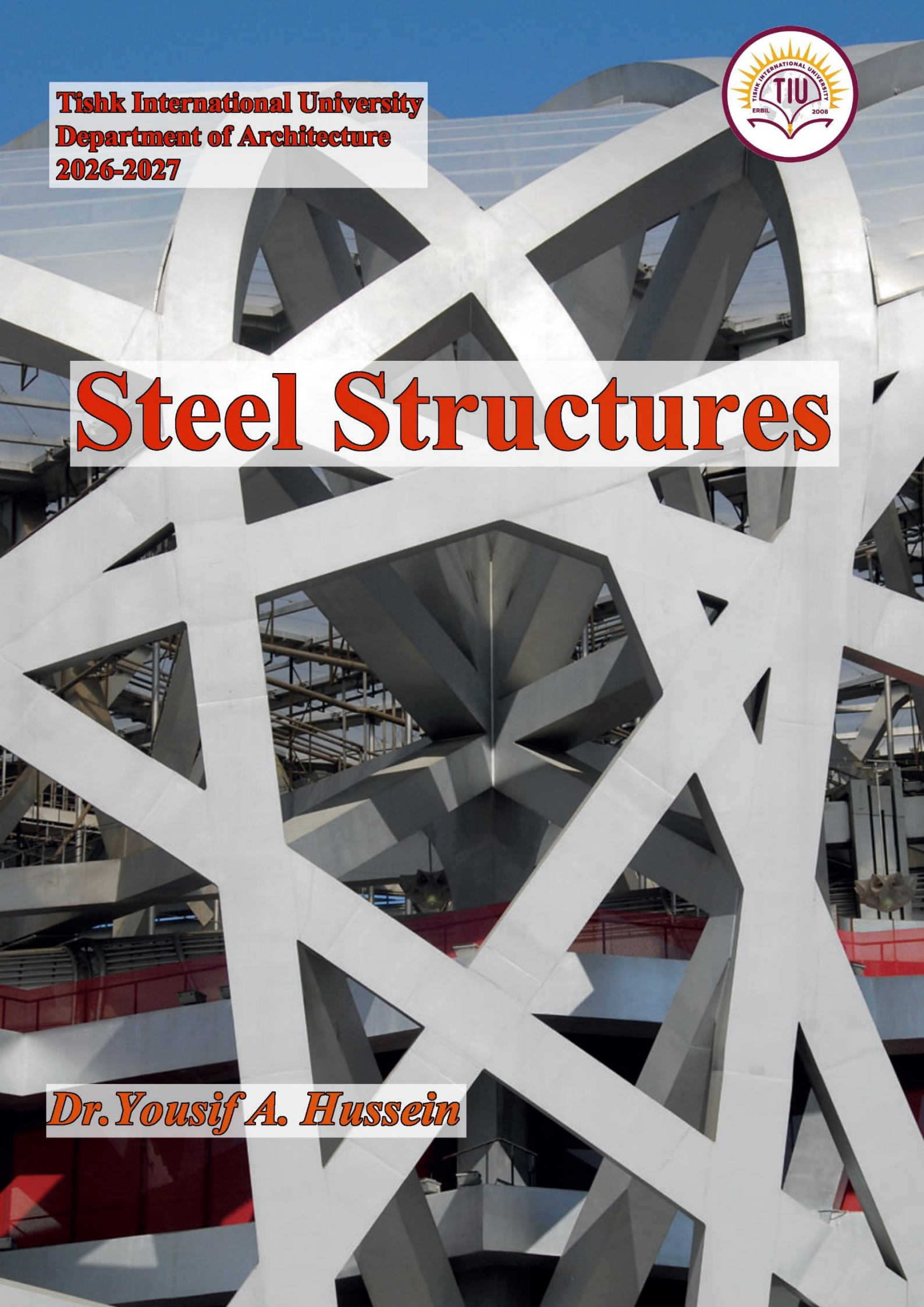

Tishk International University
Department of Architecture
2026-2027

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 Introduction to Beams
Chapter 08 Case Study: Addition To the Royal Ontario Museum (Rom)
Chapter 09 Architecturally Exposed Structural Steel (AESS)

References:

Jack C . McCormac and Stephenf. Csernak (2012) "Structural Steel Design", Prentice Hall.
American Institute of Steel Construction (2005) "Steel construction Manual".
Alan Williams (2011) "Steel Structures Design: ASD/LRFD", McGraw Hill.

