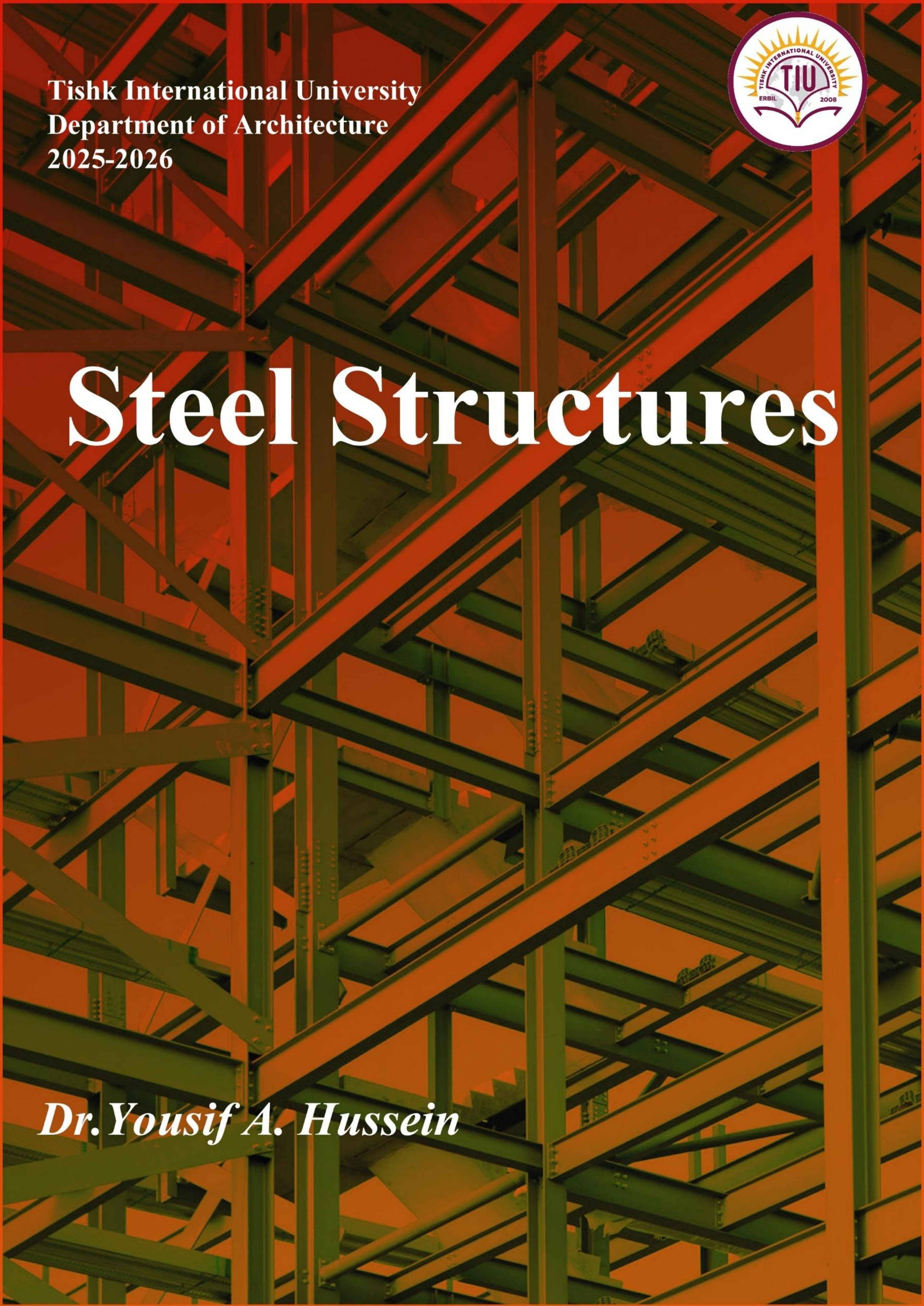

Tishk International University
Department of Architecture
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Steel Structures

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Course layout

Chapter 01 Introduction to Structural Steel Design

Chapter 02 Specifications, Loads, and Methods of Design

Chapter 03 Analysis of Tension Members

Chapter 04 Design of Tension Members

Chapter 05 Introduction to Axially Loaded Compression Members

Chapter 06 Design of Axially Loaded Compression Members

Chapter 07 Design of Axially Loaded Compression Members (Continued)

Chapter 08 Introduction to Beams

References:

Jack C . McCormac and Stephenf. Csernak (2012) “Structural Steel Design”, Prentice Hall.

American Institute of Steel Construction (2005) “Steel onstruction Manual”.

Alan Williams (2011) “Steel Structures Design: ASD/LRFD”, McGraw Hill.

CHAPTER 1

Introduction to Structural Steel Design

INTRODUCTION

- There are endless number of steel bridges, buildings, towers, and other structures in the world.
- In the United States, the first steel framed building was the Rand McNally Building in Chicago, erected in 1890.
- The Royal Insurance Building in Liverpool designed by James Francis Doyle in 1895 (erected 1896-1903) was the first to use a steel frame in the United Kingdom.
- Eiffel tower (985 ft) was constructed in 1889

ADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

1. High Strength	The high strength of steel per unit of weight means that the weight of structures will be small. This fact is of great importance for long-span bridges, tall buildings, and structures situated on poor foundations.
2. Uniformity	The properties of steel do not change appreciably with time, as do those of a reinforced- concrete structure.
3. Elasticity	Steel behaves closer to design assumptions than most materials because it follows Hooke's law up to fairly high stresses. The moments of inertia of a steel structure can be accurately calculated, while the values obtained for a reinforced-concrete structure are rather indefinite.

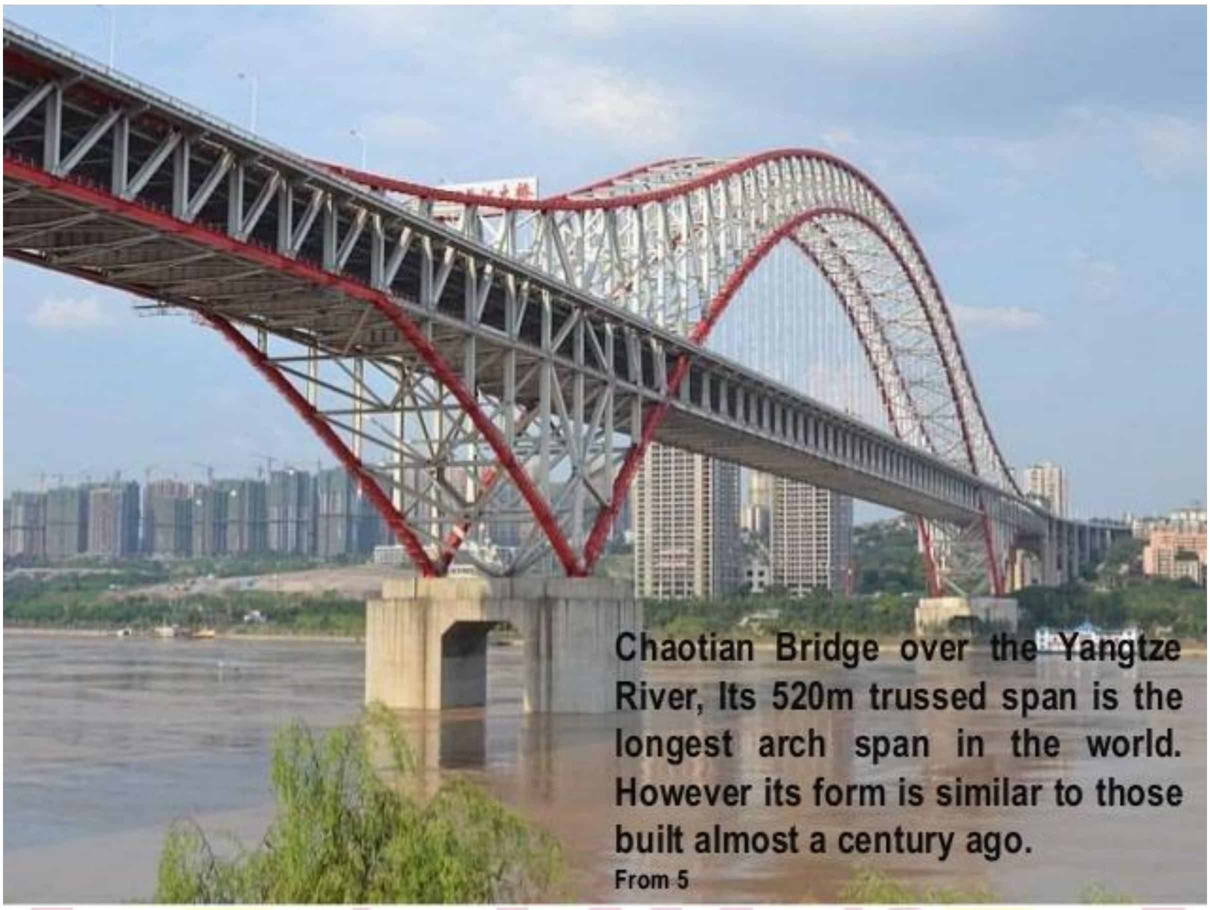


Figure Long span bridge

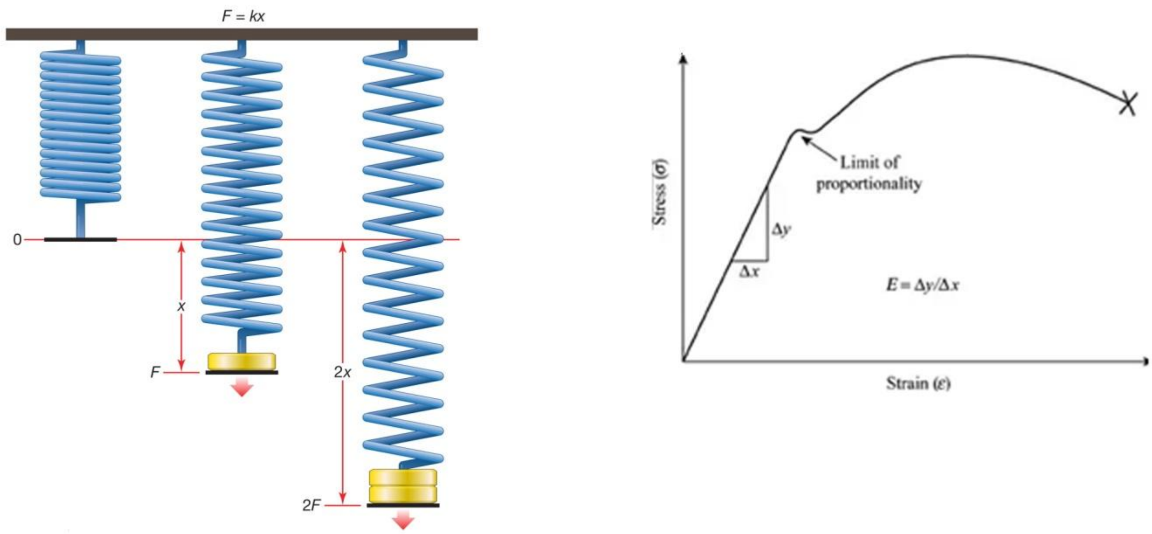
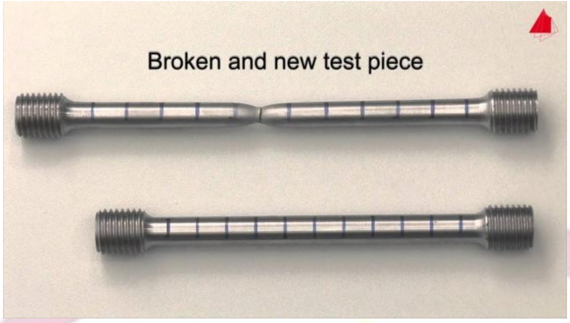


Figure definition of Hooke’s law

4. Permanence	Steel frames that are properly maintained will last indefinitely.
5. Ductility	The property of a material by which it can withstand extensive deformation without failure under high tensile stresses is its ductility.  • When a mild or low-carbon structural steel member is being tested in tension, a considerable reduction in cross section and a large amount of elongation will occur at the point of failure before the actual fracture occurs. • A material that does not have this property is generally unacceptable and is probably hard and brittle, and it might break if subjected to a sudden shock. • In structural members under normal loads, high stress concentrations develop at various points. The ductile nature of the usual structural steels enables them to yield locally at those points, thus preventing premature failures. • A further advantage of ductile structures is that when overloaded, their large deflections give visible evidence of impending failure (sometimes jokingly referred to as "running time").
6. Toughness	• Structural steels are tough—that is, they have both strength and ductility. • A steel member loaded until it has large deformations will still be able to withstand large forces. • The ability of a material to absorb energy in large amounts is called toughness.
7. Additions to Existing Structures	Steel structures are quite well suited to having additions made to them. New bays or even entire new wings can be added to existing steel frame buildings, and steel bridges may often be widened.
8. Miscellaneous	a) Ability to be fastened together by several simple connection devices, including welds and bolts.

b) Adaptation to prefabrication.

c) Speed of erection;

d) Ability to be rolled into a wide variety of sizes and shapes.

e) Possible reuse after a structure is disassembled; and

f) Scrap value, even though not reusable in its existing form. Steel is the ultimate recyclable material.

DISADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

1. Corrosion

Most steels are susceptible to corrosion when freely exposed to air and water, and therefore must be painted periodically.

2. Fireproofing Costs

The strength of structural members is tremendously reduced at temperatures commonly reached in fires when the other materials in a building burn. As a result, the steel frame of a building may have to be protected by materials with certain insulating characteristics.

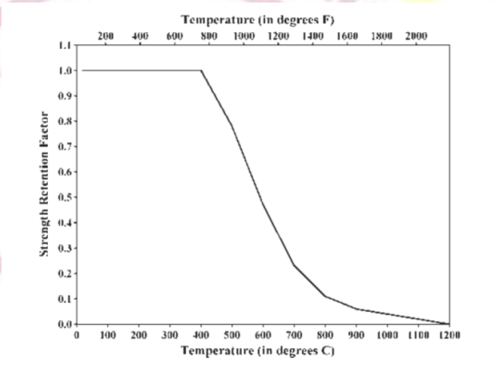
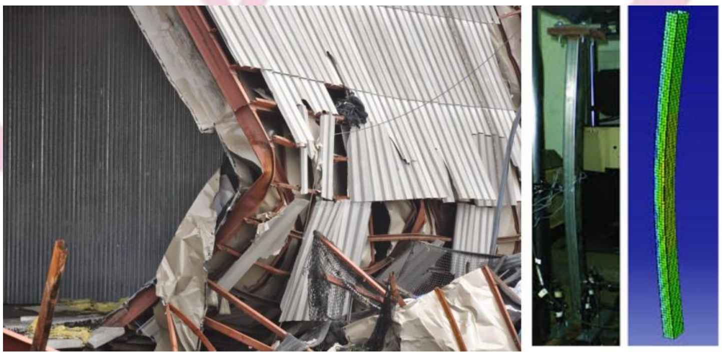


Fig. 2.1. Yield Strength Retention Factors for Structural Steel at Elevated Temperatures

3. Buckling

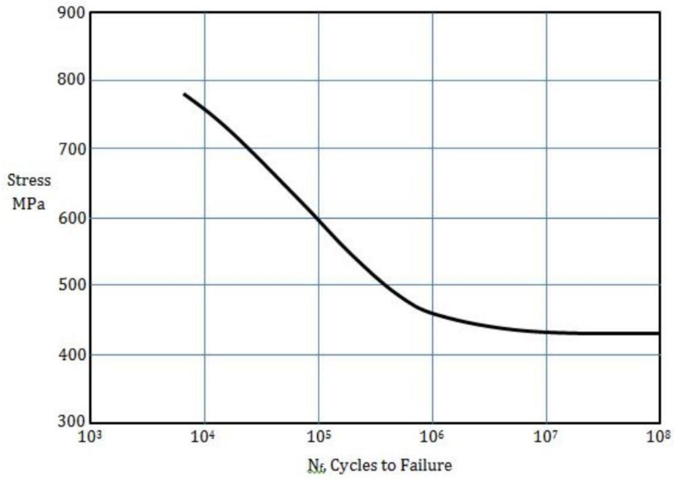
- As the length and slenderness of a compression member is increased, its danger of buckling increases.



- The use of steel columns is very economical because of their high strength-to-weight ratios. Occasionally, however, some additional steel is needed to stiffen them so they will not buckle. This tends to reduce their economy

4. Fatigue

Another undesirable property of steel is that its strength may be reduced if it is subjected to a large number of stress reversals or even to a large number of variations of tensile stress. (Fatigue problems occur only when tension is involved.)



5. Brittle Fracture

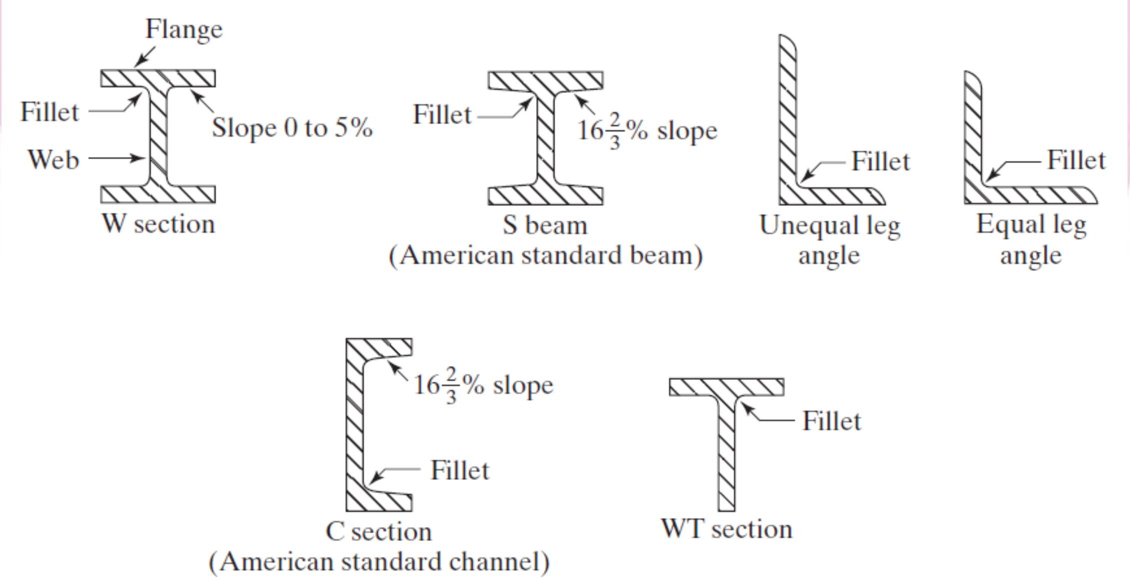
Under certain conditions steel may lose its ductility, and brittle fracture may occur at places of stress concentration. Fatigue-type loadings and very low temperatures aggravate the situation. Triaxial stress conditions can also lead to brittle fracture.

STEEL SECTIONS

- Structural steel can be economically rolled into a wide variety of shapes and sizes without appreciably changing its physical properties.
- The most desirable members are those with large moments of inertia in proportion to their areas, such as the I, T, and C shapes.
- Steel sections are usually designated by the shapes of their cross sections.
- It is necessary to make a definite distinction between American standard beams (called S beams) and wide-flange beams (called W beams), as they are both I-shaped.

S beams	W beams
The S beams, which were the first beam sections rolled in America, have a slope on their inside flange surfaces of 1 to 6.	The inner surface of the flange of a W section is either parallel to the outer surface or nearly so, with a maximum slope of 1 to 20 on the inner surface, depending on the manufacturer.

The W and S sections are shown in the figure (below), together with several other familiar steel sections. The uses of these various shapes will be discussed in detail in the chapters to follow.

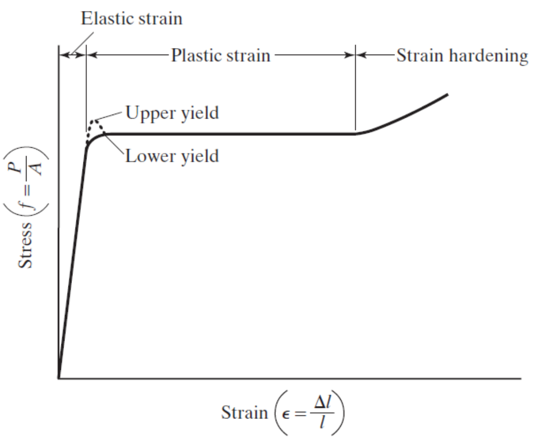


Structural shapes are identified by a certain system described in the Manual for use in drawings, specifications, and designs. Examples of this identification system are as follows:

1	W27 ×114	is a W section approximately 27 inch deep, weighing 114 lb/ft.
2	S12 × 35	is an S section 12 inch deep, weighing 35 lb/ft.
3	HP12 × 74	is a bearing pile section approximately 12 inch deep, weighing 74 lb/ft. Bearing piles are made with the regular W rolls, but with thicker webs to provide better resistance to the impact of pile driving. The width and depth of these sections are approximately equal, and the flanges and webs have equal or almost equal thickness.
4	M8 × 6.5	is a miscellaneous section 8 inch deep, weighing 6.5 lb/ft. It is one of a group of doubly symmetrical H-shaped members that cannot by dimensions be classified as a W, S, or HP section, as the slope of their inner flanges is other than 16⅔ percent.
5	C10 × 30	is a channel 10 inch deep, weighing 30 lb/ft.
6	MC18 × 58	is a miscellaneous channel 18 inch deep, weighing 58 lb/ft, which cannot be classified as a C shape because of its dimensions.
7	HSS14 × 10 × 5/8	is a rectangular hollow structural section 14 inch deep, 10 inch wide, with a 5/8-inch wall thickness. It weighs 93.10 lb/ft. Square and round HSS sections are also available.
8	L6 × 6 × 1/2	is an equal leg angle, each leg being 6 inch long and 1/2 inch thick.
9	WT18 × 151	is a tee obtained by splitting a W36 X 302 This type of section is known as a structural tee.
10	___	Rectangular steel sections are classified as wide plates or narrow bars.

