2.5 PROBABILISTIC BASIS OF LOAD AND RESISTANCE FACTORS
Dead: 34%, 14.9%, 2%, 0.1%

Line:

\[ 2^{k/ft} \times 1.2(2^{k/ft}) = 2.4^{k/ft} \]

1.4 (2^{k/ft})

5^{k/ft} \times 1.6 (5^{k/ft}) = 8^{k/ft}
Service Loads or Working Loads or Unfactored loads

1.2D? 1.6W? 1.6L?

1150
DL+WL+LL

Possibility of failure: 0.000001%

Cost: $87 Million
Probability of failure - Negligible
Cost $8.7 Million
Load Factors  Resistance Factors

\[ R_u = \sum x_i q_i \]
2.6 STEEL CONSTRUCTION MANUAL

Part 1. Dimensions and Properties. This part contains details on standard hot-rolled shapes, pipe, and hollow structural sections, including all necessary cross-sectional dimensions and properties such as area and moment of inertia.

Part 2. General Design Considerations. This part includes a brief overview of various specifications (including a detailed discussion of the AISC Specification), codes and standards, some fundamental design and fabrication principles, and a discussion of the proper selection of materials. Part 2 also lists the load combinations we discussed in Sections 2.3 and 2.4.

Part 3. Design of Flexural Members. This part contains a discussion of Specification requirements and design aids for beams, including composite beams (in which a steel shape acts in combination with a reinforced concrete floor or roof slab) and plate girders. Composite beams are covered in Chapter 9 of this textbook, "Composite Construction," and plate girders are covered in Chapter 10, "Plate Girders."

Part 4. Design of Compression Members. Part 4 includes a discussion of the Specification requirements for compression members and numerous design aids.

Part 16. Specifications and Codes. This part contains the AISC Specification and Commentary, a specification for high-strength bolts (RCSC, 2009), and the AISC Code of Standard Practice (AISC, 2010b).

Part 17. Miscellaneous Data and Mathematical Information. This part includes properties of standard steel shapes in SI units, conversion factors and other information on SI units, weights and other properties of building materials, mathematical formulas, and properties of geometric shapes.

All design aids in the Manual give values for both allowable strength design (ASD) and load and resistance factor design (LRFD). The Manual uses a color-coding scheme for these values; ASD allowable strength values ($R_n/\Omega$) are shown as black numbers on a green background, and LRFD design strength values ($\phi R_n$) are shown as blue numbers on a white background.

The AISC Specification is only a small part of the Manual. Many of the terms and
See Lecture #3 on Syllabus Flipped Class

Use of the AISC Steel Construction Manual
3.1 INTRODUCTION

bly the double-angle section, shown in Figure 3.1, along with other typical cross sections. Because the use of this section is so widespread, tables of properties of various combinations of angles are included in the AISC Steel Construction Manual. \[1 - 12\]

The stress in an axially loaded tension member is given by

\[ f = \frac{P}{A} \]

where \( P \) is the magnitude of the load and \( A \) is the cross-sectional area (the area normal to the load). The stress as given by this equation is exact, provided that the cross
3.2 TENSILE STRENGTH

A tension member can fail by reaching one of two limit states: excessive deformation or fracture. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough that the stress on the gross section is less...
\[ S = \frac{PL}{AE} = \frac{fL}{E} \quad f = \frac{P}{A} \]

\[ F_y = 50 \text{ ksi} \]
\[ F_u = 120 \text{ ksi} \]
\[ @ P = 300 \text{ ksi} \]
\[ f_{\text{net}} = \frac{300}{10} = 30 \text{ ksi} \]
\[ f_{\text{gross}} = \frac{300}{20} = 15 \text{ ksi} \]

\[ @ P = 500 \text{ ksi} \]
\[ f_{\text{net}} = \frac{500}{10} = 50 \text{ ksi} = F_y \]
\[ f_{\text{gross}} = \frac{500}{20} = 25 \text{ ksi} \]

\[ @ P = 800 \text{ ksi} \]
\[ f_{\text{net}} = \frac{800}{10} = 80 \text{ ksi} \]
\[ f_{\text{gross}} = \frac{800}{20} = 40 \text{ ksi} \]

\[ @ P = 1000 \text{ ksi} \]
\[ f_{\text{net}} = \frac{1000}{10} = 100 \text{ ksi} \]
\[ f_{\text{gross}} = \frac{1000}{20} = 50 \text{ ksi} = F_y \]

BAD NEWS: Excessive deformation over a long length.