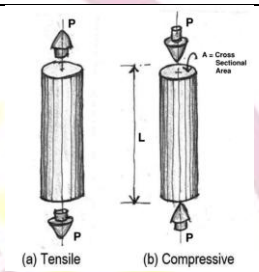
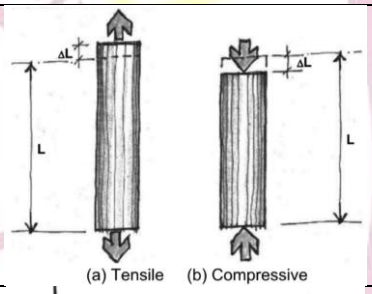
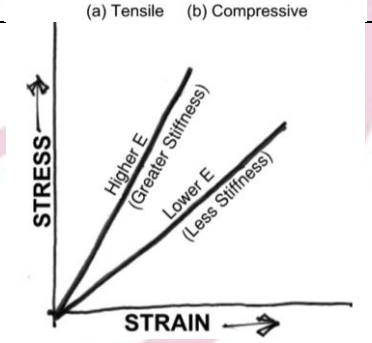
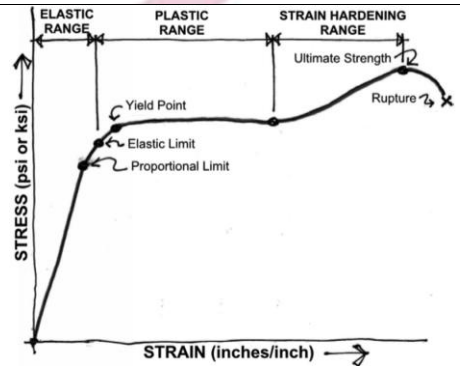
IMPORTANT: Students must bring these lecture notes to every lecture. The notes will be used directly during class discussions, explanations, and problem-solving activities.

CHAPTER 1
Introduction to Structural Steel Design

INTRODUCTION

Steel structures have been widely used in bridges, buildings, towers, and other structures around the world. Early examples include the **Rand McNally Building in Chicago**, the first steel-framed building in the United States (1890), the **Royal Insurance Building in Liverpool**, one of the first steel-framed buildings in the United Kingdom (1896–1903), and the **Eiffel Tower**, constructed in 1889 with a height of 985 ft (300 m).

STRESS, STRAIN AND MATERIALS BEHAVIOR

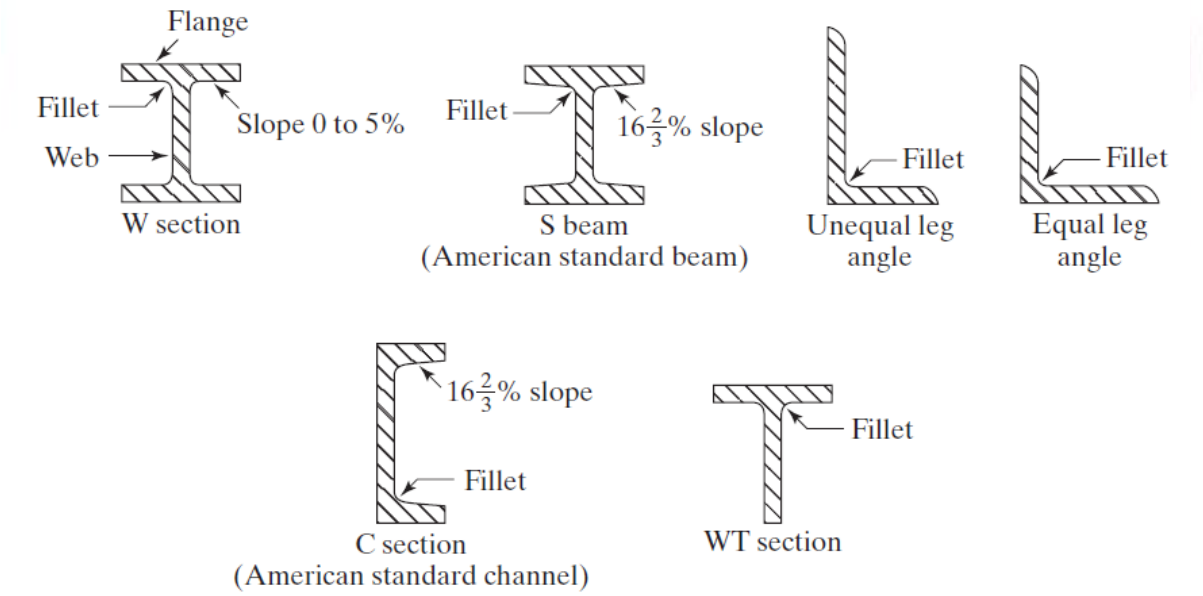
Stress	If we apply an axial tensile or compressive force (P) on a material having a length (L) and a cross-sectional area (A), the uniform stress (F) in the material in tension or compression is given by: F = P/A	
Strain	Under the applied load, the material will deform— i.e., it will elongate under tension, or shorten under compression (ΔL). The amount of deformation divided by the original length is known as strain and is given by: STRAIN = ΔL / L	
MATERIAL STIFFNESS	Material stiffness is a measure of a material's resistance to deformation. It is represented by the modulus of elasticity (E), also known as Young's modulus.	
<u>Modulus of elasticity</u>	is defined as the ratio of stress to strain within the proportional limit of a material's stress-strain curve, and is a linear relationship. E = STRESS / STRAIN	
Yield Strength (Fy)	The stress at which steel starts to permanently deform. Below Fy, the steel returns approximately to its original shape when the load is removed.	 **Steel's stress-strain curve in tension**
Ultimate Tensile Strength (Fu):	The maximum tensile stress that steel can withstand before it begins to fail (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.