STRESS–STRAIN RELATIONSHIPS IN STRUCTURAL STEEL

- To understand the behavior of steel structures, an engineer must be familiar with the properties of steel.
- Stress–strain diagrams present valuable information necessary to understand how steel will behave in a given situation.
- If a piece of ductile structural steel is subjected to a tensile force, it will begin to elongate. If the tensile force is increased at a constant rate, the amount of elongation will increase linearly within certain limits. In other words, elongation will double when the stress goes from 6000 to 12,000 psi (pounds per square inch).



- When the tensile stress reaches a value roughly equal to three-fourths of the ultimate strength of the steel, the elongation will begin to increase at a greater rate without a corresponding increase in the stress.
- The largest stress for which Hooke's law applies, or the highest point on the linear portion of the stress–strain diagram, is called the proportional limit. The largest stress that a material can withstand without being permanently deformed is called the elastic limit.
- The stress at which there is a significant increase in the elongation, or strain, without a corresponding increase in stress is said to be the **yield stress**.
- The strain that occurs before the yield stress is referred to as the **elastic strain**; the strain that occurs after the yield stress, with no increase in stress, is referred to as the **plastic strain**.
- Plastic strains are usually from 10 to 15 times as large as the elastic strains.
- Following the plastic strain, there is a range in which additional stress is necessary to produce additional strain. This is called **strain-hardening**.

C H A P T E R 2

Specifications, Loads, and Methods of Design

SPECIFICATIONS AND BUILDING CODES

- The design of most structures is controlled by building codes and design specifications.
- Engineering specifications that are developed by various organizations present the best opinion of those organizations as to what represents good practice.
- Engineering specifications and codes are actually laws or ordinances specify minimum design loads, design stresses, construction types, material quality, and other factors.
- Several organizations publish recommended practices for regional or national use. Among these organizations are the AISC and AASHTO (American Association of State Highway and Transportation Officials). Nearly all municipal and state building codes have adopted the AISC Specification, and nearly all state highway and transportation departments have adopted the AASHTO Specifications.
- Another very important code, the International Building Code (IBC).

LOADS

- Perhaps the most important and most difficult task faced by the structural engineer is the accurate estimation of the loads that may be applied to a structure during its life.
- After loads are estimated, the next problem is to determine the worst possible combinations of these loads that might occur at one time. For instance, would a highway bridge completely covered with ice and snow be simultaneously subjected to fast-moving lines of heavly loaded trailer trucks in every lane and to a 90-mile lateral wind, or is some lesser combination of these loads more likely?
- AISC Specification states the nominal loads to be used for structural design. Also, the American Society of Civil Engineers (ASCE) provides a publication entitled Minimum Design Loads for Buildings and Other Structures.

In general, loads are classified as ***dead loads, live loads, and environmental loads*** according to their character and duration of application.
Each of these types of loads are discussed in the next few sections.

DEAD LOADS

Dead loads are loads of constant magnitude that remain in one position. They consist of the structural frame's own weight and other loads that are permanently attached to the frame. For a steel-frame building, the frame, walls, floors, roof, plumbing, and fixtures are dead loads.

The approximate weights of some common building materials for roofs, walls, floors, and so on are presented in Table 2.1.

TABLE 2.1 Typical Dead Loads for Some Common Building Materials

Reinforced concrete	150 lb/cu ft
Structural steel	490 lb/cu ft
Plain concrete	145 lb/cu ft
Movable steel partitions	4 psf
Plaster on concrete	5 psf
Suspended ceilings	2 psf
5-Ply felt and gravel	6 psf
Hardwood flooring (7/8 in)	4 psf
2 × 12 × 16 in double wood floors	7 psf
Wood studs with 1/2 in gypsum each side	8 psf
Clay brick wythes (4 in)	39 psf

LIVE LOADS

Live loads are loads that may change in position and magnitude. They are caused when a structure is occupied, used, and maintained.

Live loads include:

1. Floor loads:

- The minimum gravity live loads to be used for building floors are clearly specified by the applicable building code.
- A few of the typical values for floor loadings are listed in Table 2.2, and some typical concentrated loads are listed in Table 2.3.

TABLE 2.2 Typical Minimum Uniform Live Loads for Design of Buildings	
Type of building	LL (psf)
Apartment houses	
Apartments	40
Public rooms	100
Dining rooms and restaurants	100
Garages (passenger cars only)	40
Gymnasiums, main floors, and balconies	100
Office buildings	
Lobbies	100
Offices	50
Schools	
Classrooms	40
Corridors, first floor	100
Corridors above first floor	80
Storage warehouses	
Light	125
Heavy	250
Stores (retail)	
First floor	100
Other floors	75

TABLE 2.3 Typical Concentrated Live Loads for Buildings

Hospitals—operating rooms, private rooms, and wards	1000 lb
Manufacturing building (light)	2000 lb
Manufacturing building (heavy)	3000 lb
Office floors	2000 lb
Retail stores (first floors)	1000 lb
Retail stores (upper floors)	1000 lb
School classrooms	1000 lb
School corridors	1000 lb

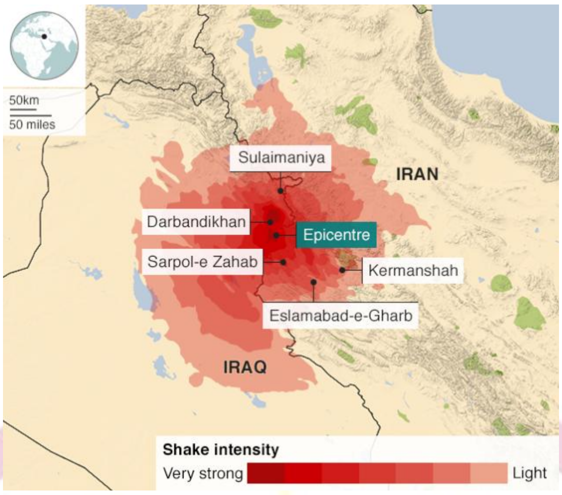
2. **Traffic loads for bridges:** Bridges are subjected to series of concentrated loads of varying magnitude caused by groups of truck or train wheels.
3. **Impact loads:** Impact loads are caused by the vibration of moving or movable loads.
4. **Longitudinal loads:** Longitudinal loads are another type of load that needs to be considered in designing some structures. Stopping a train on a railroad bridge or a truck on a highway bridge causes longitudinal forces to be applied.
5. **Other live loads:** such as soil pressures, hydrostatic pressures, and blast loads.

ENVIRONMENTAL LOADS

Environmental loads are caused by the environment in which a particular structure is located. For buildings, environmental loads are caused by rain, snow, wind, temperature change, and earthquakes.

1. **Snow:** For roof designs, snow loads varying from 10 to 40 psf are commonly used. A load of approximately 10 psf might be used for 45° (degree) slopes and a 40-psf load for flat roofs.

2. **Rain**: If water on a flat roof accumulates faster than it runs off, the result is called ponding, because the increased load causes the roof to deflect into a dish shape that can hold more water, which causes greater deflections, and so on. This process continues until equilibrium is reached or until collapse occurs. The best method of preventing ponding is to have an appreciable slope of the roof (1/4 in/ft or more), together with good drainage facilities.
3. **Wind loads:** A survey of engineering literature for the past 150 years reveals many references to structural failures caused by wind. Wind forces act as pressures on vertical windward surfaces, pressures or suction on sloping windward surfaces (depending on the slope), and suction on flat surfaces.
   For some common structures, uplift loads may be as large as 20 to 30 psf or even more.
4. **Earthquake loads:** Many areas of the world fall in "earthquake territory," and in those areas it is necessary to consider seismic forces in design for all types of structures.



LOAD AND RESISTANCE FACTOR DESIGN (LRFD)
AND ALLOWABLE STRENGTH DESIGN (ASD)

The AISC Specification provides two acceptable methods for designing structural steel members and their connections. These are Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD).

COMPUTATION OF LOADS FOR LRFD AND ASD

- With both the LRFD and the ASD procedures, expected values of the individual loads (dead, live, wind, snow, etc.) are first estimated in exactly the same manner as required by the applicable specification. These loads are referred to as **service** or **working loads**.
- Various combinations of these loads that feasibly may occur at the same time are grouped together. The largest load group (in ASD) or the largest linear combination of loads in a group (in LRFD) is then used for analysis and design.

COMPUTING COMBINED LOADS WITH LRFD EXPRESSIONS

- Load factors are calculated to increase the magnitudes of service loads to use with the LRFD procedure.
- The purpose of these factors is to account for the uncertainties involved in estimating the magnitudes of dead and live loads.
- The AISC Manual provides the following load factors for buildings.

1 $U = 1.4D$

2 $U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

3 $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L^* \text{ or } 0.5w)$

4 $U = 1.2D + 1.0W + L^* + 0.5(L_r \text{ or } S \text{ or } R)$

5 $U = 1.2D + 1.0E + L^* + 0.2S$

6 $U = 0.9D + 1.0W$

7 $U = 0.9D + 1.0E$

*The load factor on L in combinations (3.), (4.), and (5.) is to be taken as 1.0 for floors in places of public assembly, for live loads in excess of 100 psf and for parking garage live load. The load factor is permitted to equal 0.5 for other live loads.

In these load combinations, the following abbreviations are used:

U = the design or factored load

D = Dead load

L = Live load

L_r = roof live load

S = Snow load

R = nominal load due to initial rainwater or ice, exclusive of the ponding contribution.

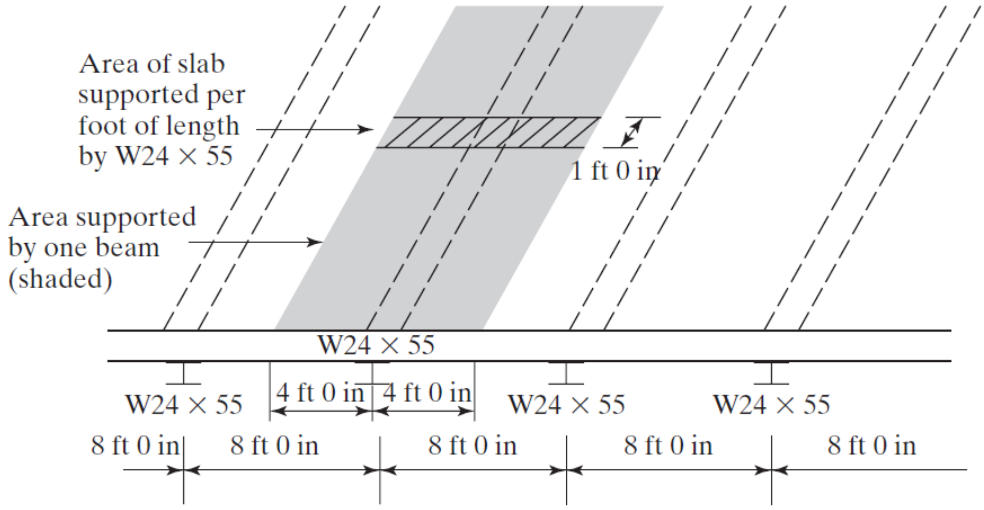
W = Wind load

E = Earthquake load

Examples 2-1 to 2-3 show the calculation of the factored loads, using the applicable LRFD load combinations. The largest value obtained is referred to as the critical or governing load combination and is to be used in design.

Example 2-1

The interior floor system shown in Figure (below) has W24 X 55 sections spaced 8 ft on center and is supporting a floor dead load of 50 psf and a live floor load of 80 psf. Determine the governing load in lb/ft that each beam must support.



Solution:

Note that each foot of the beam must support itself (a dead load) plus 8 X 1 = 8 ft^2 of the building floor

$$D = 55\frac{lb}{ft} + (8\,ft)(50\,psf) = 455\frac{lb}{ft}$$

$$L = (8\,ft)(80\,psf) = 640\,lb/ft$$

Computing factored loads, using the LRFD load combinations.

Note that with a floor live load of 80 psf a load factor of 0.5 has been added to load combinations (3.), (4.), and (5.) per the exception stated in ASCE 7-10 and this text for floor live loads.

1 $W_u = (1.4)(455) = 637\,lb/ft$

2 $W_u = (1.2)(455) + 1.6(640) = 1570\,lb/ft$

3 $W_u = (1.2)(455) + (0.5)(640) = 866\,lb/ft$

4 $W_u = (1.2)(455) + (0.5)(640) = 866\,lb/ft$

5 $W_u = (1.2)(455) + (0.5)(640) = 866\,lb/ft$

6 $W_u = (0.9)(455) = 409.5\,lb/ft$

7 $W_u = (0.9)(455) = 409.5\,lb/ft$

Governing factored load = 1570 lb/ft to be used for design.

Example 2-2

A roof system with W16 x 40 section spaced 9 ft on center is to be used to support a dead load of 40 psf; a roof live, snow, or rain load of 30 psf; and a wind load of $\pm$ 32 psf. Compute the governing factored load per linear foot.

Solution

$D = 40\ lb/ft\ +\ (9\ ft)\ (40\ psf) =\ 400\ lb/ft$

$L = 0$

$L_r\ or\ S\ or\ R = (9\ ft)(30\ psf) = 270\ lb/ft$

$W\ = (9\ ft)(32\ psf) = 288\ lb/ft$

Substituting into the load combination expressions and noting that the wind can be downward , - or uplift, + in Equation 6, we derive the following loads:

1 $W_u = (1.4)(400) = 560\ lb/ft$

2 $W_u = (1.2)(400) + 0.5(270) = 615\ lb/ft$

3 $W_u = (1.2)(400) + (1.6)(270) + (0.5)(288) = 1056\ lb/ft$

4 $W_u = (1.2)(400) + (1.0)(288) + (0.5)(270) = 903\ lb/ft$

5 $W_u = (1.2)(400) + (0.2)(270) = 534\ lb/ft$

6 $W_u = (0.9)(400) + (1.0)(288) = 648\ lb/ft$

7 $W_u = (0.9)(400) + (1.0)(-288) = 72\ lb/ft$

Governing factored load = 1056 lb/ft for design

Example 2-3

The various axial loads for a building column have been computed according to the applicable building code, with the following results: dead load 200 k, load from roof= 50 k (roof live load); live load from floors (reduced as applicable for large area and multistory columns)=250 k; compression wind = 128 k; tensile wind = 104 k; compression earthquake =60 k; and tensile earthquake =70 k.

Determine the critical design column load, Pu, using LRFD load combinations.

Solution.

This problem solution assumes the column floor live load meets the exception for the use of the load factor of 0.5 in load combinations (3.), (4.), and (5.)

1 $P_u = (1.4)(200) = 280\ k$

2 $P_u = (1.4)(200) + (1.6)(250) + (0.5)(50) = 655\ k$

3.(a) $P_u = (1.2)(200) + (1.6)(50) + (0.5)(250) = 445\ k$

(b) $P_u = (1.2)(200) + (1.6)(50) + (0.5)(128) = 384\ k$

4.(a) $P_u = (1.2)(200) + (1.0)(128) + (0.5)(250) = 518\ k$

(b) $P_u = (1.2)(200) - (1.0)(104) + (0.5)(250) = 286$

5.(a) $P_u = (1.2)(200) + (1.0)(60) + (0.5)(250) = 425\ k$

(b) $P_u = (1.2)(200) - (1.0)(70) + (0.5)(250) = 295\ k$

6. (a) $P_u = (0.9)(200) + (1.0)(128) = 308\ k$

(b) $P_u = (0.9)(200) - (1.0)(104) = 76\ k$

7. (a) $P_u = (0.9)(200) + (1.0)(60) = 240\ k$

(b) $P_u = (0.9)(200) - (1.0)(70) = 110\ k$

PROBLEMS FOR SOLUTION

For Probs. 2-1 through 2-4 determine the maximum combined loads using the recommended AISC expressions for LRFD.

2-1 D = 100 psf, L = 70 psf, R = 12 psf, L_r = 20 psf and S = 30 psf (Ans. 247 psf)

2-2 D = 10,000 lb,W = ±32,000 lb

2-3 D = 9000 lb, L = 5000 lb, L_r = 2500 lb, E = ±6500 lb (Ans. 20,050 lb)

2-4 D = 25 psf, L_r = 20 psf and W = ±26 psf

CHAPTER 3

Analysis of Tension Members

INTRODUCTION

- Tension members are found in bridge and roof trusses, towers, and bracing systems, and in situations where they are used as tie rods.

- The selection of a section to be used as a tension member is one of the simplest problems encountered in design. As there is no danger of the member buckling, the designer needs to determine only the load to be supported. Then the area required to support that load is calculated, and finally a steel section is selected that provides the required area.
- One of the simplest forms of tension members is the circular rod, but there is some difficulty in connecting it to many structures.
- When rods are used in wind bracing, it is a good practice to produce initial tension in them, as this will tighten up the structure and reduce rattling and swaying.

  A common rule of thumb is to detail the rods about 1/16 in short for each 20 ft of length. Approximate stress is:

$$f = \epsilon E = \frac{\left(\frac{1}{16}\ in\right)}{\left(12\frac{in}{ft}\right)(20\ ft)}(29 \times 10^6\ psi) = 7550\ psi$$

Another very satisfactory method involves tightening the rods with some sort of sleeve nut or turnbuckle.

- Today, although the use of cables is increasing for suspended-roof structures, tension members usually consist of single angles, double angles, tees, channels, W sections, or sections built up from plates or rolled shapes.

A few of the various types of tension members in general use are illustrated in Fig. 3.1. In this figure, the dotted lines represent the intermittent tie plates or bars used to connect the shapes.

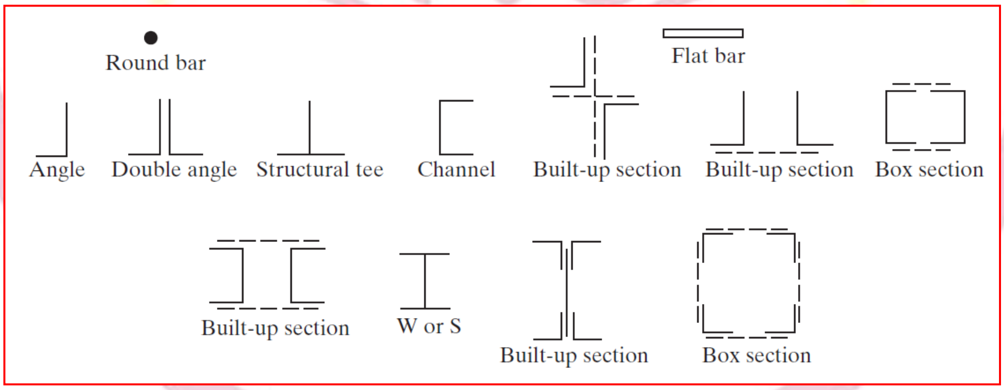


FIGURE 3.1 Types of tension members.

- The tension members of steel roof trusses may consist of single angles as small as 2 x 1/2 or 2 x 1/4 for minor members.
- For bridges and large roof trusses, tension members may consist of channels, W or S shapes, or even sections built up from some combination of angles, channels, and plates.

- Cross bracing is often done with tensile members as these members need only to act in tension.



- Steel cables are made with special steel alloy wire ropes that are cold-drawn to the desired diameter. The resulting wires with strengths of about 200,000 to 250,000 psi can be economically used for suspension bridges, cable supported roofs, ski lifts, and other similar applications.

NOMINAL STRENGTHS OF TENSION MEMBERS

A ductile steel member without holes and subject to a tensile load can resist without fracture a load larger than its gross cross-sectional area times its yield stress $[A.\sigma_{yield}]$ because of strain hardening.However, a tension member loaded until strain hardening is reached will lengthen a great deal before fracture—a fact that will, in all probability, end its usefulness and may even cause failure of the structural system of which the member is a part.

If, on the other hand, we have a tension member with bolt holes, it can possibly fail by fracture at the net section through the holes.

This failure load may very well be smaller than the load required to yield the gross section, apart from the holes.

NET AREAS

The presence of a hole increases the unit stress in a tension member, even if the hole is occupied by a bolt.

There is still less area of steel to which the load can be distributed, and there will be some concentration of stress along the edges of the hole.

Tension is assumed to be uniformly distributed over the net section of a tension member, although photoelastic studies show there is a decided increase in stress intensity around the edges of holes, sometimes equaling several times what the stresses would be if the holes were not present.

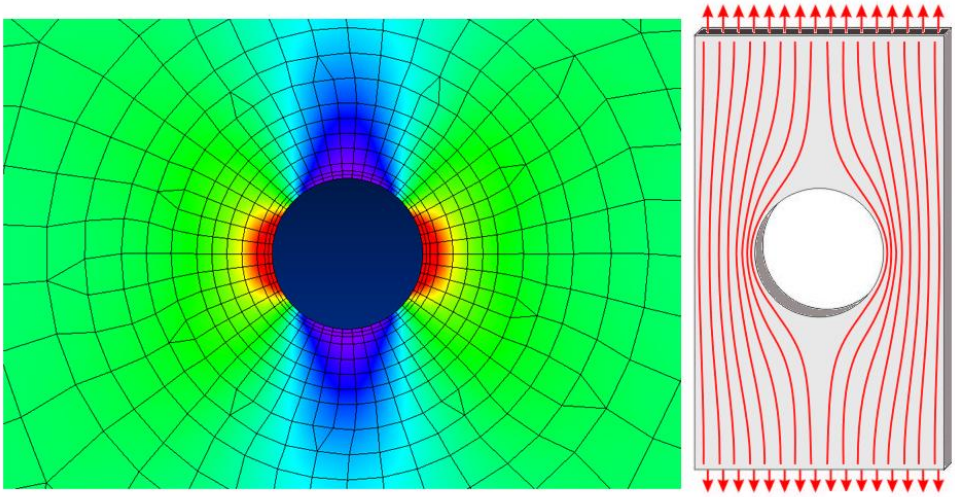


Figure. Stresses around hole (red color represents concentration of stresses)

- The term "net cross-sectional area," or simply, "net area," refers to the gross cross-sectional area of a member, minus any holes, notches, or other indentations.
- It is usually necessary to subtract an area a little larger than the actual hole. For instance, holes are ***punched*** to a diameter 1/16 in larger than that of the bolts. When this practice was followed, the punching of a hole was assumed to damage or even destroy 1/16 in more of the surrounding metal. As a result, the diameter of the hole subtracted was 1/8 in larger than the diameter of the bolt. The area of the hole was rectangular and equalled the diameter of the bolt plus 1/8 in times the thickness of the metal.
- Today, ***drills*** enable fabricators to drill very large numbers of holes. For such holes, only 1/16 in. is added to the bolt diameters for such holes.

