2-3 and 2-4). |
|---------------------------------|---|---|---|
|                                | Grade 36 | A572 | A992 |
| \( F_y \), ksi                | 36        | 50   | 50   |
| \( F_u \), ksi                | 58        | 65   | 65   |
| \( E \), ksi                  | 29,000    | 29,000 | 29,000 |
| Shapes                         | C, L, plates, bars | HP | W |
Tension Member Failure Modes

The tensile load is uniform along the length of a member. Since the net cross-section is smallest at the bolted connection, the stress is highest in this location. As the tensile load on a member is increased, the steel adjacent to the bolt holes yields first. Since the bolt holes represent a small segment of the overall length of the member, the elongation due to the yielding adjacent to the bolt holes is negligible.

As load continues to increase, one of the following occurs:

- the gross section yields, possibly leading to excessive deformation,
- the cross section through the bolt holes ruptures, resulting in a loss of integrity of the structure, or
- a cross section through the bolt holes fails in a combination of shear and tension called, block shear

![Yield Failure](image1)

![Rupture Failure](image2)

![Block Shear Failure](image3)

Figure 2. Failure modes of tension members

Tension Member Design Criteria

The specifications for tension member design for this class are provided in Section D1 through D3 (pp 26-29) and J4.1 (pg 112) of the AISC Specification for Structural Steel Buildings (hereafter called the Specification).

The equations for the tension member strength criteria are shown in Table 2 below, and the equations for available tensile strength ($P_n$) are shown for each failure mode in Table 3. Definitions of all of the symbols in this section are provided on pg 7 of this handout. The yield stress ($F_y$) and tensile strength ($F_u$) depend on the type and grade of steel specified by
the designer (see Tables 2-4 and 2-5). The gross cross-sectional area of the member is listed in Part I Dimensions and Properties of the Manual. Equations for calculating the effective area are shown in the next sections.

**Table 2.** Tension member strength criteria with example load combination

<table>
<thead>
<tr>
<th></th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi P_n \geq P_r$</td>
<td>$\frac{P_n}{\Omega} \geq P_r$</td>
<td></td>
</tr>
<tr>
<td>$P_r = 1.2 P_D + 1.6 P_L$</td>
<td>$P_r = P_D + P_L$</td>
<td></td>
</tr>
</tbody>
</table>

**Table 3.** Available strength ($P_n$) for each tension member failure mode

<table>
<thead>
<tr>
<th>Available Strength, $P_n$</th>
<th>$\Omega$</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross section yields</td>
<td>$F_y A_g$</td>
<td>1.67</td>
</tr>
<tr>
<td>Net section ruptures</td>
<td>$F_u A_e$</td>
<td>2.00</td>
</tr>
<tr>
<td>Block Shear</td>
<td>$0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$</td>
<td>2.00</td>
</tr>
</tbody>
</table>

The serviceability criterion for tension members is specified in Section D1 of the Specification:

“... the slenderness ratio $L / r$ preferably should not exceed 300.”

**Net Area, $A_n$**

The net area is calculated by subtracting the area of the bolt holes from the gross area and adding in a factor to account for staggered holes, if present.

$$A_n = A_g - t \cdot n_{holes} (\phi_{hole} + 1/8”) + t \cdot n_{gage \ spaces} \cdot s^2/4g$$

**Figure 3.** Example of gage length ($g$) and pitch ($s$) for plate with two gage spaces
Effective Net Area, $A_e$

The effective net area accounts for uneven distribution of the tensile force in the member near the connection. For example, if only one leg of an angle is bolted, the unbolted leg has less stress adjacent to the connection, resulting in higher stress in the bolted leg. The stress in the unbolted leg is said to “lag” the stress in the bolted leg, due to shear deformation of the member.

The effective net area ($A_e$) is calculated by multiplying the net area by a shear lag factor, $U$.

$$A_e = A_n \times U$$

Shear Lag Factor, $U$

Equations and values for the shear lag factor ($U$) are listed in full in Table D3.1 on pg. 29 of the Specification, and in summary form in Table 3 below. Members with cross-section elements in different planes (angles for example) can calculate $U$ two different ways and take the largest value. For these types of members, the shear lag factor decreases with

- increasing eccentricity of the connection ($\bar{x}$), and
- decreasing length of connection ($l$).

The connection eccentricity is the perpendicular distance between the member tensile force (located at the centroid of the member cross-section) and the center of resistance (located at the interface between the two connected members). Connection length ($l$) and width ($w$) for welded connections with longitudinal welds only are illustrated in Figure 4; and connection eccentricity ($\bar{x}$) and length ($l$) for bolted connections are illustrated in Figure 5.

**Table 3. Shear Lag Factor, $U$**

<table>
<thead>
<tr>
<th>Type of Tension Member</th>
<th>Condition</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plates with fasteners or longitudinal and transverse welds</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>Plates with longitudinal welds only</td>
<td>$l \geq 2w$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>$2w &gt; l \geq 1.5w$</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td>$1.5w &gt; l \geq w$</td>
<td>0.75</td>
</tr>
<tr>
<td>Single angles</td>
<td>max of: $l - \bar{x} = \frac{l}{l}$</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>with 4 or more fasteners in direction of loading</td>
<td>0.60</td>
</tr>
</tbody>
</table>
Block Shear
Block shear failure (described in Section J4.3, pg 16.1-112 of the Specification) occurs when a piece of the tension member “tears out”, as indicated in Figure 6 below. In block shear failure, the surface perpendicular to the direction of load fails due to tensile rupture, and the surface parallel to the direction of load fails in shear. Shear failure may be either due to yielding of the gross section or rupture of the net section (with holes subtracted), whichever occurs at a smaller load. Shear yield and strength are 0.6 of the corresponding tensile values.

The nominal axial strength \( P_n \) for block shear failure is calculated by summing the tensile rupture strength and the larger of the shear strengths

\[
P_n = U_{bs} F_u A_{nt} + \max[0.6 F_y A_{gv}, 0.6 F_u A_{nv}]
\]

\( U_{bs} \) is a reduction coefficient to account for a non-uniform tensile stress distribution. Figure C-J4.2 on pg 16.1-352 of the Commentary shows examples of connections for \( U_{bs} = 1.0 \) and an example of \( U_{bs} = 0.5 \). \( U_{bs} \) should equal 1.0 for plate and angle tension members.
Symbols

- $A_e =$ effective net area, in$^2$
- $A_g =$ gross area of member, in$^2$
- $A_{gv} =$ gross area subject to shear, in$^2$
- $A_{nt} =$ net area subject to tension, in$^2$
- $A_{nv} =$ net area subject to shear, in$^2$
- $F_u =$ specified minimum tensile strength of the type of steel being used, ksi
- $F_y =$ specified minimum yield stress of the type of steel being used, ksi
- $g =$ gage = tranverse center-to-center spacing between fastener gage lines, in
- $l =$ length of connection, in
- $P_D =$ tensile force due to dead loads
- $P_L =$ tensile force due to live loads
- $P_n =$ nominal tensile strength
- $P_r =$ required tensile strength, LRFD or ASD
- $s =$ pitch = longitudinal center-to-center spacing of any two consecutive holes, in
- $t =$ thickness of tension member, in
- $U =$ shear lag factor (see Table D3.1, pg 29 in Spec.)
- $U_{bs} =$ 1 for uniform tensile stress, = 0.5 of non-uniform tensile stress
- $w =$ plate width, in
- $\bar{x} =$ connection eccentricity, in
- $\phi_t =$ resistance factor for tension
- $\Omega_t =$ safety factor for tension