Fails in Gross Section Yield, not in Net Section Fracture.

\[ \phi_{\text{yielding}} = 0.9 \]
\[ \phi_{\text{fracture}} = 0.75 \]
Thus, the load $P$ must be less than $FA$, or

$$ P < FA $$

The **nominal** strength in yielding is

$$ P_n = F_y A_g $$

and the nominal strength in fracture is

$$ P_n = F_u A_e = F_u A_{net} U $$

where $A_e$ is the **effective** net area, which may be equal to either the net area or, in some cases, a smaller area. We discuss effective net area in Section 3.2. (pg. 50)

**LRFD:** In load and resistance factor design, the factored tensile load is compared to the design strength. The design strength is the resistance factor times the nominal strength. Equation 2.6, (pg. 24)

$$ R_u = \phi R_n $$

can be written for tension members as

$$ P_u \leq \phi P_n $$

where $P_n$ is the governing combination of factored loads. The resistance factor $\phi_i$ is smaller for fracture than for yielding, reflecting the more serious nature of fracture.

For yielding, $\phi_i = 0.90$

For fracture, $\phi_i = 0.75$

Because there are two limit states, both of the following conditions must be satisfied:

$$ P_u \leq 0.90 F_y A_g \quad (GSY) \quad \text{Gross section yield} $$

$$ P_u \leq 0.75 F_u A_e \quad (NSR) \quad \text{Net section rupture} $$

The smaller of these is the design strength of the member.
\( Ru = \text{Factored load (ultimate request)} \)
\[ = Pu, Mu, Vu \]
\[ = 1.4D \]
\[ 1.2D + 1.6L + \ldots \]
\[ \text{etc.} \]

\( R_n = \text{Nominal strength} \)
\[ = \text{CVEN 305} \]
\[ = \text{Average of 100 tests} \]
\[ = F_y A_g \]
\[ = F_u A_{\text{effective}} \]
\[ = F_u A_{\text{net}} U \]

\( \phi = \text{an appropriate resistance factor} \)
\[ = 0.9 \text{ for tension yield (excessive deformation)} \]
\[ = 0.75 \text{ for tension rupture} \]

\( \phi R_n = \text{design strength} \)

\[ Ru \leq \phi R_n \]
\[ \frac{3}{4} \text{ Bolt T} + \frac{1}{16}'' + \frac{1}{16}'' = \]

\[ \frac{6P}{q} / \text{A with 3 holes} \]

\[ \frac{8(P)}{q} / \text{A with 2 holes} \]
in the least available strength. As an example, the controlling limit-state for bending of a
simple beam may be yielding, local buckling, or lateral-torsional buckling for strength, or
deflection or vibration for serviceability. The tabulated values may either reflect a single
limit-state or a combination of several limit-states. This will be clearly stated in the intro-
duction to the particular tables.

**Loads, Load Factors, and Load Combinations**
Based on Specification Sections B3.3 and B3.4, the required strength (either \( P_e, M_e, V_e \) etc.
for LRFD or \( P_a, M_a, V_a \) etc. for ASD) is determined for the appropriate load magnitudes,
load factors, and load combinations given in the applicable building code. These are usually
based on ASCE-7, which may be used when there is no applicable building code. The com-
mon loads found in building structures are:

- **D** = dead load
- **L** = live load due to occupancy
- **L_r** = roof live load
- **S** = snow load
- **R** = nominal load due to initial rainwater or ice exclusive of the ponding contribution
- **W** = wind load
- **E** = earthquake load

**Load and Resistance Factor Design**
For LRFD, the required strength is determined from the following factor combinations \(^1\),
which are based on ASCE-7 Section 2.3:

\[
\begin{align*}
1.4D & \\
1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) & \\
1.2D + 1.6L + 0.8(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) & \\
1.2D + 1.6L + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) & \\
1.2D \pm 1.0E + 0.5L + 0.2S & \\
0.9D \pm (1.6W \text{ or } 1.0E) & 
\end{align*}
\]
This Specification is based on strength limit states that apply to structural steel design in general. The Specification permits design for strength using either load and resistance factor design (LRFD) or allowable strength design (ASD). It should be noted that the terms strength and stress reflect whether the appropriate section approached. The load factors reflect uncertainty in individual load magnitudes and in the analysis that transforms load to load effect. The nominal loads in ASCE/SEI 7 are substantially in excess of the arbitrary point-in-time values. The nominal live, wind and snow loads historically have been associated with mean return periods of approximately 50 years. Wind loads historically have been adjusted upward by a high load factor in previous editions to approximate a longer return period; in the 2010 edition of ASCE/SEI 7 the load factor is 1.0 and the wind-speed maps correspond to return periods deemed appropriate for the design of each occupancy type (approximately 700 years for common occupancies).