Example 3-1

Determine the net area of 3/8 x 8-in the plate shown in Fig. 3.2. The plate is connected at its end with two lines of 3/4-in bolts.

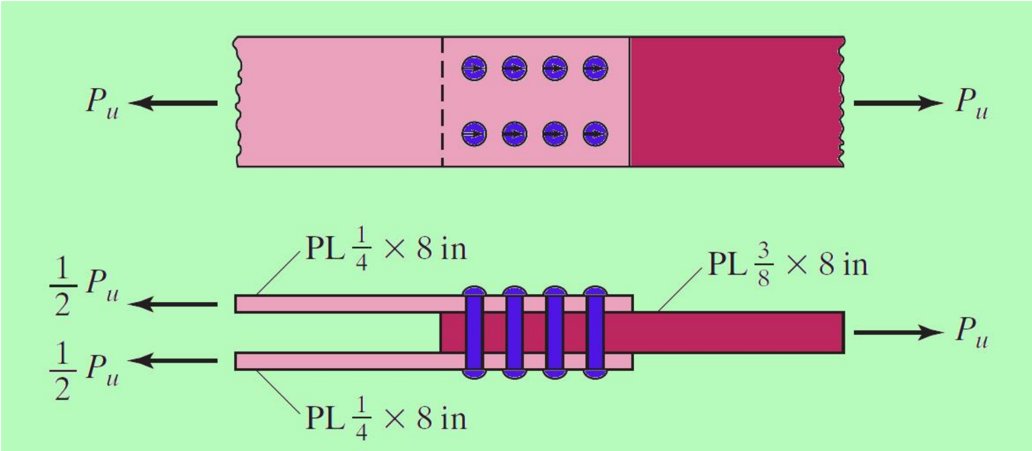


Fig. 3-2 Illustration of Example 3-1

$$A_n = \left(\frac{3}{8}\text{in}\right)(8\ in) - 2\left(\frac{3}{4}\text{in} + \frac{1}{8}\text{in}\right)\left(\frac{3}{8}\text{in}\right) = 2.34\ \text{in}^2(1510\ \text{mm}^2)$$

- The connections of tension members should be arranged so that no eccentricity is present.
- Should the connections have eccentricities, moments will be produced that will cause additional stresses in the vicinity of the connection.
- The centroidal axes of truss members meeting at a joint are assumed to coincide. Should they not coincide, eccentricity is present and secondary stresses are the result.

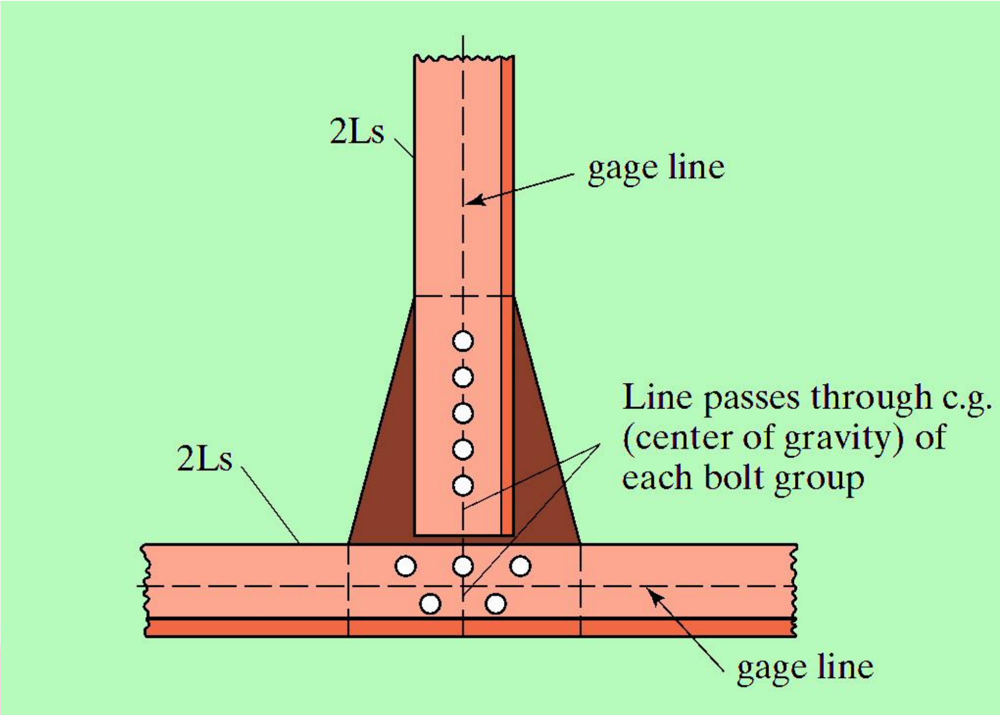


FIGURE 3.3 Lining up centroidal axes of members.

EFFECT OF STAGGERED HOLES

- Tensile members could fail transversely along line AB in either Fig. 3.4(a) or 3.4(b).

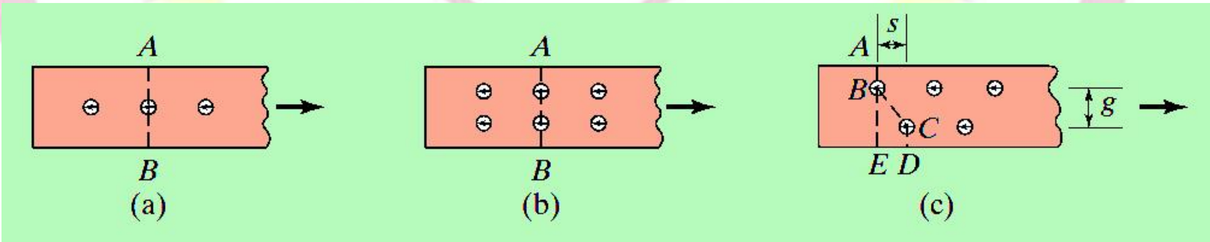


FIGURE 3.4 Possible failure sections in plates.

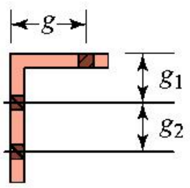
- Figure 3.4(c) shows a member in which a failure other than a transverse one is possible. The holes are staggered, and failure along section ABCD is possible unless the holes are a large distance apart.

The strength of the member along section ABCD can be determined accoridng to the AISC Specification, which offeres offer a very simple method for computing the net width of a tension member along a zigzag section. The method is to take the gross width of the member, regardless of the line along which failure might occur, subtract the diameter of the holes along the zigzag section being considered, and add for each inclined line the quantity given by the expression $s^2/4g$. where: s is the longitudinal spacing (or pitch) of any two holes and g is the transverse spacing (or gage) of the same holes. The values of s and g are shown in Fig. 3.4(c).

There may be several paths, any one of which may be critical at a particular joint. Each possibility should be considered, and the one giving the least value should be used.The smallest net width obtained is multiplied by the plate thickness to give the net area.

Holes for bolts and rivets are normally drilled or punched in steel angles at certain standard locations.These locations or gages are dependent on the angle-leg widths and on the number of lines of holes. Table 3.1 shows these gages.

TABLE 3.1 Workable Gages for Angles, in Inches



Leg	8	7	6	5	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{1}{4}$	1
g	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$
g_1	3	$2\frac{1}{2}$	$2\frac{1}{4}$	2										
g_2	3	3	$2\frac{1}{2}$	$1\frac{3}{4}$										

Example 3-2

Determine the critical net area of the 1/2-in-thick plate shown in Fig. 3.5, using the AISC Specification. The holes are punched for 3/4-in bolts.

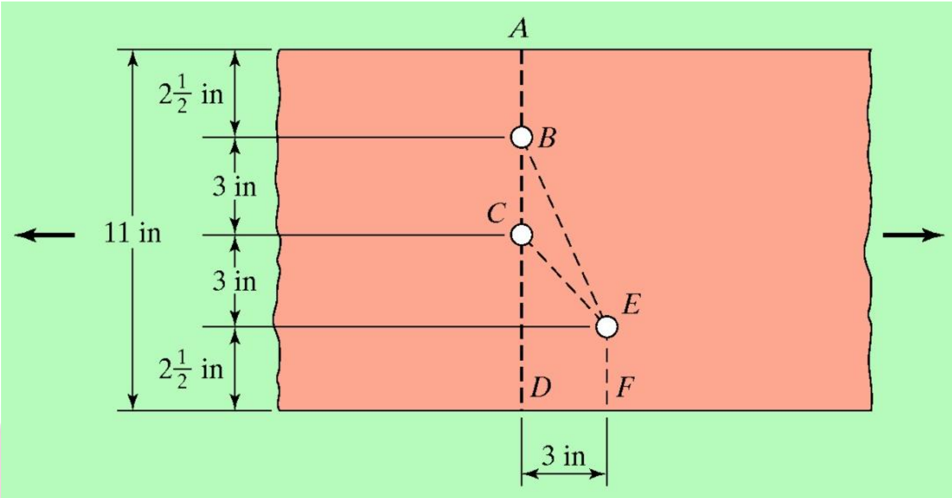


FIGURE 3.5

Solution:

- The critical section could possibly be ABCD, ABCEF, or ABEF.
- Hole diameters to be subtracted are 3/4 + 1/8=7/8 in.
- The net areas for each case are as follows:

$$ABCD = (11\ in)\left(\frac{1}{2}\ in\right) - 2\left(\frac{7}{8}\ in\right)\left(\frac{1}{2}\ in\right) = 4.63\ in^2$$

$$ABCEF = (11\ in)\left(\frac{1}{2}\ in\right) - 3\left(\frac{7}{8}\ in\right)\left(\frac{1}{2}\ in\right) + \frac{(3\ in)^2}{4(3\ in)}\left(\frac{1}{2}\ in\right) = 4.56\ in^2 \leftarrow$$

$$ABEF = (11\ in)\left(\frac{1}{2}\ in\right) - 2\left(\frac{7}{8}\ in\right)\left(\frac{1}{2}\ in\right) + \frac{(3\ in)^2}{4(6\ in)}\left(\frac{1}{2}\ in\right) = 4.81\ in^2$$

Example 3-3

For the two lines of bolt holes shown in Fig. 3.6, determine the pitch that will give a net area DEFG equal to the one along ABC. The problem may also be stated as follows: Determine the pitch that will give a net area equal to the gross area less one bolt hole. The holes are punched for 3/4-in bolts.

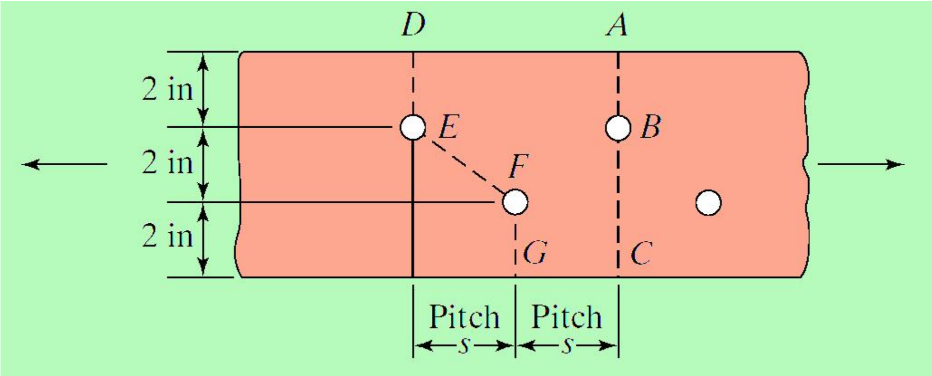


FIGURE 3.6

Solution

The hole diameters to be subtracted are 3/4 in + 1/8 in = 7/8 in.

$$ABC = 6\ in - (1)\left(\frac{7}{8}\ in\right) = 5.13\ in$$

$$DEFG = 6\ in - 2\left(\frac{7}{8}\ in\right) + \frac{s^2}{4(2\ in)} = 4.25\ in + \frac{s^2}{8\ in}$$

$$ABC = DEFG$$

$$5.13 = 4.25\ in + \frac{s^2}{8\ in}$$

$$s = 2.65\ in$$

The AISC Specification does not include a method for determining the net widths of sections other than plates and angles. For channels,W sections, S sections, and others, the web and flange thicknesses are not the same. As a result, it is necessary to work with net areas rather than net widths. If the holes are placed in straight lines across such a member, the net area can be obtained by simply subtracting the crosssectional areas of the holes from the gross area of the member. If the holes are staggered, the $\frac{s^2}{4g}$ values must be multiplied by the applicable thickness to change it to an area.

Example 3-4

Determine the net area of the $W12 \times 16$ ($A_g = 4.71\ in^2$) shown in Fig. 3.7, assuming that the holes are for 1-in bolts.

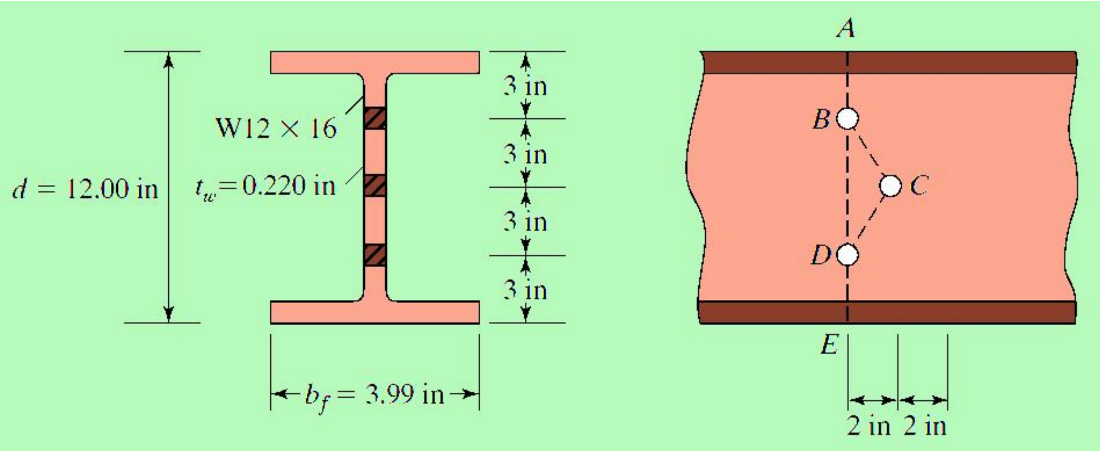


FIGURE 3.7

Solution. Net areas: hole ϕ is 1 in $+ \frac{1}{8}$ in $= 1\frac{1}{8}$ in

$$ABDE = 4.71\text{ in}^2 - 2\left(1\frac{1}{8}\text{ in}\right)(0.220\text{ in}) = 4.21\text{ in}^2$$

$$ABCDE = 4.72\text{ in}^2 - 3\left(1\frac{1}{8}\text{ in}\right)(0.220\text{ in}) + (2)\frac{(2\text{ in})^2}{4(3\text{ in})}(0.220\text{ in}) = 4.11\text{ in}^2 \leftarrow$$

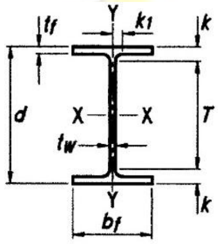


Table 1-1 (continued)

W Shapes

Dimensions

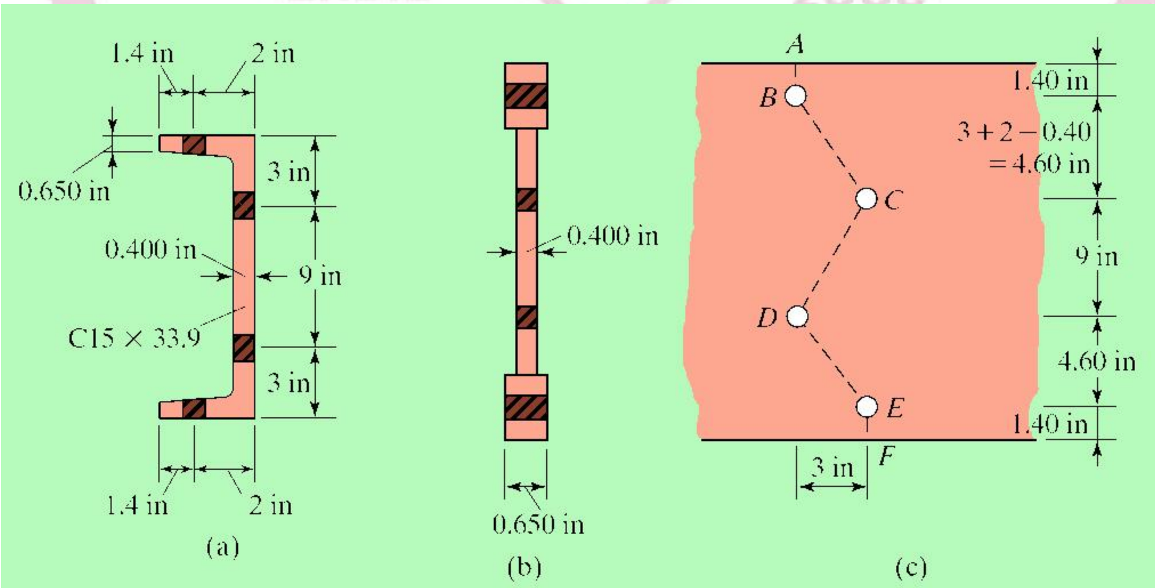
Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k: k_{des}	k: k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8		
W12×35[c]	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4	↓	↓
×26[c]	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4		
W12×22[c]	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 3/8	2 1/4[g]
×19[c]	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16	↓	↓
×16[c]	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	9/16		
×14[c,v]	4.16	11.9	11 7/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16		

If the zigzag line goes from a web hole to a flange hole, the thickness changes at the junction of the flange and web.

In Example 3-5, the net area of a channel that has bolt holes staggered in its flanges and web has been computed. The channel is assumed to be flattened out into a single plate, as shown in parts (b) and (c) of Fig. 3.8. The net area along route ABCDEF is determined by taking the area of the channel minus the area of the holes along the route in the flanges and web plus the $\frac{s^2}{4g}$ values for each zigzag line times the appropriate thickness. For line CD, $\frac{s^2}{4g}$ has been multiplied by the thickness of the web. For lines BC and DE (which run from holes in the web to holes in the flange), an approximate procedure has been used in which the $\frac{s^2}{4g}$ values have been multiplied by the average of the web and flange thicknesses.

Example 3-5

Determine the net area along route ABCDEF for the $C15 \times 33.9$ ($A_g = 10.00\ in^2$) shown in Fig. 3.8.Holes are for ¾ in bolts.



Solution

Approximate net A along

$$ABCDEF = 10.00\text{ in}^2 - 2\left(\frac{7}{8}\text{ in}\right)(0.650\text{ in}) - 2\left(\frac{7}{8}\text{ in}\right)(0.400\text{ in}) + \frac{(3\text{ in})^2}{4(9\text{ in})}(0.400\text{ in}) + (2)\frac{(3\text{ in})^2}{(4)(4.60\text{ in})}\left(\frac{0.650\text{ in} + 0.400\text{ in}}{2}\right)$$

$$= 8.78\text{ in}^2$$

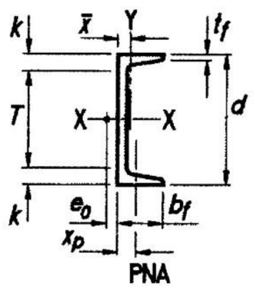


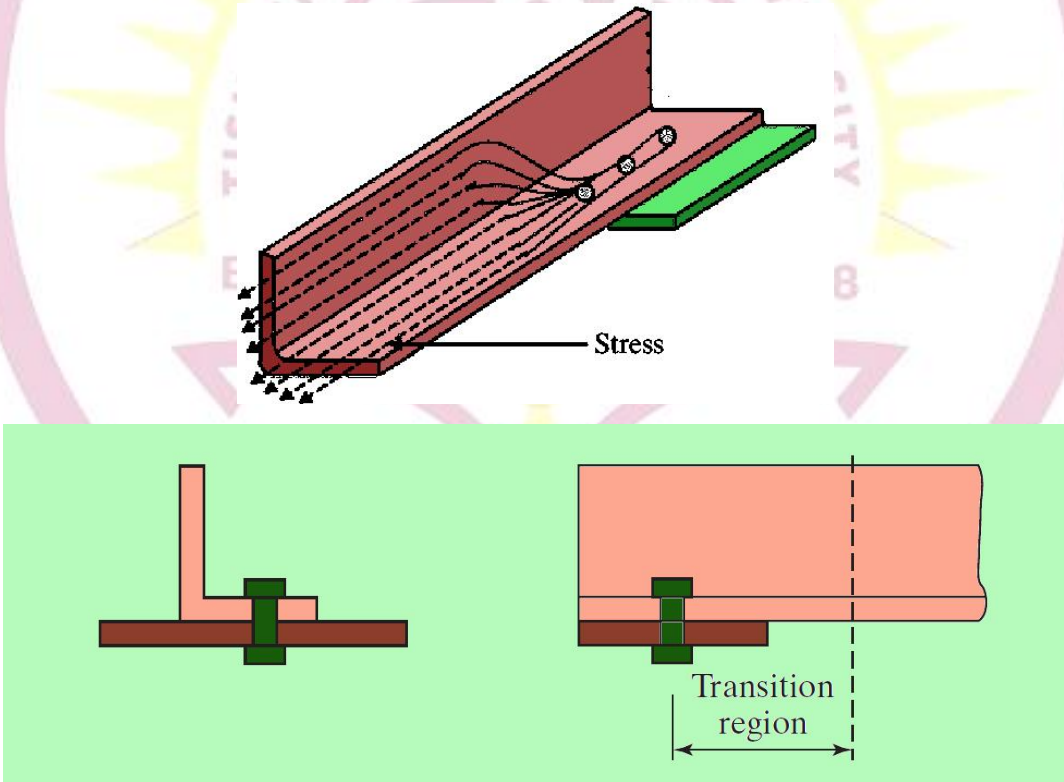
Table 1–5
C Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Distance Workable Gage	r_{ts}	h_o
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4	1.17	14.4
×40	11.8	15.0	15	0.520	1/2	1/4	3.52	3 1/2	0.650	5/8	1 7/16	12 1/8	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	3/8	3/16	3.40	3 3/8	0.650	5/8	1 7/16	12 1/8	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.01	11.5
×25	7.34	12.0	12	0.387	3/8	3/16	3.05	3	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	[illegible]	11.5

EFFECTIVE NET AREAS

If the forces are not transferred uniformly across a member cross section, there will be a transition region of uneven stress running from the connection out long the member for some distance. This is the situation shown in Fig. 3.9(a), where a single angle tension member is connected by one leg only. At the connection more of the load is carried by the connected leg, and it takes the transition distance shown in part (b) of the figure for the stress to spread uniformly across the whole angle.