L-section (angle)

S-section

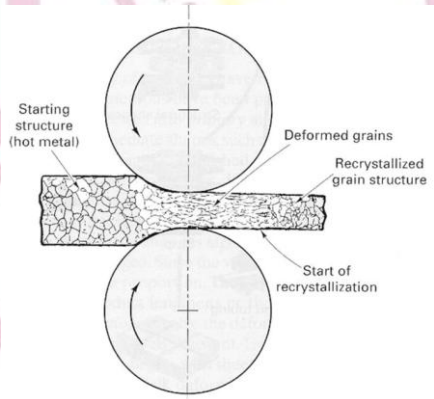
rectangular HSS section

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	C10 × 30	is a channel 10 inch deep, weighing 30 lb/ft.
5	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.
6	L6 × 6 × 1/2	is an equal leg angle, each leg being 6 inch long and 1/2 inch thick.
7	WT18 × 151	is a tee obtained by splitting a W36 X 302 This type of section is known as a structural tee.
8	___	Rectangular steel sections are classified as wide plates or narrow bars.

HOT ROLLED STRUCTURAL STEEL SECTIONS

Hot rolled sections are produced by passing steel billets at very high temperatures through a succession of rollers that progressively squeeze the billets into the desired configuration (Figure 5.8).



Hot Rolling Steel

Common hot rolled shapes are wide flange (W), channel (C), and angle (L), see the figure below.

COLD FORMED STRUCTURAL STEEL SECTIONS

Cold formed sections are produced by a series of shaping operations at relatively low temperatures that bend sheets of steel of specified thickness into the desired configuration.

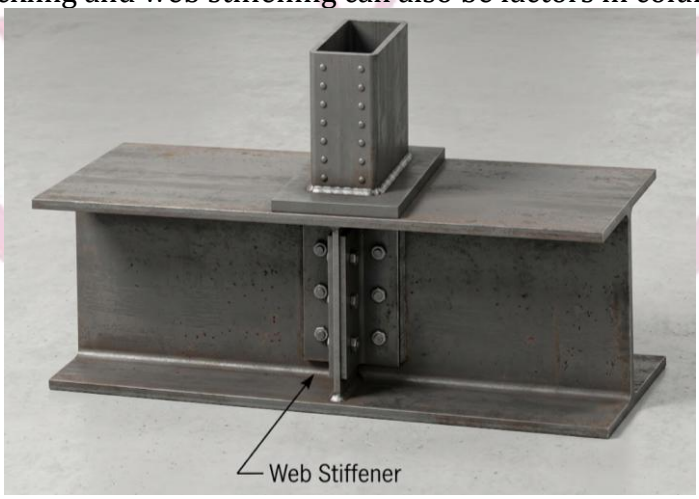
Common cold formed shapes are hollow structural sections (HSS), circular, square, and rectangular (Figure 5.10).