The return period associated with earthquake loads has been more complex histori-
Gross Section Yield:

\[ P_n = F_y A_g \]

Net Section Fracture:

\[ P_n = F_u A_e \]

\[ = F_u A_{NET} U \]

Shear Lag Factor = 1.0 for bolted plates
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<th>Definition</th>
<th>Section</th>
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<tbody>
<tr>
<td>Δ₁</td>
<td>Deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, ( r_i ), in. (mm)</td>
<td>J2.4</td>
</tr>
<tr>
<td>( \lambda_{nf} )</td>
<td>Limiting slenderness parameter for noncompact range</td>
<td>F3.4</td>
</tr>
<tr>
<td>( \lambda_{nvw} )</td>
<td>Limiting slenderness parameter for noncompact web</td>
<td>F4.2</td>
</tr>
<tr>
<td>( \mu )</td>
<td>Mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests</td>
<td>J3.8</td>
</tr>
<tr>
<td>( \phi )</td>
<td>Resistance factor, specified in Chapters B through K</td>
<td>B3.3</td>
</tr>
<tr>
<td>( \phi_B )</td>
<td>Resistance factor for bearing on concrete</td>
<td>I6.3a</td>
</tr>
<tr>
<td>( \phi_b )</td>
<td>Resistance factor for flexure</td>
<td>F1</td>
</tr>
<tr>
<td>( \phi_c )</td>
<td>Resistance factor for compression</td>
<td>B3.7</td>
</tr>
<tr>
<td>( \phi_c )</td>
<td>Resistance factor for axially loaded composite columns</td>
<td>I2.1b</td>
</tr>
<tr>
<td>( \phi_{sf} )</td>
<td>Resistance factor for shear on the failure path</td>
<td>D5.1</td>
</tr>
<tr>
<td>( \phi_T )</td>
<td>Resistance factor for torsion</td>
<td>H3.1</td>
</tr>
<tr>
<td>( \phi_t )</td>
<td>Resistance factor for steel headed stud anchor in tension</td>
<td>I8.3b</td>
</tr>
<tr>
<td>( \phi_v )</td>
<td>Resistance factor for shear</td>
<td>G1</td>
</tr>
<tr>
<td>( \phi_v )</td>
<td>Resistance factor for steel headed stud anchor in shear</td>
<td>I8.3a</td>
</tr>
<tr>
<td>( \phi )</td>
<td>Safety factor, specified in Chapters B through K</td>
<td>R3.4</td>
</tr>
<tr>
<td>( \delta_t )</td>
<td>Safety factor for tension</td>
<td>D2</td>
</tr>
</tbody>
</table>
CHAPTER D
DESIGN OF MEMBERS FOR TENSION

This chapter applies to members subject to axial tension caused by static forces acting through the centroidal axis.

The chapter is organized as follows:

D1. Slenderness Limitations
D2. Tensile Strength
D3. Effective Net Area
D4. Built-Up Members
D5. Pin-Connected Members
D6. Eyebars

D2. TENSILE STRENGTH

The design tensile strength, \( \phi_t P_n \), and the allowable tensile strength, \( P_n/\Omega_t \), of tension members shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

(a) For tensile yielding in the gross section:

\[
P_n = F_y A_g
\]

\( \phi_t = 0.90 \) (LRFD) \quad \Omega_t = 1.67 \) (ASD)

(b) For tensile rupture in the net section:

\[
P_n = F_y A_g
\]

\( \phi_t = 0.75 \) (LRFD) \quad \Omega_t = 2.00 \) (ASD)

*Specification for Structural Steel Buildings*, June 22, 2010
*American Institute of Steel Construction*
The gross area, $A_g$, and net area, $A_n$, of tension members shall be determined in accordance with the provisions of Section B4.3.

The effective net area of tension members shall be determined as follows:

$$A_e = A_n U$$

where $U$, the shear lag factor, is determined as shown in Table D3.1.

For open cross sections such as W, M, S, C or HP shapes, WTts, STs, and single and double angles, the shear lag factor, $U$, need not be less than the ratio of the gross area of the connected element(s) to the member gross area. This provision does not apply to closed sections, such as HSS sections, nor to plates.

**User Note:** For bolted splice plates $A_e = A_n \leq 0.85A_g$, according to Section J4.1.
CHAPTER D
DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

For single angles, the radius of gyration about the z-axis produces the maximum $L/r$ and, except for very unusual support conditions, the maximum $KL/r$.

D2. TENSILE STRENGTH

Because of strain hardening, a ductile steel bar loaded in axial tension can resist without rupture a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

D3. EFFECTIVE NET AREA

Section D3 deals with the effect of shear lag, applicable to both welded and bolted tension members. Shear lag is a concept used to account for uneven stress distribu-