- In the transition region the stress in the connected part of the member may very well exceed and go into the strain-hardening range.
- In the transition region, the shear transfer has "lagged" and the phenomenon is referred to as shear lag.



(a) Angle connected by one leg only. (b) Stress in transition range > F_y.

FIGURE 3.9 Shear lag.

In such a situation, the flow of tensile stress between the full member cross section and the smaller connected cross section is not 100 percent effective.As a result, the AISC Specification states that the effective net area, A_e of such a member is to be determined by multiplying an area A (which is the net area or the gross area or the directly connected area) by a reduction factor U. The use of a factor such as U accounts for the nonuniform stress distribution, in a simple manner.

$$A_e = A_n U$$

The value of the reduction coefficient, U, is affected by the cross section of the member and by the length of its connection.

Investigators have found that one measure of the effectiveness of a member such as an angle connected by one leg is the:

1. The distance **$\bar{x}$measured from the plane of the connection to the centroid of the area of the whole section.**
2. The length of its connection, L.

The effect of these two parameters, $\bar{x}$ and L, is expressed empirically with the reduction factor

$$U = 1 - \frac{\bar{x}}{L}$$

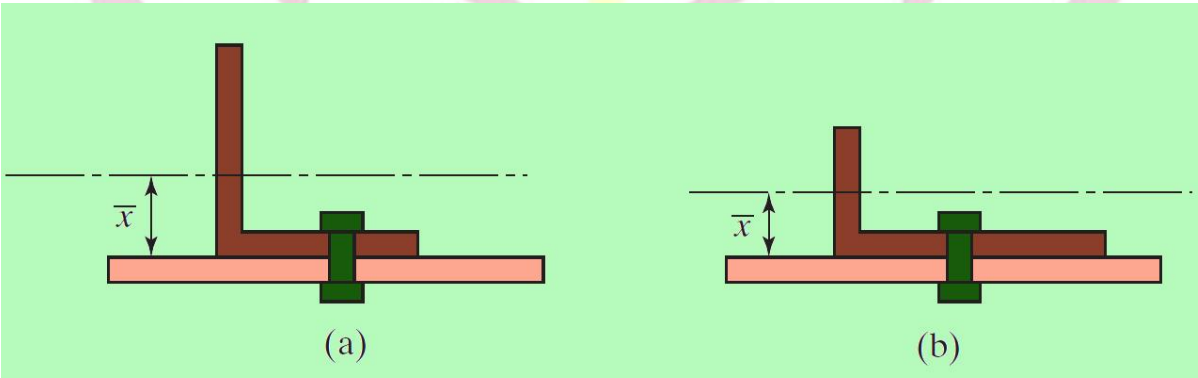


FIGURE 3-10

Bolted Members

Should a tension load be transmitted by bolts, the gross area is reduced to the net area A_n of the member, and U is computed as follows:

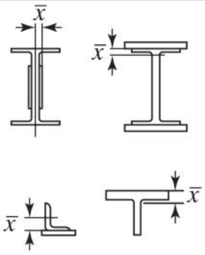
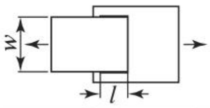
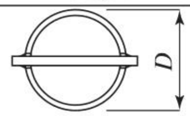
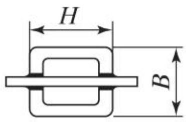
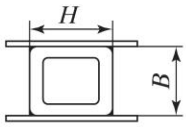
$$U = 1 - \frac{\bar{x}}{L}$$

L is computed as follows

One line of bolts	L is the distance between the first and last bolts in the line.
Two or more lines of bolts	L is the length of the line with the maximum number of bolts.
Blots be staggered	L is the out-to-out dimension between the extreme bolts in a line.

Table 3.2 provides a detailed list of shear lag or U factors for different situations. [This table is a copy of Table D3.1 of the AISC Specification]

TABLE 3.2 Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	——
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	——
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B + H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B + H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with 3 or more fasteners per line in the direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	——
		with web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	——
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in the direction of loading	$U = 0.80$	——
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	——

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm); B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

In order to calculate U for a W section connected by its flanges only, we will assume that the section is split into two structural tees. Then the value of used will be the distance from the outside edge of the flange to the c.g. of the structural tee, as shown in parts (a) and (b) of Fig. 3.11.

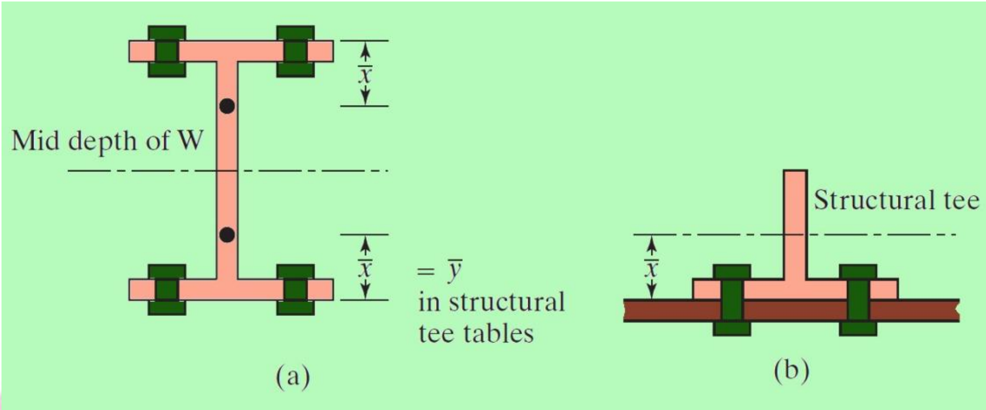


FIGURE 3.11 Values of $\bar{x}$ for different shapes.

Example 3-6

Determine the effective area of a $W10 \times 45$ with two lines of ¾-in diameter bolts in each flange. There are assumed to be at least three bolts in each line 4-in on center, and the bolts are not staggered with respect to each other.

Solution:

Find the area and the flange thickness from the steel manaul

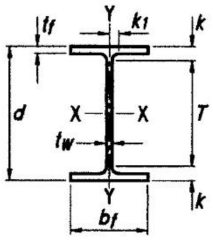


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2
×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16	↓	↓

$$A_n = 13.3 - 4\left(\frac{3}{4}+\frac{1}{8}\right)(0.62) = 11.13$$

Referring to tables in Manual for ond-half of a $W10 \times 45$ (or, that is a $WT5 \times 22.5$), we find that

$$\bar{x} = 0.907\ in$$

Length of connection $L = 8$ in

From Table 3.2 (case 2)

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.907}{8} = 0.89$$

But

$$b_f = 8.02 > \frac{2}{3}d = \left(\frac{2}{3}\right)10.1 = 6.73\ in$$

$\therefore U$ from Table 3.2 (case 7) is 0.9

$$A_e = UA_n = (0.9)(11.13) = 10.02\ in^2$$

Example 3-7

Determine the effective area for $L6 \times 6 \times 3/8$ that is connected at its ends with one line of four 7/8-in-diameter bolts in standard holes 3 in on center in one leg of the angle.

Solution:

Find A_g, $\bar{y}$ and $\bar{x}$ from the steel manual (next page)

$$A_n = 4.38 - (1)\left(\frac{7}{8}+\frac{1}{8}\right)\left(\frac{3}{8}\right) = 4.00\ in^2$$

Length of connection, $L = 9\ in.$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.62}{9} = 0.82$$

From Table 3.2, case 8, for 4 or more fastenrs in the direction of loading, $U = 0.80$.

Use calculated $U = 0.82$

$$A_e = UA_n = (0.82)(4.00) = 3.28\ in^2$$

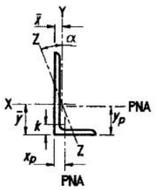


Table 1–7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X I	S	r	ȳ	Z	y_p	Flexural-Torsional Properties J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L8×8×1⅛	1¾	56.9	16.7	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32.5	4.29
×1	1⅝	51.0	15.0	89.1	15.8	2.43	2.36	28.5	0.943	5.08	23.4	4.32
×⅞	1½	45.0	13.2	79.7	14.0	2.45	2.31	25.3	0.832	3.46	16.1	4.36
×¾	1⅜	38.9	11.4	69.9	12.2	2.46	2.26	22.0	0.720	2.21	10.4	4.39
×⅝	1¼	32.7	9.61	59.6	10.3	2.48	2.21	18.6	0.606	1.30	6.16	4.42
×9/16	1 3/16	29.6	8.68	54.2	9.33	2.49	2.19	16.8	0.548	0.961	4.55	4.43
×½	1⅛	26.4	7.75	48.8	8.36	2.49	2.17	15.1	0.490	0.683	3.23	4.45
L8×6×1	1½	44.2	13.0	80.9	15.1	2.49	2.65	27.3	1.47	4.34	16.3	3.88
×⅞	1⅜	39.1	11.5	72.4	13.4	2.50	2.60	24.3	1.41	2.96	11.3	3.92
×¾	1¼	33.8	9.94	63.5	11.7	2.52	2.55	21.1	1.34	1.90	7.28	3.95
×⅝	1⅛	28.5	8.36	54.2	9.86	2.54	2.50	17.9	1.27	1.12	4.33	3.98
×9/16	1 1/16	25.7	7.56	49.4	8.94	2.55	2.48	16.2	1.23	0.823	3.20	3.99
×½	1	23.0	6.75	44.4	8.01	2.55	2.46	14.6	1.20	0.584	2.28	4.01
×7/16	15/16	20.2	5.93	39.3	7.06	2.56	2.43	12.9	1.16	0.396	1.55	4.02
L8×4×1	1½	37.4	11.0	69.7	14.0	2.51	3.03	24.3	2.47	3.68	12.9	3.75
×⅞	1⅜	33.1	9.73	62.6	12.5	2.53	2.99	21.7	2.41	2.51	8.89	3.78
×¾	1¼	28.7	8.44	55.0	10.9	2.55	2.94	18.9	2.34	1.61	5.75	3.80
×⅝	1⅛	24.2	7.11	47.0	9.20	2.56	2.89	16.1	2.27	0.955	3.42	3.83
×9/16	1 1/16	21.9	6.43	42.9	8.34	2.57	2.86	14.6	2.23	0.704	2.53	3.84
×½	1	19.6	5.75	38.6	7.48	2.58	2.84	13.1	2.20	0.501	1.80	3.86
×7/16	15/16	17.2	5.06	34.2	6.59	2.59	2.81	11.6	2.16	0.340	1.22	3.87
L7×4×¾	1¼	26.2	7.69	37.8	8.39	2.21	2.50	14.8	1.87	1.47	3.97	3.31
×⅝	1⅛	22.1	6.48	32.4	7.12	2.23	2.45	12.5	1.80	0.868	2.37	3.34
×½	1	17.9	5.25	26.6	5.79	2.25	2.40	10.2	1.74	0.456	1.25	3.37
×7/16	15/16	15.7	4.62	23.6	5.11	2.26	2.38	9.03	1.70	0.310	0.851	3.38
×⅜	⅞	13.6	3.98	20.5	4.42	2.27	2.35	7.81	1.67	0.198	0.544	3.40
L6×6×1	1½	37.4	11.0	35.4	8.55	1.79	1.86	15.4	0.918	3.68	9.24	3.18
×⅞	1⅜	33.1	9.75	31.9	7.61	1.81	1.81	13.7	0.813	2.51	6.41	3.21
×¾	1¼	28.7	8.46	28.1	6.64	1.82	1.77	11.9	0.705	1.61	4.17	3.24
×⅝	1⅛	24.2	7.13	24.1	5.64	1.84	1.72	10.1	0.594	0.955	2.50	3.28
×9/16	1 1/16	21.9	6.45	22.0	5.12	1.85	1.70	9.18	0.538	0.704	1.85	3.29
×½	1	19.6	5.77	19.9	4.59	1.86	1.67	8.22	0.481	0.501	1.32	3.31
×7/16	15/16	17.2	5.08	17.6	4.06	1.86	1.65	7.25	0.423	0.340	0.899	3.32
×⅜	⅞	14.9	4.38	15.4	3.51	1.87	1.62	6.27	0.365	0.218	0.575	3.34
×5/16	13/16	12.4	3.67	13.0	2.95	1.88	1.60	5.26	0.306	0.129	0.338	3.35

Note: For compactness criteria, refer to the end of Table 1–7.

Table 1–7 (continued)
Angles
Properties

L8-L6

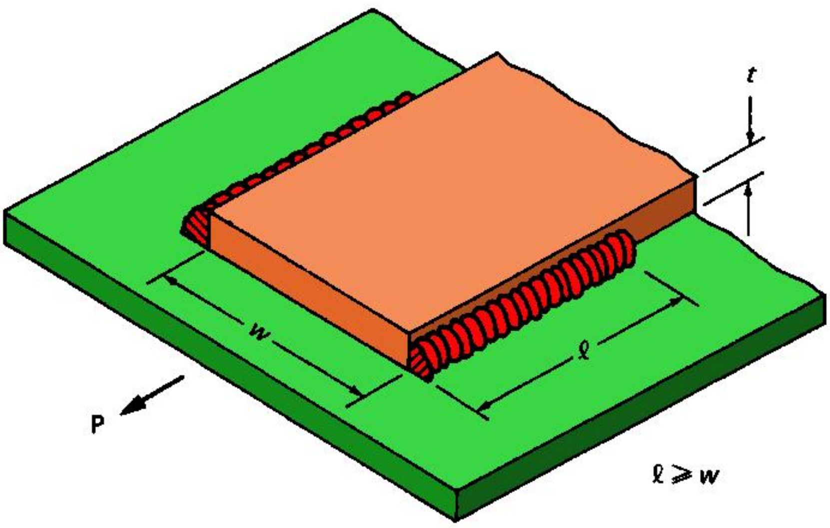
Shape	Axis Y-Y I	S	r	x̄	Z	x_p	Axis Z-Z I	S	r	Tan α	Q_s F_y = 36 ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L8×8×1⅛	98.1	17.5	2.41	2.40	31.6	1.05	40.9	7.23	1.56	1.00	1.00
×1	89.1	15.8	2.43	2.36	28.5	0.943	36.8	6.51	1.56	1.00	1.00
×⅞	79.7	14.0	2.45	2.31	25.3	0.832	32.7	5.78	1.57	1.00	1.00
×¾	69.9	12.2	2.46	2.26	22.0	0.720	28.5	5.04	1.57	1.00	1.00
×⅝	59.6	10.3	2.48	2.21	18.6	0.606	24.2	4.27	1.58	1.00	0.997
×9/16	54.2	9.33	2.49	2.19	16.8	0.548	22.0	3.88	1.58	1.00	0.959
×½	48.8	8.36	2.49	2.17	15.1	0.490	19.7	3.49	1.59	1.00	0.912
L8×6×1	38.8	8.92	1.72	1.65	16.2	0.816	21.3	4.84	1.28	0.542	1.00
×⅞	34.9	7.94	1.74	1.60	14.4	0.721	18.9	4.31	1.28	0.546	1.00
×¾	30.8	6.92	1.75	1.56	12.5	0.624	16.5	3.78	1.29	0.550	1.00
×⅝	26.4	5.88	1.77	1.51	10.5	0.526	14.1	3.22	1.29	0.554	0.997
×9/16	24.1	5.34	1.78	1.49	9.52	0.476	12.8	2.94	1.30	0.556	0.959
×½	21.7	4.79	1.79	1.46	8.52	0.425	11.5	2.64	1.30	0.557	0.912
×7/16	19.3	4.23	1.80	1.44	7.50	0.374	10.2	2.35	1.31	0.559	0.850
L8×4×1	11.6	3.94	1.03	1.04	7.73	0.691	7.87	2.15	0.844	0.247	1.00
×⅞	10.5	3.51	1.04	0.997	6.77	0.612	7.01	1.93	0.846	0.252	1.00
×¾	9.37	3.07	1.05	0.949	5.82	0.531	6.13	1.70	0.850	0.257	1.00
×⅝	8.11	2.62	1.06	0.902	4.86	0.448	5.24	1.47	0.856	0.262	0.997
×9/16	7.44	2.38	1.07	0.878	4.39	0.405	4.79	1.34	0.859	0.264	0.959
×½	6.75	2.15	1.08	0.854	3.91	0.363	4.32	1.22	0.863	0.266	0.912
×7/16	6.03	1.90	1.09	0.829	3.42	0.320	3.84	1.09	0.867	0.268	0.850
L7×4×¾	9.00	3.01	1.08	1.00	5.60	0.550	5.64	1.71	0.855	0.324	1.00
×⅝	7.79	2.56	1.10	0.958	4.69	0.464	4.80	1.47	0.860	0.329	1.00
×½	6.48	2.10	1.11	0.910	3.77	0.376	3.95	1.21	0.866	0.334	0.965
×7/16	5.79	1.86	1.12	0.886	3.31	0.331	3.50	1.08	0.869	0.337	0.912
×⅜	5.06	1.61	1.12	0.861	2.84	0.286	3.05	0.942	0.873	0.339	0.840
L6×6×1	35.4	8.55	1.79	1.86	15.4	0.918	15.0	3.53	1.17	1.00	1.00
×⅞	31.9	7.61	1.81	1.81	13.7	0.813	13.3	3.13	1.17	1.00	1.00
×¾	28.1	6.64	1.82	1.77	11.9	0.705	11.6	2.73	1.17	1.00	1.00
×⅝	24.1	5.64	1.84	1.72	10.1	0.594	9.83	2.32	1.17	1.00	1.00
×9/16	22.0	5.12	1.85	1.70	9.17	0.538	8.94	2.11	1.18	1.00	1.00
×½	19.9	4.59	1.86	1.67	8.22	0.481	8.04	1.89	1.18	1.00	1.00
×7/16	17.6	4.06	1.86	1.65	7.25	0.423	7.11	1.68	1.18	1.00	0.973
×⅜	15.4	3.51	1.87	1.62	6.26	0.365	6.17	1.45	1.19	1.00	0.912
×5/16	13.0	2.95	1.88	1.60	5.26	0.306	5.20	1.23	1.19	1.00	0.826

Note: For compactness criteria, refer to the end of Table 1–7.

Welded Members

When tension loads are transferred by welds, the rules from AISC (Table 3.2 in this text) that are to be used to determine values for A and U (A_e as for bolted connections $= AU$) are as follows:

1. Should the load be transmitted only by longitudinal welds to other than a plate member, or by longitudinal welds in combination with transverse welds, A is to equal the gross area of the member A_g (Table 3.2, Case 2).
2. Should a tension load be transmitted only by transverse welds, A is to equal the area of the directly connected elements and U is to equal 1.0 (Table 3.2, Case 3).
3. For flat plates or bars connected by longitudinal fillet welds, use the values of U listed in Table 3.2, Case 4.



	Length, ℓ	U
$A_g = wt$	$\ell > 2w$	1.0
$A_e = UA_g$	$2w > \ell > 1.5w$	0.87
	$1.5w > \ell > w$	0.75

FIGURE 3.12 a

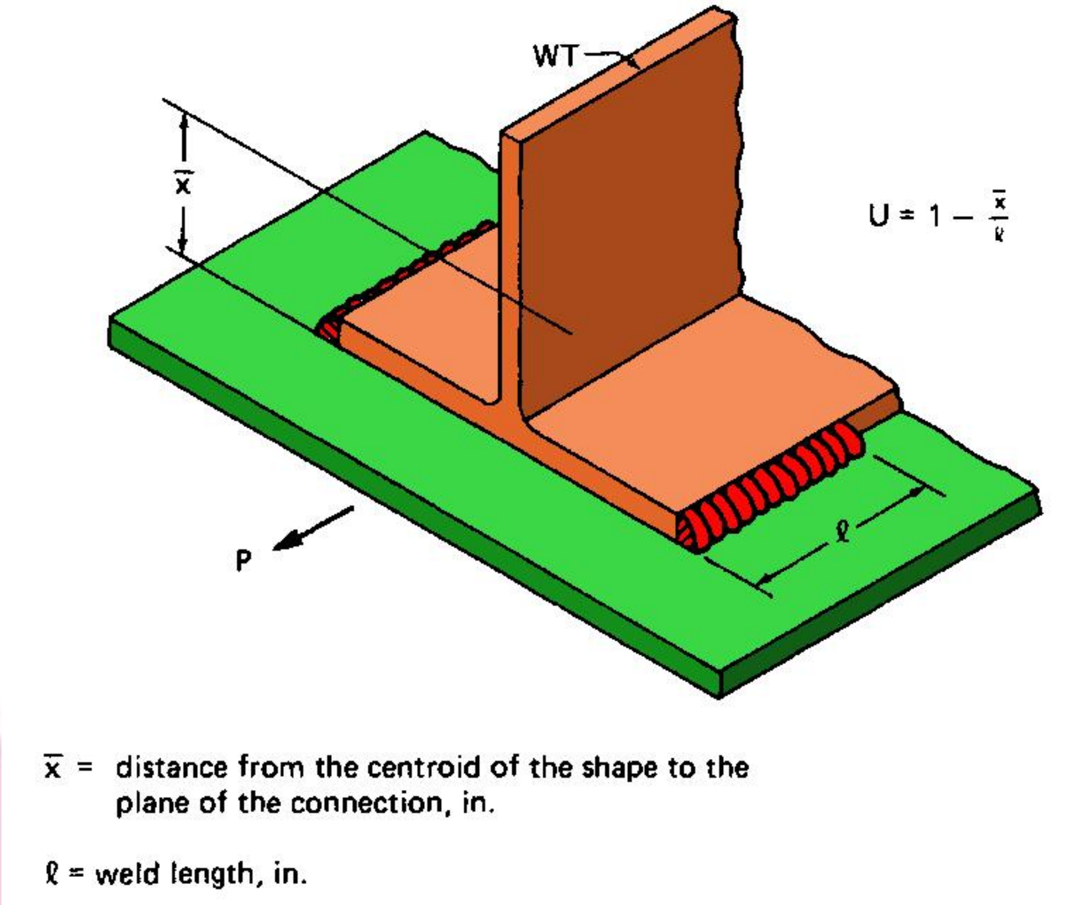


FIGURE 3.12 b

Example 3-8

The $1 \times 6\ in$ plate shown in Fig. 3.13 is connected to a $1 \times 10\ in$ plate with longitudinal fillet welds to transfer a tensile load. Determine the effecitve area.