Common cold formed hollow structural sections (HSS)

WEB STIFFENING

In steel construction, when large shear forces are transferred through a beam's web, the shear capacity of the web may need to be increased to prevent the web from buckling. Figure 5.12 shows one method of doing so.

Web buckling is referred to as web crippling and the web reinforcing is referred to as web
stiffening. Web buckling and web stiffening can also be factors in column design.



Web stiffener on a steel beam

STEEL FRAMING

Discrete members are either bolted or welded together.

BASIC CONNECTION STRATEGIES

Type of connection	Situation
Bolted or welded methods	Resists vertical and horizontal shear forces (low or no moments)
plates or angles to the connection by additional welding or bolting in order to resist moment forces	High moments
bracing systems that introduce triangles into the frame	High lateral loads

Bolts

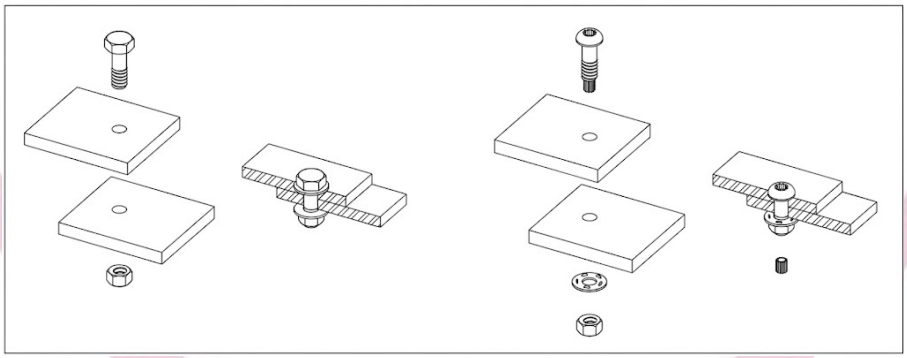
There are two types of bolts typically used:

- **The Hex Head bolts** need access from both sides for tightening, but no special equipment.
- **Tension Control (TC) bolts** need a special type of equipment to install and snap off the end, but only one side needs access for tightening.

Hex head bolt

TC bolt

If the steel is concealed then the choice of bolt type is purely a structural consideration.



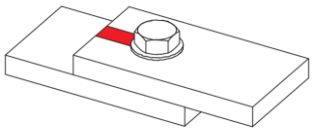
Left: Assembly of a Hex Head bolt. Right: Assembly of a Tension Control bolt.

Left: Tension Control bolt.

Right: Hex Head bolts

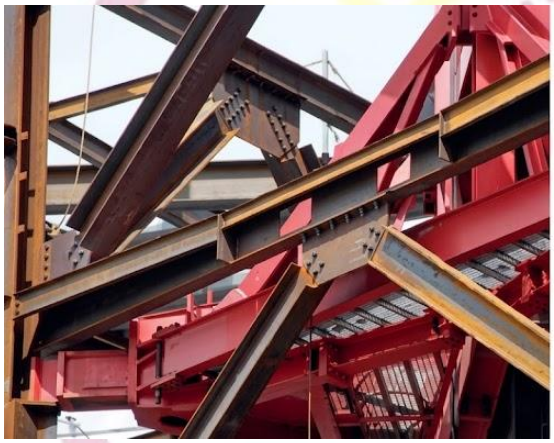
Splices

- Splice: A connection used to join two steel members.
- Splice plate: An additional steel plate used to connect two steel members.
- Lap joint: A joint in which two steel pieces overlap each other.
- Butt joint: A joint in which two steel pieces are placed end-to-end (in line).