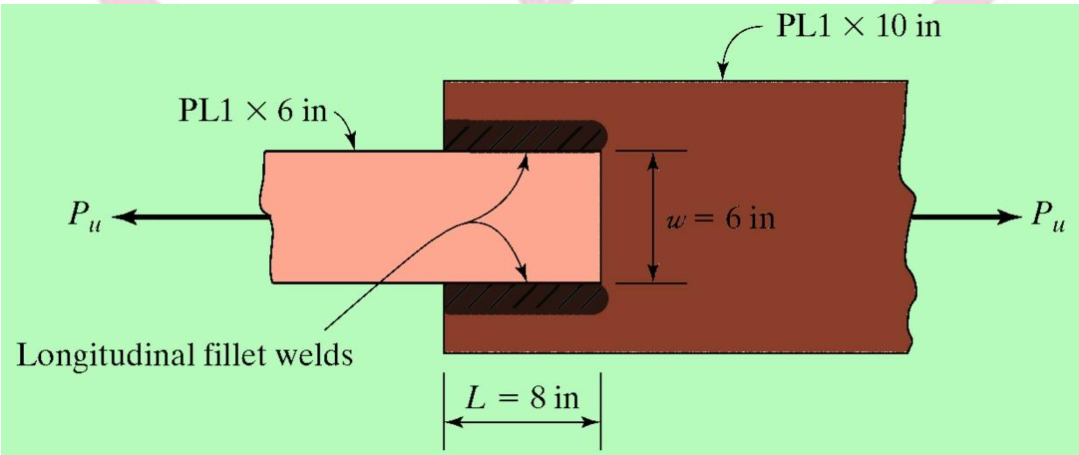


FIGURE 3.13

Solution:

$$(1.5w = 1.5 \times 6 = 9\ in.) > (L = 8\ in.) > (w = 6)$$

$$\therefore U = 0.75 \text{from Table 3.2 case 4}$$

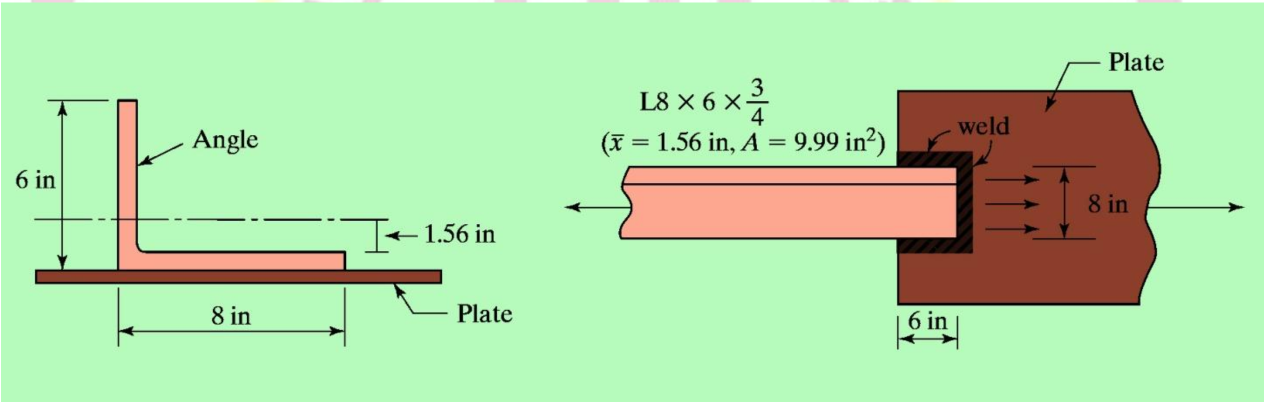
$$A_e = UA_n = (0.75)(6.0) = 4.5\ in^2$$

……………………………………………….

Sometimes an angle has one of its legs connected with both longitudinal and transverse welds, but no connections are made to the other leg.To determine U from Table 3.2 for such a case is rather puzzling. sometimes Case 2 of Table 3.2 (that is, $U = 1 - \frac{\bar{x}}{L}$) is be used for this situation.This is done in Example 3-9.

Example 3-9

Compute the net area for the angle shown in the Figure below. It is welded on the end (transverse) and sides (longitudinal) of the 8-in leg only.



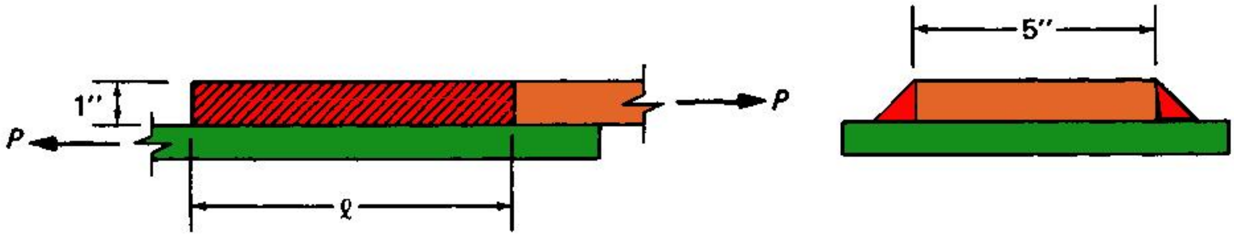
Solution:

Nominal or available tensile strength of the angle

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.56}{6} = 0.74$$

$$A_e = UA_g = (0.74)(9.99) = 7.39\ in^2$$

Example Determine A_e for the 1 × 5 in. plate shown below if (a) l = 7 in., (b) l = 8.5 in., (c) l = 11 in.



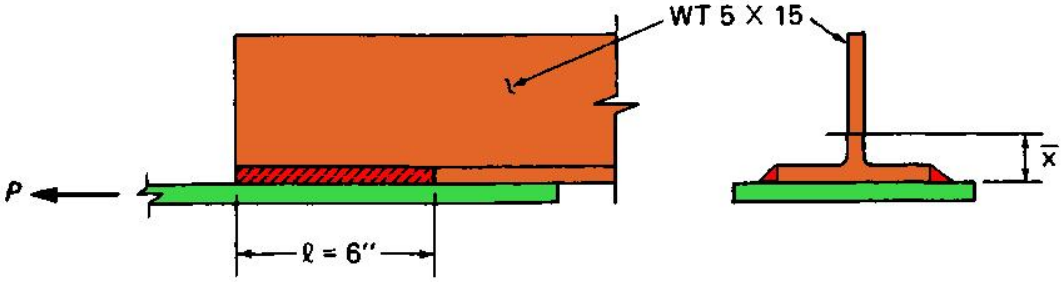
Solution. $A_e = UA_g$

$$A_g = 1 \text{ in.} \times 5 \text{ in.} = 5 \text{ in.}^2$$

$$w = 5 \text{ in.},\ 1.5w = 7.5 \text{ in.},\ 2w = 10 \text{ in.}$$

a) Since $1.5w > l > w$ in this case, then $U = 0.75$ and $A_e = 0.75 \times 5 = 3.75$ in.^{2}
b) Since $2w > l > 1.5w$ in this case, then $U = 0.87$ and $A_e = 0.87 \times 5 = 4.35$ in.^{2}
c) Since $l > 2w$, then $U = 1.0$ and $A_e = 5$ in.^{2}

Example Determine A_e for the WT 5 × 15 shown below.



Solution.

$$U = 1 - \frac{\bar{x}}{l}$$

$\bar{x} = 1.1$ in. (see AISCM Properties Section)

$l = 6$ in.

$$U = 1 - \frac{1.1}{6} = 0.82$$

$$A_e = UA_g = 0.82 \times 4.42 = 3.61 \text{ in.}^2$$

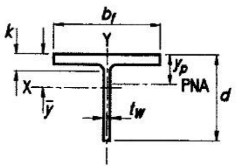


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem Thickness, t_w		$\frac{t_w}{2}$	Stem Area	Flange Width, b_f		Flange Thickness, t_f		Distance k		Workable Gage
	in.²	in.		in.		in.	in.²	in.		in.		in.	in.	in.
WT6×11[c]	3.24	6.16	6 1/8	0.260	1/4	1/8	1.60	4.03	4	0.425	7/16	0.725	15/16	2 1/4[g]
×9.5[c]	2.79	6.08	6 1/8	0.235	1/4	1/8	1.43	4.01	4	0.350	3/8	0.650	7/8	↓
×8[c]	2.36	6.00	6	0.220	1/4	1/8	1.32	3.99	4	0.265	1/4	0.565	13/16	↓
×7[c,v]	2.08	5.96	6	0.200	3/16	1/8	1.19	3.97	4	0.225	1/4	0.525	3/4	↓
WT5×56	16.5	5.68	5 5/8	0.755	3/4	3/8	4.29	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	5 1/2
×50	14.7	5.55	5 1/2	0.680	11/16	3/8	3.77	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	↓
×44	12.9	5.42	5 3/8	0.605	5/8	5/16	3.28	10.3	10 1/4	0.990	1	1.49	1 11/16	↓
×38.5	11.3	5.30	5 1/4	0.530	1/2	1/4	2.81	10.2	10 1/4	0.870	7/8	1.37	1 9/16	↓
×34	9.99	5.20	5 1/4	0.470	1/2	1/4	2.44	10.1	10 1/8	0.770	3/4	1.27	1 7/16	↓
×30	8.82	5.11	5 1/8	0.420	7/16	1/4	2.15	10.1	10 1/8	0.680	11/16	1.18	1 3/8	↓
×27	7.91	5.05	5	0.370	3/8	3/16	1.87	10.0	10	0.615	5/8	1.12	1 5/16	↓
×24.5	7.21	4.99	5	0.340	5/16	3/16	1.70	10.0	10	0.560	9/16	1.06	1 1/4	↓
WT5×22.5	6.63	5.05	5	0.350	3/8	3/16	1.77	8.02	8	0.620	5/8	1.12	1 5/16	↓
×19.5	5.73	4.96	5	0.315	5/16	3/16	1.56	7.99	8	0.530	1/2	1.03	1 3/16	↓
×16.5	4.85	4.87	4 7/8	0.290	5/16	3/16	1.41	7.96	8	0.435	7/16	0.935	1 1/8	↓
WT5×15	**4.42**	**5.24**	**5 1/4**	**0.300**	**5/16**	**3/16**	**1.57**	**5.81**	**5 3/4**	**0.510**	**1/2**	**0.810**	**1 1/8**	**2 3/4[g]**
×13[c]	3.81	5.17	5 1/8	0.260	1/4	1/8	1.34	5.77	5 3/4	0.440	7/16	0.740	1 1/16	↓
×11[c]	3.24	5.09	5 1/8	0.240	1/4	1/8	1.22	5.75	5 3/4	0.360	3/8	0.660	15/16	↓
WT5×9.5[c]	2.81	5.12	5 1/8	0.250	1/4	1/8	1.28	4.02	4	0.395	3/8	0.695	15/16	2 1/4[g]
×8.5[c]	2.50	5.06	5	0.240	1/4	1/8	1.21	4.01	4	0.330	5/16	0.630	7/8	↓
×7.5[c]	2.21	5.00	5	0.230	1/4	1/8	1.15	4.00	4	0.270	1/4	0.570	13/16	↓
×6[c,f]	1.77	4.94	4 7/8	0.190	3/16	1/8	0.938	3.96	4	0.210	3/16	0.510	3/4	↓
WT4×33.5	9.84	4.50	4 1/2	0.570	9/16	5/16	2.57	8.28	8 1/4	0.935	15/16	1.33	1 5/8	5 1/2
×29	8.54	4.38	4 3/8	0.510	1/2	1/4	2.23	8.22	8 1/4	0.810	13/16	1.20	1 1/2	↓
×24	7.05	4.25	4 1/4	0.400	3/8	3/16	1.70	8.11	8 1/8	0.685	11/16	1.08	1 3/8	↓
×20	5.87	4.13	4 1/8	0.360	3/8	3/16	1.49	8.07	8 1/8	0.560	9/16	0.954	1 1/4	↓
×17.5	5.14	4.06	4	0.310	5/16	3/16	1.26	8.02	8	0.495	1/2	0.889	1 3/16	↓
×15.5[f]	4.56	4.00	4	0.285	5/16	3/16	1.14	8.00	8	0.435	7/16	0.829	1 1/8	↓
WT4×14	4.12	4.03	4	0.285	5/16	3/16	1.15	6.54	6 1/2	0.465	7/16	0.859	15/16	3 1/2
×12	3.54	3.97	4	0.245	1/4	1/8	0.971	6.50	6 1/2	0.400	3/8	0.794	7/8	3 1/2

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[v] Shear strength controlled by buckling effects (C_v < 1.0) with F_y = 50 ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT6-WT4

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	Axis X-X I	S	r	ȳ	Z	y_p	Axis Y-Y I	S	r	Z	Q_s (F_y = 50 ksi)	Torsional J	C_w
lb/ft			in.⁴	in.³	in.	in.	in.³	in.	in.⁴	in.³	in.	in.³		in.⁴	in.⁶
11	4.74	23.7	11.7	2.59	1.90	1.63	4.63	0.402	2.33	1.15	0.847	1.83	0.711	0.146	0.137
9.5	5.72	25.9	10.1	2.28	1.90	1.65	4.11	0.348	1.88	0.939	0.821	1.49	0.598	0.0899	0.0934
8	7.53	27.3	8.70	2.04	1.92	1.74	3.72	0.639	1.41	0.706	0.773	1.13	0.539	0.0511	0.0678
7	8.82	29.8	7.67	1.83	1.92	1.76	3.32	0.760	1.18	0.593	0.753	0.947	0.451	0.0350	0.0493
56	4.17	7.52	28.6	6.40	1.32	1.21	13.4	0.791	118	22.6	2.67	34.6	1.00	7.50	16.9
50	4.62	8.16	24.5	5.56	1.29	1.13	11.4	0.711	103	20.0	2.65	30.5	1.00	5.41	11.9
44	5.18	8.96	20.8	4.77	1.27	1.06	9.65	0.631	89.3	17.4	2.63	26.5	1.00	3.75	8.02
38.5	5.86	10.0	17.4	4.05	1.24	0.990	8.06	0.555	76.8	15.1	2.60	22.9	1.00	2.55	5.31
34	6.58	11.1	14.9	3.49	1.22	0.932	6.85	0.493	66.7	13.2	2.58	20.0	1.00	1.78	3.62
30	7.41	12.2	12.9	3.04	1.21	0.884	5.87	0.438	58.1	11.5	2.57	17.5	1.00	1.23	2.46
27	8.15	13.6	11.1	2.64	1.19	0.836	5.05	0.395	51.7	10.3	2.56	15.6	1.00	0.909	1.78
24.5	8.93	14.7	10.0	2.39	1.18	0.807	4.52	0.361	46.7	9.34	2.54	14.1	1.00	0.693	1.33
22.5	6.47	14.4	10.2	2.47	1.24	0.907	4.65	0.413	26.7	6.65	2.01	10.1	1.00	0.753	0.981
19.5	7.53	15.7	8.84	2.16	1.24	0.876	3.99	0.359	22.5	5.64	1.98	8.57	1.00	0.487	0.616
16.5	9.15	16.8	7.71	1.93	1.26	0.869	3.48	0.305	18.3	4.60	1.94	7.00	1.00	0.291	0.356
15	**5.70**	**17.5**	**9.28**	**2.24**	**1.45**	**1.10**	**4.01**	**0.380**	**8.35**	**2.87**	**1.37**	**4.41**	**1.00**	**0.310**	**0.273**
13	6.56	19.9	7.86	1.91	1.44	1.06	3.39	0.330	7.05	2.44	1.36	3.75	0.904	0.201	0.173
11	7.99	21.2	6.88	1.72	1.46	1.07	3.02	0.282	5.71	1.99	1.33	3.05	0.837	0.119	0.107
9.5	5.09	20.5	6.68	1.74	1.54	1.28	3.10	0.349	2.15	1.07	0.874	1.67	0.873	0.116	0.0796
8.5	6.08	21.1	6.06	1.62	1.56	1.32	2.90	0.311	1.78	0.887	0.844	1.40	0.843	0.0776	0.0610
7.5	7.41	21.7	5.45	1.50	1.57	1.37	2.71	0.305	1.45	0.723	0.810	1.15	0.810	0.0518	0.0475
6	9.43	26.0	4.35	1.22	1.57	1.36	2.20	0.322	1.09	0.551	0.785	0.869	0.593	0.0272	0.0255
33.5	4.43	7.89	10.9	3.05	1.05	0.936	6.29	0.594	44.3	10.7	2.12	16.3	1.00	2.51	3.56
29	5.07	8.58	9.12	2.61	1.03	0.874	5.25	0.520	37.5	9.13	2.10	13.9	1.00	1.66	2.28
24	5.92	10.6	6.85	1.97	0.986	0.777	3.94	0.435	30.5	7.51	2.08	11.4	1.00	0.977	1.30
20	7.21	11.5	5.73	1.69	0.988	0.735	3.25	0.364	24.5	6.08	2.04	9.24	1.00	0.558	0.715
17.5	8.10	13.1	4.82	1.43	0.968	0.688	2.71	0.321	21.3	5.31	2.03	8.05	1.00	0.384	0.480
15.5	9.19	14.0	4.28	1.28	0.969	0.668	2.39	0.285	18.5	4.64	2.02	7.03	1.00	0.267	0.327
14	7.03	14.1	4.23	1.28	1.01	0.734	2.38	0.315	10.8	3.31	1.62	5.04	1.00	0.268	0.230
12	8.12	16.2	3.53	1.08	0.999	0.695	1.98	0.272	9.14	2.81	1.61	4.28	1.00	0.173	0.144

CHAPTER 4

Design of Tension Members

SELECTION OF SECTIONS

Accoridng to the LRFD equations, the design strength of a tension member is the least of ,$\phi_t F_y A_g$, $\phi_t F_u A_e$. In addition, the slenderness ratio should, preferably, not exceed 300.

a. To satisfy the first of these expressions, the minimum gross area must be at least equal to

$$\min A_g = \frac{P_u}{\phi_t F_y}$$

b. To satisfy the second expression, the minimum A_g is

$$\min A_g = \frac{P_u}{\phi_t F_u U} + \text{ estimated area of holes}$$

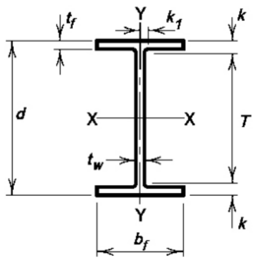
c. The maximum preferable slenderness ratio L/r is 300

$$\min r = \frac{L}{300}$$

Usually, for the examples,D and L loads are specified so that we will not have to go through all of the load combination expressions. For such problems, then, we will need only to use the following load combinations:

$$P_u = 1.4D$$

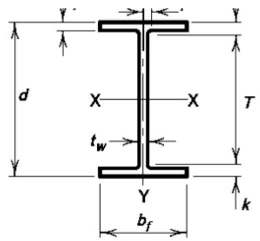
$$P_u = 1.2D + 1.6L$$



W SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		$t_w/2$	Flange Width b_f		Flange Thickness t_f		Distance T	k	k_1
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.
W12×336*	98.8	16.82	16 7/8	1.775	1 3/4	7/8	13.385	13 3/8	2.955	2 15/16	9 1/2	3 11/16	1 1/2
×305*	89.6	16.32	16 3/8	1.625	1 5/8	13/16	13.235	13 1/4	2.705	2 11/16	9 1/2	3 7/16	1 7/16
×279*	81.9	15.85	15 7/8	1.530	1 1/2	3/4	13.140	13 1/8	2.470	2 1/2	9 1/2	3 3/16	1 3/8
×252*	74.1	15.41	15 3/8	1.395	1 3/8	11/16	13.005	13	2.250	2 1/4	9 1/2	2 15/16	1 5/16
×230*	67.7	15.05	15	1.285	1 5/16	11/16	12.895	12 7/8	2.070	2 1/16	9 1/2	2 3/4	1 1/4
×210*	61.8	14.71	14 3/4	1.180	1 3/16	5/8	12.790	12 3/4	1.900	1 7/8	9 1/2	2 5/8	1 1/4
×190	55.8	14.38	14 3/8	1.060	1 1/16	9/16	12.670	12 5/8	1.735	1 3/4	9 1/2	2 7/16	1 3/16
×170	50.0	14.03	14	0.960	15/16	1/2	12.570	12 5/8	1.560	1 9/16	9 1/2	2 1/4	1 1/8
×152	44.7	13.71	13 3/4	0.870	7/8	7/16	12.480	12 1/2	1.400	1 3/8	9 1/2	2 1/8	1 1/16
×136	39.9	13.41	13 3/8	0.790	13/16	7/16	12.400	12 3/8	1.250	1 1/4	9 1/2	1 15/16	1
×120	35.3	13.12	13 1/8	0.710	11/16	3/8	12.320	12 3/8	1.105	1 1/8	9 1/2	1 13/16	1
×106	31.2	12.89	12 7/8	0.610	5/8	5/16	12.220	12 1/4	0.990	1	9 1/2	1 11/16	15/16
×96	28.2	12.71	12 3/4	0.550	9/16	5/16	12.160	12 1/8	0.900	7/8	9 1/2	1 5/8	7/8
×87	25.6	12.53	12 1/2	0.515	1/2	1/4	12.125	12 1/8	0.810	13/16	9 1/2	1 1/2	7/8
×79	23.2	12.38	12 3/8	0.470	1/2	1/4	12.080	12 1/8	0.735	3/4	9 1/2	1 7/16	7/8
×72	21.1	12.25	12 1/4	0.430	7/16	1/4	12.040	12	0.670	11/16	9 1/2	1 3/8	7/8
×65	19.1	12.12	12 1/8	0.390	3/8	3/16	12.000	12	0.605	5/8	9 1/2	1 5/16	13/16
W12×58	17.0	12.19	12 1/4	0.360	3/8	3/16	10.010	10	0.640	5/8	9 1/2	1 3/8	13/16
×53	15.6	12.06	12	0.345	3/8	3/16	9.995	10	0.575	9/16	9 1/2	1 1/4	13/16
W12×50	14.7	12.19	12 1/4	0.370	3/8	3/16	8.080	8 1/8	0.640	5/8	9 1/2	1 3/8	13/16
×45	13.2	12.06	12	0.335	5/16	3/16	8.045	8	0.575	9/16	9 1/2	1 1/4	13/16
×40	11.8	11.94	12	0.295	5/16	3/16	8.005	8	0.515	1/2	9 1/2	1 1/4	3/4
W12×35	10.3	12.50	12 1/2	0.300	5/16	3/16	6.560	6 1/2	0.520	1/2	10 1/2	1	9/16
×30	8.79	12.34	12 3/8	0.260	1/4	1/8	6.520	6 1/2	0.440	7/16	10 1/2	15/16	1/2
×26	7.65	12.22	12 1/4	0.230	1/4	1/8	6.490	6 1/2	0.380	3/8	10 1/2	7/8	1/2
W12×22	6.48	12.31	12 1/4	0.260	1/4	1/8	4.030	4	0.425	7/16	10 1/2	7/8	1/2
×19	5.57	12.16	12 1/8	0.235	1/4	1/8	4.005	4	0.350	3/8	10 1/2	13/16	1/2
×16	4.71	11.99	12	0.220	1/4	1/8	3.990	4	0.265	1/4	10 1/2	3/4	1/2
×14	4.16	11.91	11 7/8	0.200	3/16	1/8	3.970	4	0.225	1/4	10 1/2	11/16	1/2

Properties



Nominal Wt. per ft	Compact Section Criteria $\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	F_y'''	X_1	$X_2 \times 10^6$	Axis X-X I	S	r	Axis Y-Y I	S	r	Plastic Modulus Z_x	Z_y
lb			ksi	ksi	(1/ksi)^2	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{3}
336	2.3	5.5	—	12800	6.05	4060	483	6.41	1190	177	3.47	603	274
305	2.4	6.0	—	11800	8.17	3550	435	6.29	1050	159	3.42	537	244
279	2.7	6.3	—	11000	10.8	3110	393	6.16	937	143	3.38	481	220
252	2.9	7.0	—	10100	14.7	2720	353	6.06	828	127	3.34	428	196
230	3.1	7.6	—	9390	19.7	2420	321	5.97	742	115	3.31	386	177
210	3.4	8.2	—	8670	26.6	2140	292	5.89	664	104	3.28	348	159
190	3.7	9.2	—	7940	37.0	1890	263	5.82	589	93.0	3.25	311	143
170	4.0	10.1	—	7190	54.0	1650	235	5.74	517	82.3	3.22	275	126
152	4.5	11.2	—	6510	79.3	1430	209	5.66	454	72.8	3.19	243	111
136	5.0	12.3	—	5850	119	1240	186	5.58	398	64.2	3.16	214	98.0
120	5.6	13.7	—	5240	184	1070	163	5.51	345	56.0	3.13	186	85.4
106	6.2	15.9	—	4660	285	933	145	5.47	301	49.3	3.11	164	75.1
96	6.8	17.7	—	4250	405	833	131	5.44	270	44.4	3.09	147	67.5
87	7.5	18.9	—	3880	586	740	118	5.38	241	39.7	3.07	132	60.4
79	8.2	20.7	—	3530	839	662	107	5.34	216	35.8	3.05	119	54.3
72	9.0	22.6	—	3230	1180	597	97.4	5.31	195	32.4	3.04	108	49.2
65	9.9	24.9	—	2940	1720	533	87.9	5.28	174	29.1	3.02	96.8	44.1
58	7.8	27.0	—	3070	1470	475	78.0	5.28	107	21.4	2.51	86.4	32.5
53	8.7	28.1	—	2820	2100	425	70.6	5.23	95.8	19.2	2.48	77.9	29.1
50	6.3	26.2	—	3170	1410	394	64.7	5.18	56.3	13.9	1.96	72.4	21.4
45	7.0	29.0	—	2870	2070	350	58.1	5.15	50.0	12.4	1.94	64.7	19.0
40	7.8	32.9	59	2580	3110	310	51.9	5.13	44.1	11.0	1.93	57.5	16.8
35	6.3	36.2	49	2420	4340	285	45.6	5.25	24.5	7.47	1.54	51.2	11.5
30	7.4	41.8	37	2090	7950	238	38.6	5.21	20.3	6.24	1.52	43.1	9.56
26	8.5	47.2	29	1820	13900	204	33.4	5.17	17.3	5.34	1.51	37.2	8.17
22	4.7	41.8	37	2160	8640	156	25.4	4.91	4.66	2.31	0.847	29.3	3.66
19	5.7	46.2	30	1880	15600	130	21.3	4.82	3.76	1.88	0.822	24.7	2.98
16	7.5	49.4	26	1610	32000	103	17.1	4.67	2.82	1.41	0.773	20.1	2.26
14	8.8	54.3	22	1450	49300	88.6	14.9	4.62	2.36	1.19	0.753	17.4	1.90

Example 4-1

Select a 30-ft-long W12 steel to support a tensile service dead load $P_D = 130\ k$ and a tensile service live load $P_L = 110\ k$. As shown in Fig. 4.1, the member is to have two lines of bolts in each flange for 7/8-in bolts (at least three in a line 4 in on center). [$F_y = 50\ ksi, F_u = 65\ ksi$].

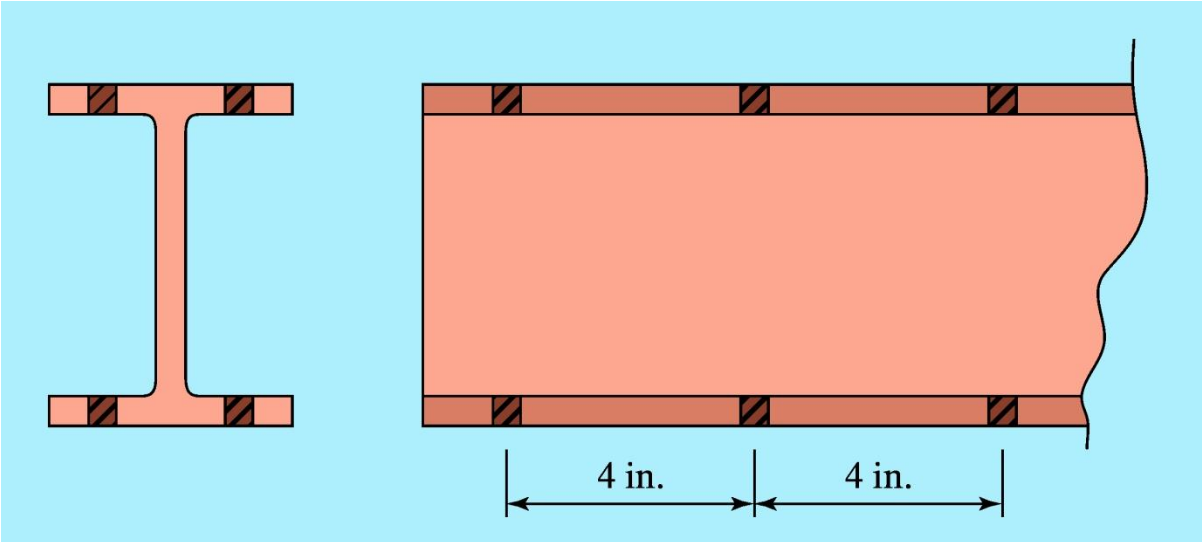


FIGURE 4.1 Cross section of member for Example 4-1.

Solution

(a) considering load the necessary load combinaitons

$$P_u = 1.4D = 1.4(130\ k) = 182k$$

$$P_u = 1.2D + 1.6L = (1.2)(130\ k) + (1.6)(110\ k) = 332\ k$$

(b) computing the mininum A_g required, using LRFD equations

$$1.\ \min A_g = \frac{P_u}{\phi_t F_y} = \frac{332\ k}{(0.9)(50\ ksi)} = 7.38\ in^2$$

$$2.\ \min A_g = \frac{P_u}{\phi_t F_u U} + \text{ estimated area of holes}$$

Assume that U = 0.85 from Table 3.2, Case 7, and assume that flange thickness is about 0.380 in after looking at W12 sections in the LRFD Manual which have areas of 7.38 in^2 or more. U = 0.85 was assumed since b_f appears to be less than 2/3 d.

$$\min A_g = \frac{332\ k}{(0.75)(65\ ksi)(0.85)} + (4)\left(\frac{7}{8}\text{in} + \frac{1}{8}\text{in}\right)(0.381\ \text{in}) = 9.53\ \text{in}^2 \leftarrow$$

(c) Preferable miniumum r

$$\min r = \frac{L}{300} = \frac{(12\ in/ft)(30\ ft)}{300} = 1.2\ in$$

$$\text{Try } W12 \times 35\ (A_g = 10.3\ in^2, d = 12.5\ in, b_f = 6.56\ in.$$

$$t_f = 0.520\ in, r_{min} = r_y = 1.54\ in$$

Checking

(a) Gross section yielding

$$P_u < \phi_t P_n$$

$$P_n = F_y A_g = (50\ ksi)(10.3\ in^2) = 515\ k$$

$$\phi_t P_n = (0.9)(515\ k) = 463.5\ k > 332\ k\ [\therefore OK]$$

(b) Tensile rupture strength

From Table 3.2, case 2

$\bar{x}$ for half of $W12 \times 35$ or, that is, a $WT6 \times 17.5 = 1.30\ in$

$$L = (2)(4\ in) = 8\ in$$

$$U = \left(1 - \frac{\bar{x}}{L}\right) = \left(1 - \frac{1.3}{8}\right) = 0.84$$

From table 3.2, case 7

$U = 0.85$, since $b_f = 6.56 < \frac{2}{3}d = \left(\frac{2}{3}\right)(12.50\ in) = 8.33\ in$

$$A_n = 10.3\ in^2 - (4)\left(\frac{7}{8}in + \frac{1}{8}in\right)(0.520\ in) = 8.22\ in^2$$

$$A_e = (0.85)(8.22\ in^2) = 6.99\ in^2$$

$$P_u < \phi_t P_n$$

$$P_n = F_u A_e = (65\ ksi)(6.99\ in^2) = 454.2\ k$$

LRFD with $\phi_t = 0.75$

$$\phi_t P_n = (0.75)(454.2\ k) = 340.7\ k > 332\ k\ [\therefore OK]$$

(c) Slenderness ratio

$$\frac{L}{r_y} = \frac{(12\ in/ft)(30\ ft)}{1.54} = 234 < 300[\therefore OK]$$

$$\therefore \boldsymbol{OK\ to\ use\ W12 \times 35}$$

BUILT-UP TENSION MEMBERS

Sections D4 and J3.5 of the AISC Specification provide a set of definite rules describing how the different parts of built-up tension members are to be connected together.

RODS AND BARS

When rods and bars are used as tension members, they may be simply welded at their ends, or they may be threaded and held in place with nuts. The AISC nominal tensile design stress for threaded rods, $F_{nt} = 0.75F_u$.

The gross area of the rod area, A_D, required for a particular tensile load can be calculated as:

$$A_D \geq \frac{P_u}{\phi\ 0.75\ F_u}$$

Example 4-3

Using the AISC Specification, select a standard threaded steel rod to support a tensile working dead load of 10 k and a tensile working live load of 20 k. [$F_u = 58\ ksi$].

Solution

$$P_u = 1.4D = 1.4(10\ k) = 14\ k$$

$$P_u = 1.2D + 1.6L = (1.2)(10\ k) + (1.6)(20\ k) = 44\ k$$

$$A_D \geq \frac{P_u}{\phi\ 0.75\ F_u} = \frac{44\ k}{(0.75)(0.75)(58\ ksi)} = 1.35\ in^2$$

From AISC Tables, try $1\frac{3}{8}\ in$ steel rod [has $A_D = 1.49\ in^2$]

Checking

Gross section yielding

$$P_u < \phi_t R_n$$

$$R_n = 0.75 F_u A_D = (0.75)(58\ ksi)(1.49\ in^2) = 64.8\ k$$

$$\phi_t R_n = (0.9)(64.8\ k) = 48.6\ k > 44\ k\ [\therefore OK]$$

Summary

Design of Tension Members

1. $\min A_g = \dfrac{P_u}{\phi_t F_y}$ $\qquad \phi_t = 0.9$

2. $\min A_g = \dfrac{P_u}{\phi_t F_u U} +$ estimated area of holes $\qquad \phi_t = 0.75$

3. $\min r = \dfrac{L}{300}$

Checking

(a) Gross section yielding

$$P_u < \phi_t P_n$$

$$P_n = F_y A_g \text{ and } \phi_t = 0.9$$

(b) Tensile rupture strength

$$P_u < \phi_t P_n$$

$$P_n = F_u A_e \text{ and } \phi_t = 0.75$$

(c) Slenderness ratio

$$\frac{L}{r} < 300$$

C H A P T E R 5

Introduction to Axially Loaded Compression Members

GENERAL

There are three general modes by which axially loaded columns can fail. These are flexural buckling, local buckling, and torsional buckling. These modes of buckling are briefly defined as follows:

1. Flexural buckling (also called Euler buckling): Members are subject to flexure, or bending, when they become unstable.
2. Local buckling occurs when some part or parts of the cross section of a column are so thin that they buckle locally in compression before the other modes of buckling can occur.
3. Flexural torsional buckling: Columns fail by twisting (torsion) or by a combination of torsional and flexural buckling.

SECTIONS USED FOR COLUMNS

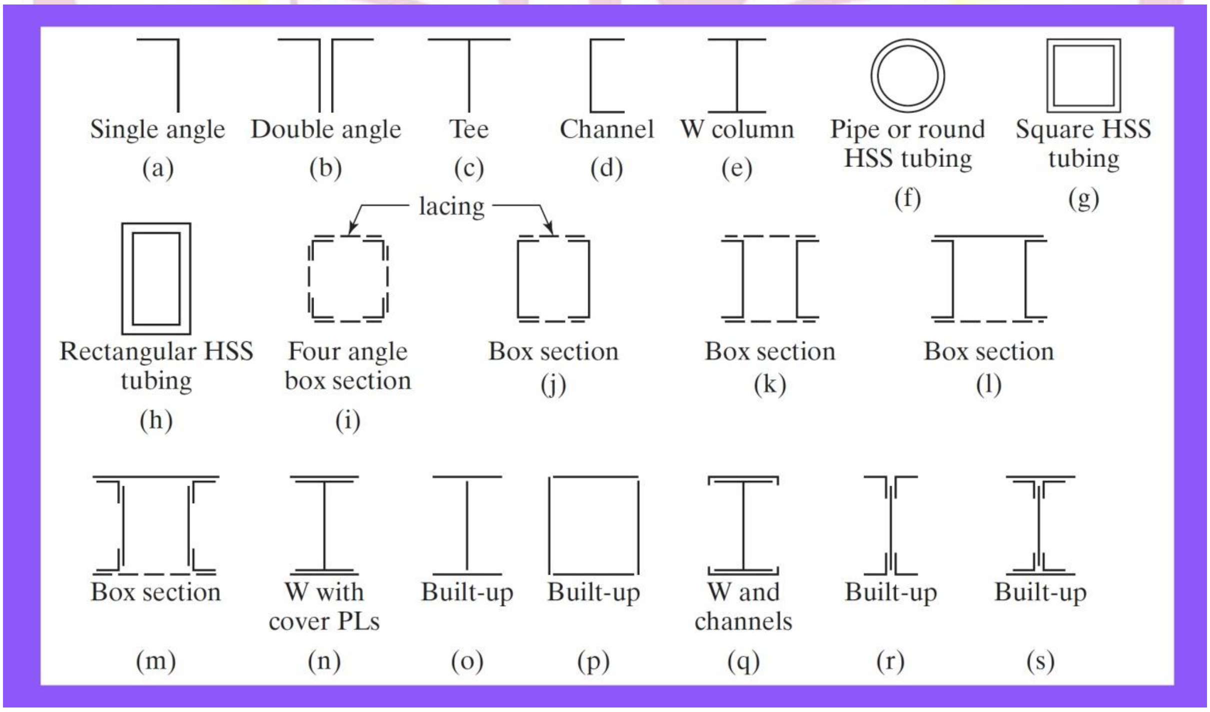


FIGURE 5.1 Types of compression members.

THE EULER FORMULA

For a column to buckle elastically, it will have to be long and slender. Its buckling load P can be computed with the Euler formula that follows:

$$\frac{P}{A} = \frac{\pi^2 E}{(L/r)^2} = F_e$$

Example 5-1 illustrates the application of the Euler formula to a steel column. If the value obtained for a particular column exceeds the steel's proportional limit, the elastic Euler formula is not applicable.

Example 5-1

(a) A W10 × 22 is used as a 15-ft long pin-connected column. Using the Euler expression, determine the column's critical or buckling load. Assume that the steel has a proportional limit of 36 ksi.

(b) Repeat part (a) if the length is changed to 8 ft.

Solution

(a) Using a 15-ft long W10 × 22 ($A = 6.49 \text{ in}^2, r_x = 4.27 \text{ in}, r_y = 1.33 \text{ in}$)

Minimum $r = r_y = 1.33$ in

$$\frac{L}{r} = \frac{(12 \text{ in/ft})(15 \text{ ft})}{1.33 \text{ in}} = 135.34$$

Elastic or buckling stress $F_e = \dfrac{(\pi^2)(29 \times 10^3 \text{ ksi})}{(135.34)^2}$

$= 15.63$ ksi $<$ the proportional limit of 36 ksi

OK column is in elastic range

Elastic or buckling load $= (15.63 \text{ ksi})(6.49 \text{ in}^2) = 101.4$ k

(b) Using an 8-ft long W10 × 22,

$$\frac{L}{r} = \frac{(12 \text{ in/ft})(8 \text{ ft})}{1.33 \text{ in}} = 72.18$$

Elastic or buckling stress $F_e = \dfrac{(\pi^2)(29 \times 10^3 \text{ ksi})}{(72.18)^2} = 54.94 \text{ ksi} > 36 \text{ ksi}$

$\therefore$ column is in inelastic range and Euler equation is not applicable.

END RESTRAINT AND EFFECTIVE LENGTHS OF COLUMNS

- End restraint and its effect on the load-carrying capacity of columns is a very important subject indeed.
- In steel specifications, the effective length of a column is referred to as KL, where K is the effective length factor. K is the number that must be multiplied by the length of the column to find its effective length.

Columns with different end conditions have entirely different effective lengths. For this initial discussion, it is assumed that no sidesway or joint translation is possible between the member ends. Sidesway or joint translation means that one or both ends of a column can move laterally with respect to each other. Should a column be connected with frictionless hinges, as shown in Fig. 5.2(a), its effective length would be equal to the actual length of the column and K would equal 1.0. If there were such a thing as a perfectly fixed-ended column, its points of inflection (or points of zero moment) would occur at its one-fourth points and its effective length would equal L / 2, as shown in Fig. 5.2(b). As a result, its K value would equal 0.50.

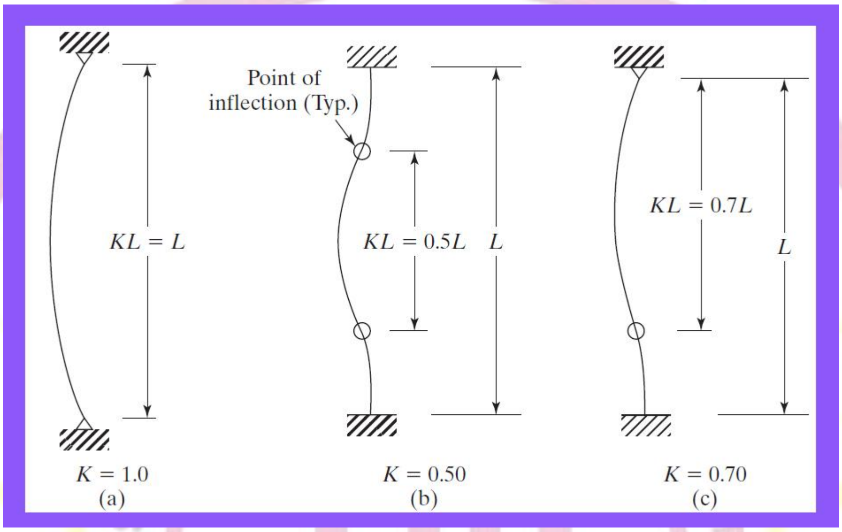
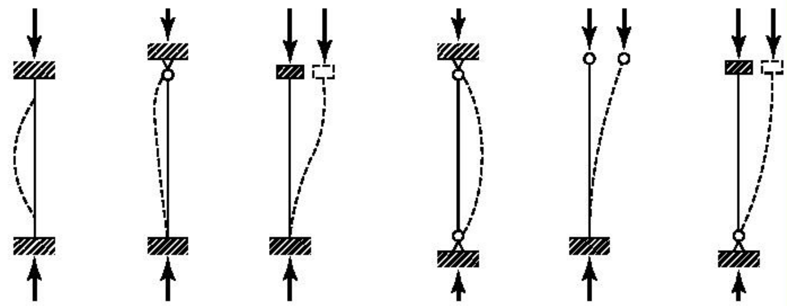


FIGURE 5.2 Effective length (KL) for columns in braced frames (sidesway prevented).

TABLE 5.1 Approximate Values of Effective Length Factor, K

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		*Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*				

LONG, SHORT, AND INTERMEDIATE COLUMNS

A column subject to an axial compression load will shorten in the direction of the load. If the load is increased until the column buckles, the shortening will stop and the column will suddenly bend or deform laterally and may at the same time twist in a direction perpendicular to its longitudinal axis.

Columns are sometimes classed as being long, short, or intermediate.A brief discussion of each of these classifications is presented in the paragraphs to follow.

1. Long Columns	The Euler formula predicts very well the strength of long columns where the axial buckling stress remains below the proportional limit. Such columns will buckle elastically.
2. Short Columns	For very short columns, the failure stress will equal the yield stress and no buckling will occur. (For a column to fall into this class, it would have to be so short as to have no practical application.Thus, no further reference is made to them here.)
3. Intermediate Columns	For intermediate columns, some of the fibers will reach the yield stress and some will not.The members will fail by both yielding and buckling, and their behavior is said to be inelastic. Most columns fall into this range.