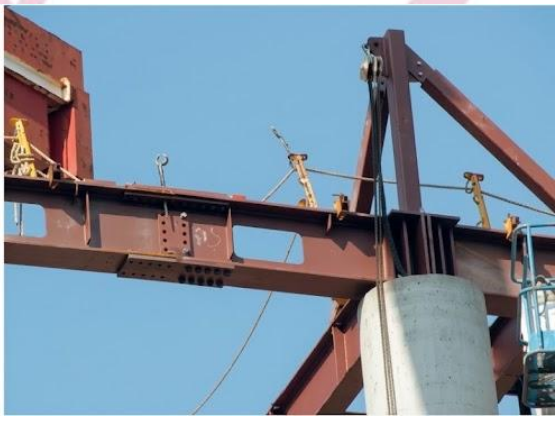


lap joint

Butt joint



Left: The bracing connections all use simple lapped connections.
Right: Where extra resistance is required, the number of lapping plates (Also visible in this connection are two different bolt types).

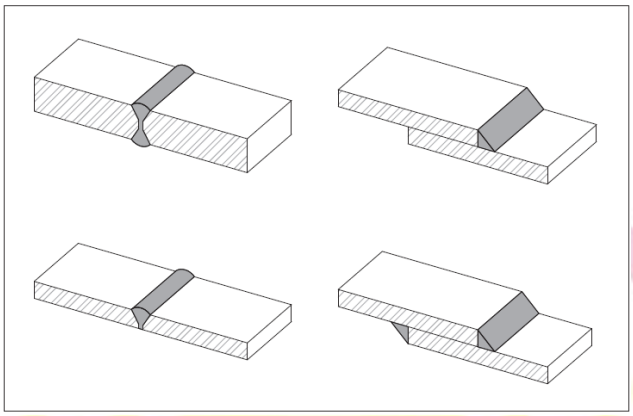


Left: The splices between the wide-flange members (Additional reinforcing plates can be seen on the top and bottom of the connection flanges).
Right: A butt joint is used to splice the beams.

Welded connections:

Plates can be spliced together using two basic types of welded connections.

1- **Groove welds** are used where the two plates must be maintained in line (see below figure/left).
2- **Lap welds** are used if it is not important to align the plates (see below figure/right).



Grinding is used for special high-profile applications as it is expensive and time consuming. Grinding also weakens the weld by removing weld material.

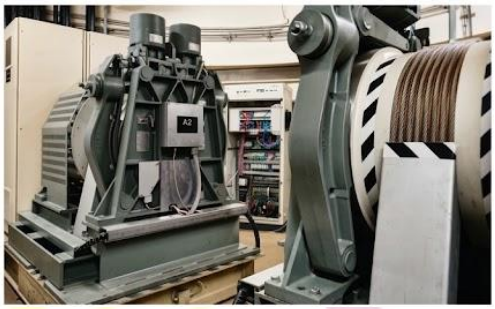
Welded connections will normally be used when fabricating large primary elements like a large plate girder or composite sections in the shop. Quality welding is best done under controlled conditions.

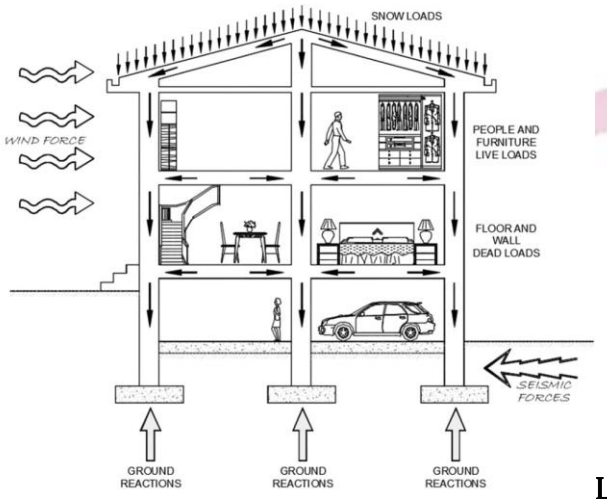
Welded connections are also preferred when fabricating complex trusses from HSS members.

CHAPTER 2
Specifications, Loads, and Methods of Design

LOADS

There are many types of loads on a structure, which can be broadly classified as dead, live, and environmental.