COLUMN FORMULAS

The AISC Specification provides one equation (the Euler equation) for long columns with elastic buckling and an empirical parabolic equation for short and intermediate columns.With these equations, a flexural buckling stress, F_{cr}, is determined for a compression member. Once this stress is computed for a particular member, it is multiplied by the cross-sectional area of the member to obtain its nominal strength P_n . The LRFD design strength of a column may be determined as follows:

$$P_n = F_{cr}A_g$$

LRFD compression strength $\phi_c P_n = \phi_c F_{cr} A_g$

$[\phi_c = 0.90]$

The following expressions shsow how F_{cr}, the flexureal bucking stressof a column, may be determined

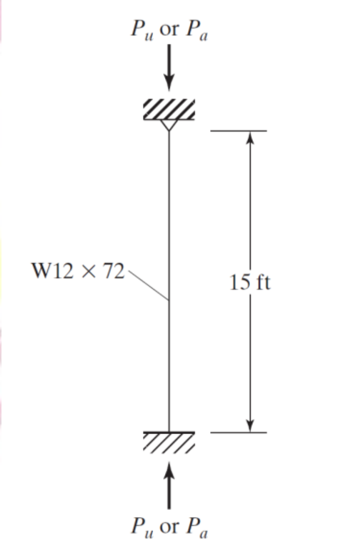
a) If $\frac{KL}{R} \leq 4.71\sqrt{\frac{E}{F_y}}$ $\quad\Rightarrow F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$

b) If $\frac{KL}{R} > 4.71\sqrt{\frac{E}{F_y}}$ $\quad\Rightarrow F_{cr} = 0.877F_e$

where $F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

Example 5-2

a. Using the column critical stress values in Table 4-22 of the Manual, determine the LRFD design strength $\phi_c P_n$ for the column shown below, if a 50-ksi steel is used.
b. Repeat the problem, using Table 4-1 of the Manual.
c. Calculate $\phi_c P_n$ using AISC equations.



Solution

From thable 5.1 $K = 0.8$

$$\left(\frac{KL}{r}\right)_x = \frac{(0.8)(12 \times 15)}{5.31} = 27.11$$

$$\left(\frac{KL}{r}\right)_y = \frac{(0.8)(12 \times 15)}{3.04} = 47.37 \leftarrow controls$$

From Table 4-22 (MANUAL), by stright line interpolation, $\phi_c F_{cr} = 38.19\ ksi$

For LRFD, $\phi_c P_n = \phi_c F_{cr} A_g = (38.19)(21.1) = 805.8\ k$

(b) from Table 4-1 (MANUAL), for $KL = (0.8)(15) = 12\ ft,\ \Rightarrow \phi_c P_n = 807\ k$

(c) Elastic critical bucking stress

$\left(\frac{KL}{r}\right)_y = 47.37$ [from part (a)]

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(47.37)^2} = 127.55\ ksi$$

$$4.71\sqrt{\frac{E}{F_y}} = \sqrt{\frac{29000\ ksi}{50\ ksi}} = 113.3 > \left(\frac{KL}{r}\right)_y = 47.37$$

$$\Rightarrow F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\left(\frac{50}{127.55}\right)}\right] 50 = 42.43\ ksi$$

LRFD method, $\phi_c F_{cr} = (0.90)(42.43) = 38.19\ ksi$

$\phi_c P_n = \phi_c F_{cr} A_g = (38.19)(21.1) = 805.8\ k$

Example 5-3

An HSS $16 \times 16 \times \frac{1}{2}$ with $F_y = 46$ ksi is used for an 18-ft-long column with simple end supports.

(a) Determine $\phi_c P_n$ with the appropriate AISC equations.

(b) Repeat part (a), using Table 4-4 in the AISC Manual.

Solutuion

(a) Using an HSS

$$16 \times 16 \times \frac{1}{2}(A = 28.3\ \text{in}^2, t_{\text{wall}} = 0.465\ \text{in}, r_x = r_y = 6.31\ \text{in})$$

$K = 1.0$

$$\left(\frac{KL}{r}\right)_x = \left(\frac{KL}{r}\right)_y = \frac{(1.0)(12 \times 18)\ \text{in}}{6.31\ \text{in}} = 34.23$$

$$< 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{46}} = 118.26$$

$\therefore$ Use AISC Equation for F_{cr}

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29{,}000)}{(34.23)^2} = 244.28 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{46}{244.28}}\right] 46$$

$$= 42.51 \text{ ksi}$$

LRFD $\phi_c = 0.90$

$$\phi_c F_{cr} = (0.90)(42.51) = 38.26 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A = (38.26)(28.3) = 1082 \text{ k}$$

(b) from Table 4-4 (MANUAL), for $KL = (1.0)(18) = 18\ ft,\ \Rightarrow \phi_c P_n = 1080\ k$

Table 4–22
Available Critical Stress for Compression Members

F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
1	**21.0**	31.5	1	**21.6**	32.4	1	**25.1**	37.8	1	**27.5**	41.4	1	**29.9**	45.0
2	**21.0**	31.5	2	**21.6**	32.4	2	**25.1**	37.8	2	**27.5**	41.4	2	**29.9**	45.0
3	**20.9**	31.5	3	**21.5**	32.4	3	**25.1**	37.8	3	**27.5**	41.4	3	**29.9**	45.0
4	**20.9**	31.5	4	**21.5**	32.4	4	**25.1**	37.8	4	**27.5**	41.4	4	**29.9**	44.9
5	**20.9**	31.5	5	**21.5**	32.4	5	**25.1**	37.7	5	**27.5**	41.3	5	**29.9**	44.9
6	**20.9**	31.4	6	**21.5**	32.3	6	**25.1**	37.7	6	**27.5**	41.3	6	**29.9**	44.9
7	**20.9**	31.4	7	**21.5**	32.3	7	**25.1**	37.7	7	**27.5**	41.3	7	**29.8**	44.8
8	**20.9**	31.4	8	**21.5**	32.3	8	**25.1**	37.7	8	**27.4**	41.2	8	**29.8**	44.8
9	**20.9**	31.4	9	**21.5**	32.3	9	**25.0**	37.6	9	**27.4**	41.2	9	**29.8**	44.7
10	**20.9**	31.3	10	**21.4**	32.2	10	**25.0**	37.6	10	**27.4**	41.1	10	**29.7**	44.7
11	**20.8**	31.3	11	**21.4**	32.2	11	**25.0**	37.5	11	**27.3**	41.1	11	**29.7**	44.6
12	**20.8**	31.3	12	**21.4**	32.2	12	**24.9**	37.5	12	**27.3**	41.0	12	**29.6**	44.5
13	**20.8**	31.2	13	**21.4**	32.1	13	**24.9**	37.4	13	**27.2**	40.9	13	**29.6**	44.4
14	**20.7**	31.2	14	**21.3**	32.1	14	**24.8**	37.3	14	**27.2**	40.9	14	**29.5**	44.4
15	**20.7**	31.1	15	**21.3**	32.0	15	**24.8**	37.3	15	**27.1**	40.8	15	**29.5**	44.3
16	**20.7**	31.1	16	**21.3**	32.0	16	**24.8**	37.2	16	**27.1**	40.7	16	**29.4**	44.2
17	**20.7**	31.0	17	**21.2**	31.9	17	**24.7**	37.1	17	**27.0**	40.6	17	**29.3**	44.1
18	**20.6**	31.0	18	**21.2**	31.9	18	**24.7**	37.1	18	**27.0**	40.5	18	**29.2**	43.9
19	**20.6**	30.9	19	**21.2**	31.8	19	**24.6**	37.0	19	**26.9**	40.4	19	**29.2**	43.8
20	**20.5**	30.9	20	**21.1**	31.7	20	**24.5**	36.9	20	**26.8**	40.3	20	**29.1**	43.7
21	**20.5**	30.8	21	**21.1**	31.7	21	**24.5**	36.8	21	**26.7**	40.2	21	**29.0**	43.6
22	**20.4**	30.7	22	**21.0**	31.6	22	**24.4**	36.7	22	**26.7**	40.1	22	**28.9**	43.4
23	**20.4**	30.7	23	**21.0**	31.5	23	**24.3**	36.6	23	**26.6**	40.0	23	**28.8**	43.3
24	**20.3**	30.6	24	**20.9**	31.4	24	**24.3**	36.5	24	**26.5**	39.8	24	**28.7**	43.1
25	**20.3**	30.5	25	**20.9**	31.4	25	**24.2**	36.4	25	**26.4**	39.7	25	**28.6**	43.0
26	**20.2**	30.4	26	**20.8**	31.3	26	**24.1**	36.3	26	**26.3**	39.6	26	**28.5**	42.8
27	**20.2**	30.3	27	**20.7**	31.2	27	**24.0**	36.1	27	**26.2**	39.4	27	**28.4**	42.7
28	**20.1**	30.3	28	**20.7**	31.1	28	**24.0**	36.0	28	**26.1**	39.3	28	**28.3**	42.5
29	**20.1**	30.2	29	**20.6**	31.0	29	**23.9**	35.9	29	**26.0**	39.1	29	**28.2**	42.3
30	**20.0**	30.1	30	**20.6**	30.9	30	**23.8**	35.8	30	**25.9**	39.0	30	**28.0**	42.1
31	**20.0**	30.0	31	**20.5**	30.8	31	**23.7**	35.6	31	**25.8**	38.8	31	**27.9**	41.9
32	**19.9**	29.9	32	**20.4**	30.7	32	**23.6**	35.5	32	**25.7**	38.6	32	**27.8**	41.8
33	**19.8**	29.8	33	**20.4**	30.6	33	**23.5**	35.4	33	**25.6**	38.5	33	**27.7**	41.6
34	**19.8**	29.7	34	**20.3**	30.5	34	**23.4**	35.2	34	**25.5**	38.3	34	**27.5**	41.4
35	**19.7**	29.6	35	**20.2**	30.4	35	**23.3**	35.1	35	**25.4**	38.1	35	**27.4**	41.2
36	**19.6**	29.5	36	**20.1**	30.3	36	**23.2**	34.9	36	**25.2**	37.9	36	**27.2**	40.9
37	**19.5**	29.4	37	**20.1**	30.1	37	**23.1**	34.8	37	**25.1**	37.8	37	**27.1**	40.7
38	**19.5**	29.3	38	**20.0**	30.0	38	**23.0**	34.6	38	**25.0**	37.6	38	**26.9**	40.5
39	**19.4**	29.1	39	**19.9**	29.9	39	**22.9**	34.4	39	**24.9**	37.4	39	**26.8**	40.3
40	**19.3**	29.0	40	**19.8**	29.8	40	**22.8**	34.3	40	**24.7**	37.2	40	**26.6**	40.0

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

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Table 4–22 (continued)
Available Critical Stress for Compression Members

	F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi	
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5
51	18.3	27.6	51	18.8	28.3	51	21.4	32.2	51	23.1	34.8	51	24.8	37.2
52	18.3	27.4	52	18.7	28.1	52	21.3	32.0	52	23.0	34.5	52	24.6	36.9
53	18.2	27.3	53	18.6	28.0	53	21.2	31.8	53	22.8	34.3	53	24.4	36.7
54	18.1	27.1	54	18.5	27.8	54	21.0	31.6	54	22.6	34.0	54	24.2	36.4
55	18.0	27.0	55	18.4	27.6	55	20.9	31.4	55	22.5	33.8	55	24.0	36.1
56	17.9	26.8	56	18.3	27.5	56	20.7	31.2	56	22.3	33.5	56	23.8	35.8
57	17.7	26.7	57	18.2	27.3	57	20.6	31.0	57	22.1	33.3	57	23.6	35.5
58	17.6	26.5	58	18.1	27.1	58	20.5	30.7	58	22.0	33.0	58	23.4	35.2
59	17.5	26.4	59	17.9	27.0	59	20.3	30.5	59	21.8	32.8	59	23.2	34.9
60	17.4	26.2	60	17.8	26.8	60	20.2	30.3	60	21.6	32.5	60	23.0	34.6
61	17.3	26.0	61	17.7	26.6	61	20.0	30.1	61	21.4	32.2	61	22.8	34.3
62	17.2	25.9	62	17.6	26.5	62	19.9	29.9	62	21.3	32.0	62	22.6	34.0
63	17.1	25.7	63	17.5	26.3	63	19.7	29.6	63	21.1	31.7	63	22.4	33.7
64	17.0	25.5	64	17.4	26.1	64	19.6	29.4	64	20.9	31.4	64	22.2	33.4
65	16.9	25.4	65	17.3	25.9	65	19.4	29.2	65	20.7	31.2	65	22.0	33.0
66	16.8	25.2	66	17.1	25.8	66	19.2	28.9	66	20.5	30.9	66	21.8	32.7
67	16.7	25.0	67	17.0	25.6	67	19.1	28.7	67	20.4	30.6	67	21.6	32.4
68	16.5	24.9	68	16.9	25.4	68	18.9	28.5	68	20.2	30.3	68	21.4	32.1
69	16.4	24.7	69	16.8	25.2	69	18.8	28.2	69	20.0	30.1	69	21.1	31.8
70	16.3	24.5	70	16.7	25.0	70	18.6	28.0	70	19.8	29.8	70	20.9	31.4
71	16.2	24.3	71	16.5	24.8	71	18.5	27.7	71	19.6	29.5	71	20.7	31.1
72	16.1	24.2	72	16.4	24.7	72	18.3	27.5	72	19.4	29.2	72	20.5	30.8
73	16.0	24.0	73	16.3	24.5	73	18.1	27.2	73	19.2	28.9	73	20.3	30.5
74	15.8	23.8	74	16.2	24.3	74	18.0	27.0	74	19.1	28.6	74	20.1	30.2
75	15.7	23.6	75	16.0	24.1	75	17.8	26.8	75	18.9	28.4	75	19.8	29.8
76	15.6	23.4	76	15.9	23.9	76	17.6	26.5	76	18.7	28.1	76	19.6	29.5
77	15.5	23.3	77	15.8	23.7	77	17.5	26.3	77	18.5	27.8	77	19.4	29.2
78	15.4	23.1	78	15.6	23.5	78	17.3	26.0	78	18.3	27.5	78	19.2	28.8
79	15.2	22.9	79	15.5	23.3	79	17.1	25.8	79	18.1	27.2	79	19.0	28.5
80	15.1	22.7	80	15.4	23.1	80	17.0	25.5	80	17.9	26.9	80	18.8	28.2

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

W12

Shape	W12×									
Wt/ft	96		87		79		72		65	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y										
0	844	1270	766	1150	694	1040	633	951	571	859
6	811	1220	735	1110	667	1000	607	913	548	824
7	800	1200	725	1090	657	987	598	899	540	811
8	787	1180	713	1070	646	971	588	884	531	798
9	772	1160	699	1050	634	952	577	867	520	782
10	756	1140	685	1030	620	932	565	849	509	765
11	739	1110	669	1010	606	910	551	828	497	747
12	720	1080	652	980	590	887	537	807	484	727
13	701	1050	634	953	573	862	522	784	470	706
14	680	1020	615	924	556	836	506	761	456	685
15	659	990	595	895	538	809	490	736	441	662
16	637	957	575	864	520	781	473	710	425	639
17	614	923	554	833	501	752	455	684	409	615
18	591	888	533	801	481	723	437	657	393	591
19	567	852	511	769	461	694	419	630	377	566
20	543	816	490	736	442	664	401	603	360	541
22	495	744	446	670	402	603	365	548	327	491
24	447	672	402	605	362	544	328	493	294	442
26	401	602	360	541	323	486	293	440	262	393
28	356	534	319	479	286	430	259	389	231	347
30	312	469	279	420	250	376	226	340	202	303
32	274	412	246	369	220	331	199	299	177	267
34	243	365	218	327	195	293	176	265	157	236
36	217	326	194	292	174	261	157	236	140	211
38	195	292	174	262	156	234	141	212	126	189
40	176	264	157	236	141	212	127	191	114	171
Properties										
P_{wo} (kips)	137	206	121	181	104	157	90.9	136	78.2	117
P_{wi} (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb} (kips)	296	445	243	366	185	278	142	213	106	159
P_{fb} (kips)	152	228	123	185	101	152	84.0	126	68.5	103
L_p (ft)	10.9		10.8		10.8		10.7		11.9	
L_r (ft)	46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})	28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})	833		740		662		597		533	
I_y (in.^{4})	270		241		216		195		174	
r_y (in.)	3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

$F_y = 50$ ksi

Shape	W12×									
Wt/ft	58		53		50		45		40	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y										
0	510	767	466	701	437	657	393	590	350	526
6	481	722	438	659	396	595	356	534	316	475
7	470	707	429	644	382	574	343	516	305	458
8	459	689	418	628	367	551	329	495	292	439
9	446	670	406	610	350	526	314	472	279	419
10	432	649	393	590	332	499	298	448	264	397
11	417	627	379	569	314	471	281	422	249	375
12	401	603	364	547	294	443	264	396	234	351
13	385	578	349	525	275	413	246	370	218	328
14	368	553	333	501	255	384	228	343	202	304
15	350	527	317	477	236	354	211	317	186	280
16	333	500	301	452	217	326	193	291	171	257
17	315	473	284	427	198	297	176	265	156	234
18	297	446	268	402	180	270	160	241	141	212
19	279	420	251	378	162	244	144	217	127	191
20	262	393	235	353	146	220	130	196	115	172
22	227	342	204	306	121	182	108	162	94.8	142
24	195	293	174	261	102	153	90.4	136	79.6	120
26	166	249	148	222	86.6	130	77.0	116	67.9	102
28	143	215	127	192	74.6	112	66.4	99.8	58.5	88.0
30	125	187	111	167	65.0	97.7	57.9	87.0	51.0	76.6
32	109	165	97.6	147	57.1	85.9	50.9	76.4	44.8	67.3
34	97.0	146	86.5	130						
36	86.5	130	77.1	116						
38	77.6	117	69.2	104						
40	70.1	105	62.5	93.9						
Properties										
P_{wo} (kips)	74.4	112	67.6	101	70.3	105	60.0	90.0	49.9	74.9
P_{wi} (kips/in.)	12.0	18.0	11.5	17.3	12.3	18.5	11.2	16.8	9.83	14.8
P_{wb} (kips)	83.2	125	73.2	110	88.5	133	65.7	98.7	44.8	67.4
P_{fb} (kips)	76.6	115	61.9	93.0	76.6	115	61.9	93.0	49.6	74.6
L_p (ft)	8.87		8.76		6.92		6.89		6.85	
L_r (ft)	29.9		28.2		23.9		22.4		21.1	
A_g (in.^{2})	17.0		15.6		14.6		13.1		11.7	
I_x (in.^{4})	475		425		391		348		307	
I_y (in.^{4})	107		95.8		56.3		50.0		44.1	
r_y (in.)	2.51		2.48		1.96		1.95		1.94	
Ratio r_x/r_y	2.10		2.11		2.64		2.64		2.64	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	13600		12200		11200		9960		8790	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	3060		2740		1610		1430		1260	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

CHAPTER 6

Design of Axially Loaded Compression Members

INTRODUCTION

The design of columns by formulas involves a trial-and-error process.The LRFD design stress $\phi_c F_{cr}$ is not known until a column size is selected, and vice versa.

There are two method for design of compression memebers

1- Trial and error method (Ilustrated in Example 6-1)
2- AISC Tables method (Illustrated in Example 6-2)

Example 6-1 (Tiral and error method)

Using $F_y = 50\ ksi$, select the lightest W14 available for the service column loads $P_D = 130\ k$ and $P_L = 210\ k$, $KL = 10\ ft$.

Solution

$P_u = 1.2(130) + 1.6(210) = 492\ k$

Assume $\frac{KL}{r} = 50$

From table 4-22, for $F_y = 50\ ksi$ and $\frac{KL}{r} = 50$, we get $\phi_c F_{cr} = 37.5\ ksi$

Now caclulate required area of section,

$$\because P_u = \phi_c F_{cr} A \Rightarrow A = \frac{P_u}{\phi_c F_{cr}}$$

$$A = \frac{492\ k}{37.5\ ksi} = 13.12\ in^2$$

Try $W14 \times 48$ $(A = 14.1\ in^2, r_x = 5.85\ in, r_7 = 1.91\ in)$

Check :

$$\frac{KL}{r} = \frac{(12)(10)}{1.91} = 62.63$$

From table 4-22, for $F_y = 50\ ksi$ and $\frac{KL}{r} = 62.63$, we get $\phi_c F_{cr} = 33.75\ ksi$

$$\phi_c P_n = \phi_c F_{cr} A_g = (33.75)(14.1) = 476\ k < 492\ \therefore NOT\ GOOD$$

Try next larger section, $W14 \times 53\ (A = 15.6\ in^2, r_x = 5.89\ in, r_y = 1.92\ in)$

Check :

$$\frac{KL}{r} = \frac{(12)(10)}{1.92} = 62.5$$

From table 4-22, for $F_y = 50\ ksi$ and $\frac{KL}{r} = 62.5$, we get $\phi_c F_{cr} = 33.85\ ksi$

$$\phi_c P_n = \phi_c F_{cr} A_g = (33.85)(15.6) = 528\ k > 492\ \therefore OK$$

(1) **Table 4–22 (continued)**

Available Critical Stress for Compression Members

(2)

$F_y = 35ksi$			$F_y = 36ksi$			$F_y = 42ksi$			$F_y = 46ksi$			$F_y = 50ksi$		
Kl/r	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	Kl/r	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	Kl/r	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	Kl/r	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	Kl/r	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5
51	18.3	27.6	51	18.8	28.3	51	21.4	32.2	51	23.1	34.8	51	24.8	37.2
52	18.3	27.4	52	18.7	28.1	52	21.3	32.0	52	23.0	34.5	52	24.6	36.9

(3)

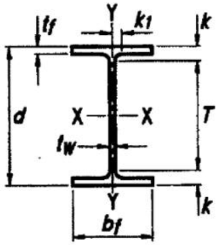


Table 1-1 (continued)
(1) W Shapes
Dimensions

(2) Shape	(3) Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k, k_{des}	Distance k, k_{det}	Distance k_1	Distance T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2	↓	↓
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2	↓	↓
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16	↓	↓
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16	↓	↓
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16	↓	↓
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16	↓	↓
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1	↓	↓
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8 (4)	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1	↓	↓
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1	↓	↓
W14×38[c]	11.2	14.1	14 1/8	0.310	5/16	3/16	6.77	6 3/4	0.515	1/2	0.915	1 1/4	13/16	11 5/8	3 1/2[g]
×34[c]	10.0	14.0	14	0.285	5/16	3/16	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3/4	↓	3 1/2
×30[c]	8.85	13.8	13 7/8	0.270	1/4	1/8	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3/4	↓	3 1/2
W14×26[c]	7.69	13.9	13 7/8	0.255	1/4	1/8	5.03	5	0.420	7/16	0.820	1 1/8	3/4	11 5/8	2 3/4[g]
×22[c]	6.49	13.7	13 3/4	0.230	1/4	1/8	5.00	5	0.335	5/16	0.735	1 1/16	3/4	11 5/8	2 3/4[g]
W12×336[h]	98.8	16.8	16 7/8	1.78	1 3/4	7/8	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	1 11/16	9 1/8	5 1/2
×305[h]	89.6	16.3	16 3/8	1.63	1 5/8	13/16	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	1 5/8		
×279[h]	81.9	15.9	15 7/8	1.53	1 1/2	3/4	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	1 5/8		
×252[h]	74.0	15.4	15 3/8	1.40	1 3/8	11/16	13.0	13	2.25	2 1/4	2.85	3 1/8	1 1/2		

Table 1-1 (continued)
W Shapes
Properties

W14 – W12

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.6	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.5	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.5	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.11	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.5	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.82	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.2	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.3	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.1	1.05	1950
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.5	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.31	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.9	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800

Example 6-2 (AISC Tables method)

If you know that $P_D = 130\ k$ and $P_L = 210\ k$, $KL = 10\ ft$, Use the AISC column tables (LRFD method) for the designs to follow.

a) Select the lightest W section available for the loads, steel, $F_y = 50\ ksi$
b) Select the lightest satisfactory rectangular HSS sections, $F_y = 46\ ksi$
c) Select the lightest satisfactory square HSS sections, $F_y = 46\ ksi$
d) Select the lightest satisfactory round HSS section, $F_y = 42\ ksi$.