Dead Loads:	Dead loads are fixed and of constant magnitude, such as floors, walls, partitions, ceiling and floor finishes, and other fixed equipment. The weights of construction materials are available from various sources, including the AISC Manual.	 Dea Load (elevator machine)
Live Loads	are temporary, transient, and variable. They include things such as people, furniture and equipment, vehicles, and stored goods. Live loads are given in local building codes according to the specific use and occupancy of a building.	 live load (runners on a bridge)
Environmental loads:	Loads resulting from natural environmental phenomena, such as rain, ice, snow, wind, and seismic activity (earthquakes). They vary depending on the geographic location of the structure.	 Earthquake damage



Loads on a structure

LOAD COMBINATIONS

- The design must account for the load combination that has the most critical effect. So, the question becomes how to combine loads reasonably.
- building codes, prescribe load combinations to determine the appropriate loading to use.
- Structures must be designed to resist the most critical effects from the load combinations.
- In the load combinations, each **service load** is multiplied by a load factor reflecting the uncertainty of that particular load. With a load factor applied, the resulting load is called a **factored load**.

The following terminology will be frequently used throughout the text:

Service loads	The loads on a structure, without the application of load factors.
Factored loads	The loads on a structure, after the application of load factors.
Design loads	The loads obtained from the governing load **combinations.**

COMPUTING COMBINED LOADS WITH LRFD EXPRESSIONS

The AISC Manual provides the following load factors for buildings.

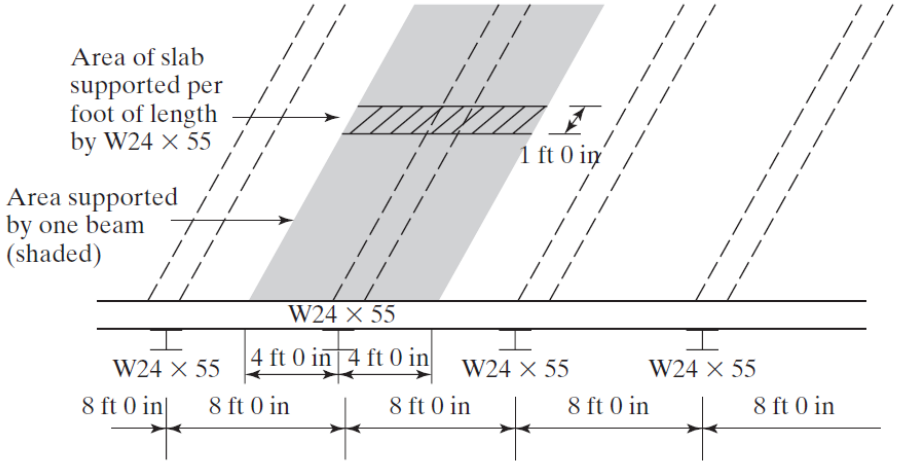
1 $U = 1.4D$
2 $U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3 $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L^* \text{ or } 0.5w)$
4 $U = 1.2D + 1.0W + L^* + 0.5(L_r \text{ or } S \text{ or } R)$
5 $U = 1.2D + 1.0E + L^* + 0.2S$
6 $U = 0.9D + 1.0W$
7 $U = 0.9D + 1.0E$

*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.

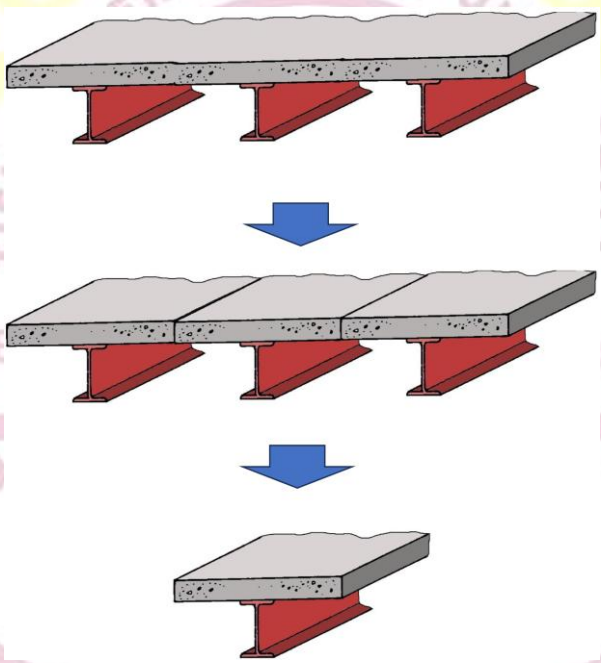
In these load combinations, the following abbreviations are used:
U = the design or factored load
D = Dead load
L = Live load
L_r = roof live load
S = Snow load
R = nominal load due to initial rainwater or ice, exclusive of the ponding contribution.
W = Wind load
E = Earthquake load

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

COMMON LENGTHS OF STEEL SECTIONS

Common mill lengths typically range from 30 ft to 65 ft, usually in 5-ft increments. The available lengths may vary depending on the individual steel mill, supplier, and method of shipment.