Solution

$P_u = 1.2(130) + 1.6(210) = 492\ k$

(a)
From Table 4-1, for $KL = 10\ ft$ and $F_y = 50\ ksi$
For $\boldsymbol{W8 \times 48}$ we get $\phi_c P_n = 497\ ksi > 492$[OK]

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes (1)

(2) $F_y = 50$ ksi

W8

Shape		W8×											
Wt/ft		67		58		48		40		35		31	
Design (3)		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
…east radius of gyration r_y	0	589	886	512	769	422	634	351	528	308	463	273	410
	6	542	814	469	706	387	581	321	482	281	422	249	374
	7	525	790	455	684	375	563	310	467	272	408	241	362
	8	507	762	439	660	361	543	299	449	261	393	232	348
	9	487	733	422	634	347	521	286	430	250	376	222	333
(4)	10	466	701	403	606	331	497	273	410	238	358	211	317
	11	444	667	383	576	314	473	259	389	226	340	200	300
	12	421	632	363	545	297	447	244	367	213	320	188	283
	13	397	596	342	514	280	420	229	344	200	300	177	265

(b) From Table 4-3, for $KL = 10\ ft$ and $F_y = 46\ ksi$

For $\boldsymbol{HSS}\ \mathbf{12 \times 8 \times 3/8}$ we get $\phi_c P_n = 498\ ksi > 492$[OK]

Table 4–3 (continued)

Available Strength in Axial Compression, kips

HSS12 (1) **Rectangular HSS** (2) F_y = 46 ksi

Shape		HSS12×8×								HSS12×6×			
		3/8		5/16[c]		1/4[c]		3/16[c]		5/8		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
Wt/ft		47.8		40.4		32.6		24.8		67.6		55.5	
Design (3)		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	0	362	545	296	444	218	327	136	204	515	774	422	634
	6	351	527	288	433	213	320	134	201	484	728	398	598
	7	347	521	286	429	211	317	133	200	474	712	390	586
(4)	8	342	514	283	425	209	314	132	199	462	694	380	571
	9	337	506	279	420	207	311	131	198	449	674	370	556
	10	331	498	275	414	204	307	130	196	435	653	359	539
...yration r_y	11	325	488	271	408	202	303	129	194	419	630	346	521
	12	318	478	267	401	199	299	128	192	403	606	334	502

(c) From Table 4-4, for $KL = 10\ ft$ and $F_y = 46\ ksi$

For $\boldsymbol{HSS}\ \mathbf{10 \times 10 \times 3/8}$ we get $\phi_c P_n = 511\ ksi > 492$[OK]

Table 4–4 (continued)

Available Strength in Axial Compression, kips

HSS12–HSS10 (1) **Square HSS** (2) F_y = 46 ksi

Shape		HSS12×12×		HSS10×10×									
		3/16[c]		5/8		1/2		3/8		5/16		1/4[c]	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Wt/ft		29.8		76.1		62.3		47.8		40.3		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
(3)	0	142	214	579	870	473	711	362	545	305	459	228	342
	6	141	212	565	850	462	695	354	533	299	449	224	337
	7	141	212	560	842	458	689	351	528	296	445	223	336
(4)	8	141	211	555	834	454	682	348	523	293	441	222	334
	9	140	211	548	824	449	675	344	518	290	436	221	331
	10	140	210	542	814	443	666	340	511	287	431	219	329
...ration r_y	11	139	209	534	803	437	657	336	505	283	426	217	326
	12	139	209	526	790	431	648	331	497	279	419	215	323

(d) From Table 4-5, for $KL = 10\ ft$ and $F_y = 46\ ksi$
For $\boldsymbol{HSS\ 16.000 \times 0.312}$ we get $\phi_c P_n = 528\ ksi > 492$[OK]

Table 4–5 (continued)
Available Strength in Axial Compression, kips

$F_y = 42$ ksi (2)

(1) **Round HSS**

HSS16.000–HSS14.000

Shape		HSS16.000×				HSS14.000×							
		0.312		0.250		0.625		0.500		0.375		0.312	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.291	
Wt/ft		52.3		42.1		89.4		72.2		54.6		45.7	
Design (3)		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
...ration r_y (4)	0	361	543	290	436	616	926	497	747	376	566	315	474
	6	357	537	287	432	607	913	490	737	371	558	311	467
	7	356	535	286	430	604	908	488	733	369	555	309	465
	8	355	533	285	428	601	903	485	729	367	552	308	462
	9	353	530	284	426	597	897	482	724	365	549	306	460
	10	351	528	282	424	592	890	478	719	362	545	304	456
	11	349	524	280	421	587	883	475	713	360	540	301	453
	12	347	521	279	419	582	875	470	707	356	536	299	449

CHAPTER 7

Design of Axially Loaded Compression Members (Continued)

INTRODUCTION

- In this chapter, the available axial strengths of columns used in unbraced **steel frames** are considered.
- In this chapter, the available strength of compression members, ϕP_n, will be determined in building frames calculating KL using the Effective Length Method.

Effective Length of a column in steel frames:

- The true effective length of a column is a property of the whole structure, of which the column is a part.
- Theoretical mathematical analyses may be used to determine effective lengths, but such procedures are too lengthy and too difficult.
- The most common method for obtaining effective lengths is to employ the charts shown in Fig. 7.1.

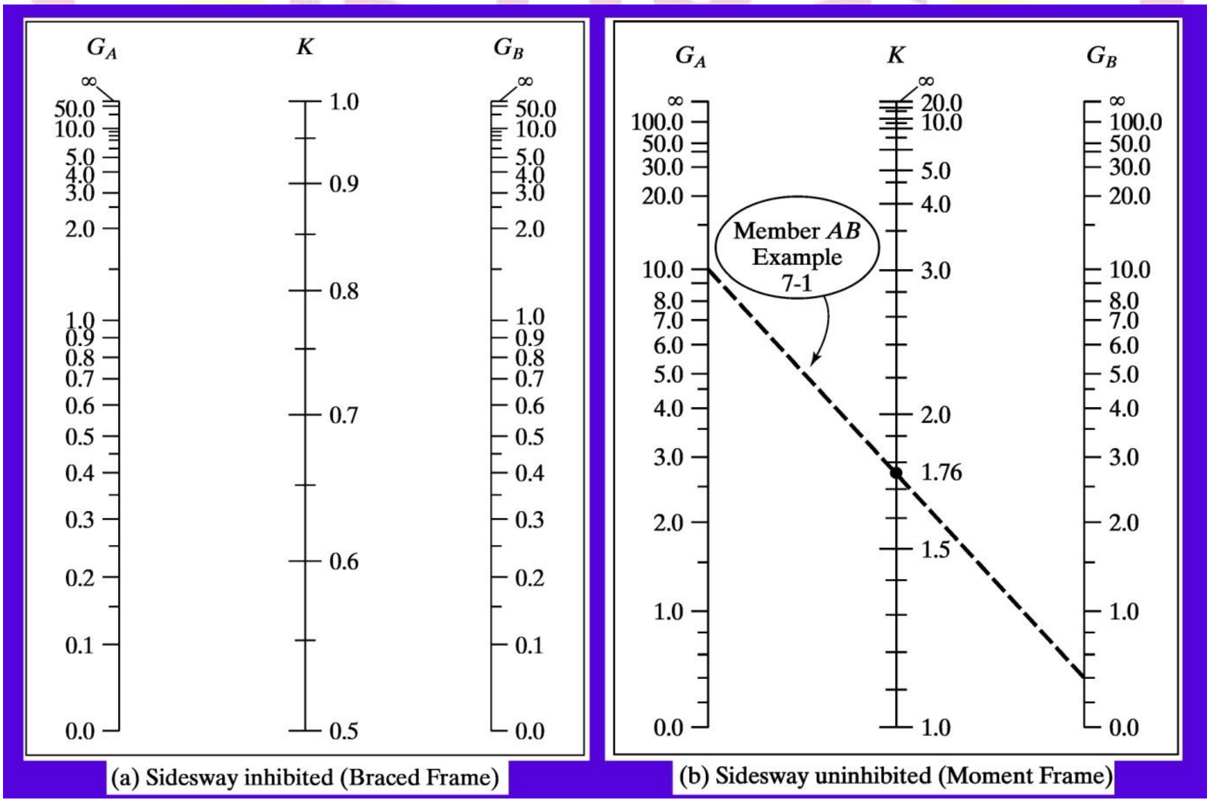


Figure 7.1 alignment charts for effective lengths of columns in continous frames.

When we say sidesway is inhibited, we mean there is something present other than just columns and girders to prevent sidesway or the horizontal translation of the joints.That means we have a definite system of lateral bracing, or we have shear walls, see Figure 7.2. If we say that sidesway is uninhibited, we are saying that resistance to horizontal translation is supplied only by the bending strength and stiffness of the girders and beams of the frame in question, with its continuous joints.

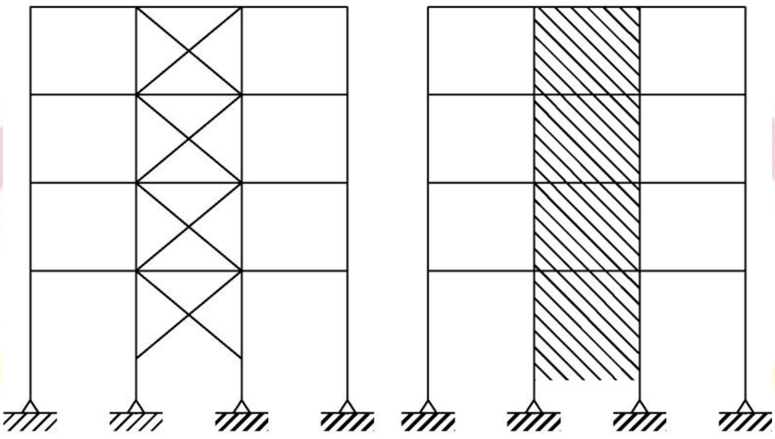


Figure 7.2 Sidesway inhibited.

The rotational restraint at the end of a particular column is proportional to the ratio of the sum of the column stiffnesses to the girder stiffnesses meeting at that joint, or

$$G = \frac{\sum \frac{EI}{L}\ for\ columns}{\sum \frac{EI}{L}\ for\ girders} == \frac{\sum \frac{E_c I_c}{L_c}}{\sum \frac{E_g I_g}{L_g}}$$

where

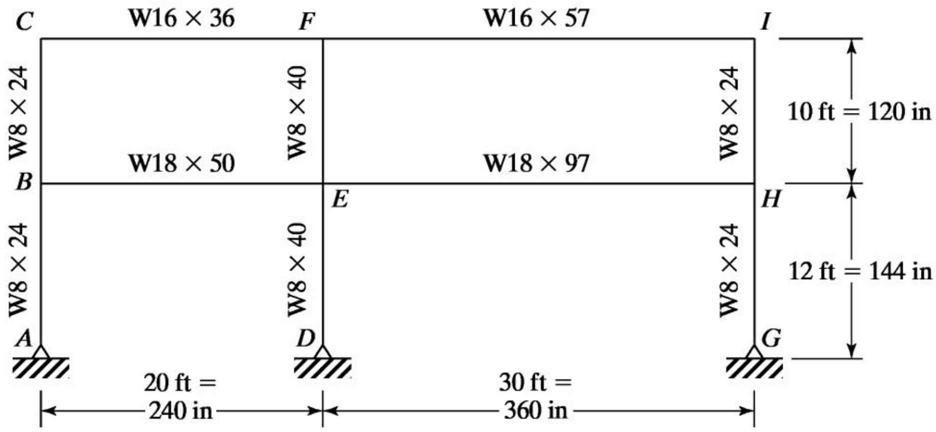
E_c	Modulus of Elasticity of column
E_g	Modulus of Elasticity of girder
I_c	Moment of Inertia of column
I_g	Moment of Inertia of girder
L_c	Length column
L_g	Legnth of girder

The recommended values of G factors at the column bases are:

1. For pinned columns,G is theoretically infinite, such as when a column is connected to a footing with a frictionless hinge. Since such a connection is not frictionless, it is recommended that G be made equal to 10.
2. For rigid connections of columns to footings,G theoretically approaches zero, but from a practical standpoint, a value of 1.0 is recommended, because no connections are perfectly rigid.

Example 7-1

Determine the effective length factor for each of the columns of the frame shown in Fig. 7.3 if the frame is not braced against sidesway.



Solution. Stiffness factors: E is assumed to be 29,000 ksi for all members and is therefore neglected in the equation to calculate G.

	Member	Shape	I	L	I/L
Columns	*AB*	W8 × 24	82.7	144	0.574
	BC	W8 × 24	82.7	120	0.689
	DE	W8 × 40	146	144	1.014
	EF	W8 × 40	146	120	1.217
	GH	W8 × 24	82.7	144	0.574
	HI	W8 × 24	82.7	120	0.689
Girders	*BE*	W18 × 50	800	240	3.333
	CF	W16 × 36	448	240	1.867
	EH	W18 × 97	1750	360	4.861
	FI	W16 × 57	758	360	2.106

G factors for each joint:

Joint	$\Sigma(I_c/L_c)/\Sigma(I_g/L_g)$	G
A	Pinned Column, $G = 10$	10.0
B	$\frac{0.574 + 0.689}{3.333}$	0.379
C	$\frac{0.689}{1.867}$	0.369
D	Pinned Column, $G = 10$	10.0
E	$\frac{1.014 + 1.217}{(3.333 + 4.861)}$	0.272
F	$\frac{1.217}{(1.867 + 2.106)}$	0.306
G	Pinned Column, $G = 10$	10.0
H	$\frac{0.574 + 0.689}{4.861}$	0.260
I	$\frac{0.689}{2.106}$	0.327

Column K factors from chart [Fig. 7.1 b]

Column	G_A	G_B	K*
AB	10.0	0.379	1.76
BC	0.379	0.369	1.12
DE	10.0	0.272	1.74
EF	0.272	0.306	1.10
GH	10.0	0.260	1.73
HI	0.260	0.327	1.10

*It is a little difficult to read the charts to the three decimal places shown by the author. He has used a larger copy of Fig. 7.2 for his work. For all practical design purposes, the K values can be read to two places, which can easily be accomplished with this figure.

C H A P T E R 8

Introduction to Beams

INTRODUCTION

Beams are usually said to be members that support transverse loads.They are probably thought of as being used in horizontal positions and subjected to gravity or vertical loads, but there are frequent exceptions—roof rafters, for example.

SECTIONS USED AS BEAMS

The W shapes will normally prove to be the most economical beam section, and they have largely replaced channels and S sections for beam usage.

DETERMINING THE ALLOWABLE BENDING STRESS

Bending stress, f_b, in a beam is determined by the flexure formula

$$f_b = \frac{M}{S}$$

Where M is bending moment.

S is section modulus

Example 8.1 calculate the maxium bending stress, f_b due to a 170 ft.k mombet about the strong axis on:

a) $W12 \times 65$ section
b) $W14 \times 61$ section

Solution

a) for a $W12 \times 65$ section, $S_x = 87.9\ in^3$

$$f_{b,x} = \frac{170\ ft.k\ \times 12\ in./ft}{87.9\ in^3} = 23.2\ ksi$$

b) for a $W14 \times 61$ section, $S_x = 92.1\ in^3$

$$f_{b,x} = \frac{170\ ft.k\ \times 12\ in./ft}{92.1\ in^3} = 22.1\ ksi$$

Example 8.2 Determine the bending stress on a $W12 \times 79$ subjected to a moment of 80 ft.k about (a) the strong axis (b) the weak axis.

Soltuion

for a $W12 \times 79$ section, $S_x = 107\ in^3$, $S_y = 35.8\ in^3$

a)

$$f_{b,x} = \frac{80\ ft.k\ \times 12\ in./ft}{107\ in^3} = 8.97\ ksi$$

b)

$$f_{b,y} = \frac{80\ ft.k\ \times 12\ in./ft}{35.8\ in^3} = 26.8\ ksi$$

Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2	↓	↓
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2	↓	↓
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16	↓	↓
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16	↓	↓
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16	↓	↓
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16	↓	↓
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1	↓	↓
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1	↓	↓
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1	↓	↓
W14×38[c]	11.2	14.1	14 1/8	0.310	5/16	3/16	6.77	6 3/4	0.515	1/2	0.915	1 1/4	13/16	11 5/8	3 1/2[g]
×34[c]	10.0	14.0	14	0.285	5/16	3/16	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3/4	↓	3 1/2
×30[c]	8.85	13.8	13 7/8	0.270	1/4	1/8	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3/4	↓	3 1/2
W14×26[c]	7.69	13.9	13 7/8	0.255	1/4	1/8	5.03	5	0.420	7/16	0.820	1 1/8	3/4	11 5/8	2 3/4[g]
×22[c]	6.49	13.7	13 3/4	0.230	1/4	1/8	5.00	5	0.335	5/16	0.735	1 1/16	3/4	11 5/8	2 3/4[g]
W12×336[h]	98.8	16.8	16 7/8	1.78	1 3/4	7/8	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	1 11/16	9 1/8	5 1/2
×305[h]	89.6	16.3	16 3/8	1.63	1 5/8	13/16	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	1 5/8	↓	↓
×279[h]	81.9	15.9	15 7/8	1.53	1 1/2	3/4	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	1 5/8	↓	↓
×252[h]	74.0	15.4	15 3/8	1.40	1 3/8	11/16	13.0	13	2.25	2 1/4	2.85	3 1/8	1 1/2	↓	↓
×230[h]	67.7	15.1	15	1.29	1 5/16	11/16	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	1 1/2	↓	↓
×210	61.8	14.7	14 3/4	1.18	1 3/16	5/8	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	1 7/16	↓	↓
×190	55.8	14.4	14 3/8	1.06	1 1/16	9/16	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	1 3/8	↓	↓
×170	50.0	14.0	14	0.960	15/16	1/2	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	1 5/16	↓	↓
×152	44.7	13.7	13 3/4	0.870	7/8	7/16	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	1 1/4	↓	↓
×136	39.9	13.4	13 3/8	0.790	13/16	7/16	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	1 1/4	↓	↓
×120	35.3	13.1	13 1/8	0.710	11/16	3/8	12.3	12 3/8	1.11	1 1/8	1.70	2	1 3/16	↓	↓
×106	31.2	12.9	12 7/8	0.610	5/8	5/16	12.2	12 1/4	0.990	1	1.59	1 7/8	1 1/8	↓	↓
×96	28.2	12.7	12 3/4	0.550	9/16	5/16	12.2	12 1/8	0.900	7/8	1.50	1 13/16	1 1/8	↓	↓
×87	25.6	12.5	12 1/2	0.515	1/2	1/4	12.1	12 1/8	0.810	13/16	1.41	1 11/16	[illegible]	↓	↓
×79	23.2	12.4	12 3/8	0.470	1/2	1/4	12.1	12 1/8	0.735	3/4	1.33	1 5/8	1 1/16	↓	↓
×72	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	↓	↓
×65[f]	19.1	12.1	12 1/8	0.390	3/8	3/16	12.0	12	0.605	5/8	1.20	1 1/2	1	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Table 1-1 (continued)
W Shapes
Properties

W14 – W12

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.6	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.5	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.5	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.11	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.5	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.82	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.2	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.3	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.1	1.05	1950
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.5	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.31	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.9	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200
210	3.37	8.23	2140	292	5.89	348	664	104	3.28	159	3.82	12.8	64.7	27200
190	3.65	9.16	1890	263	5.82	311	589	93.0	3.25	143	3.76	12.6	48.8	23600
170	4.03	10.1	1650	235	5.74	275	517	82.3	3.22	126	3.71	12.5	35.6	20100
152	4.46	11.2	1430	209	5.66	243	454	72.8	3.19	111	3.66	12.3	25.8	17200
136	4.96	12.3	1240	186	5.58	214	398	64.2	3.16	98.0	3.61	12.2	18.5	14700
120	5.57	13.7	1070	163	5.51	186	345	56.0	3.13	85.4	3.56	12.0	12.9	12400
106	6.17	15.9	933	145	5.47	164	301	49.3	3.11	75.1	3.52	11.9	9.13	10700
96	6.76	17.7	833	131	5.44	147	270	44.4	3.09	67.5	3.49	11.8	6.85	9410
87	7.48	[illegible]	740	118	[illegible]	[illegible]	241	[illegible]	[illegible]	[illegible]	[illegible]	11.7	[illegible]	[illegible]
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.6	3.84	7330
72	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780