PRACTICAL GUIDE FOR SELECTION THE SIZES OF THE STEEL SECTIONS

Beam: A horizontal structural member and transfers loads to columns or girders.

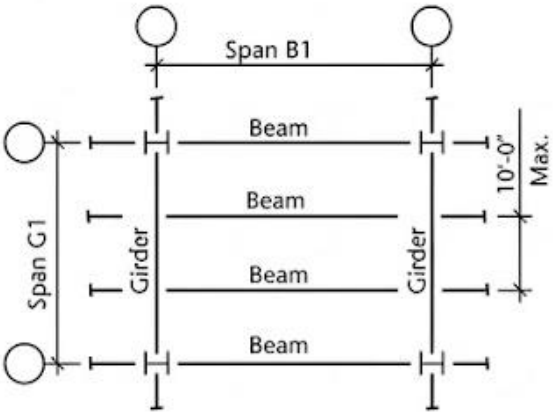
Girder: A main beam that supports other beams and transfers their loads to columns. Girders are usually larger and carry heavier loads than ordinary beams.

Column: A vertical structural member that mainly carries compressive loads and transfers the loads from beams and floors down to the foundation.

Beam → supports loads

Girder → supports beams

Column → supports beams/girders and transfers loads to the foundation



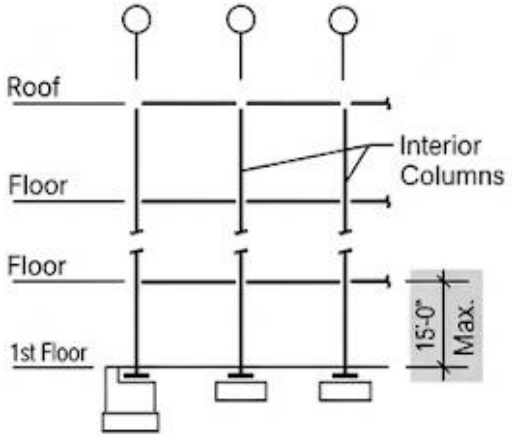
A- Sizes of beams

Table A Beam Sizes						
Live Load, psf	Beam Span: B1 (ft)					Classification
	20	25	30	35	40	
50	W10-W16	W14-W16	W16-W21	W18-W21	W21-W27	Office
100	W12-W16	W14-W18	W16-W24	W18-W24	W21-W27	Assembly
150	W14-W18	W18-W21	W18-W24	W21-W27	W24-W30	Storage

B- Sizes of girders

Table A20 Girder Sizes						Beam Span 20 feet
Live Load, psf	Girder Span: G1 (ft)					Classification
	20	25	30	35	40	
50	W16-W18	W18-W21	W21-W21	W24-W27	W24-W27	Office
100	W18-W21	W21-W24	W24-W27	W24-W30	W27-W30	Assembly
150	W21-W24	W21-W24	W24-W27	W24-W30	W30-W33	Storage

C- Column sizes



Multi-story Building

Typical Interior Column Size (Depth × Width)

BAY SPACING	NUMBER OF STORIES					
	1	2	3	4	5	6
20 X 20	4 X 4	6 X 6	10 X 10	12 X 10	14 X 10	14¼ X 10¼
20 X 25	4 X 4	8 X 8	10 X 10	12 X 10	14 X 10	14¼ X 10¼
20 X 30	5 X 5	8 X 8	10 X 10	12 X 10	14 X 10	14½ X 10¼
20 X 35	5 X 5	10 X 8	10¼ X 10	12¼ X 12	14¼ X 10¼	14 X 14½
20 X 40	5 X 5	10 X 8	10¼ X 10	12¼ X 12	14¼ X 10¼	14 X 14½
25 X 25	4 X 4	7 X 7	10 X 10	12 X 10	14¼ X 10¼	14¼ X 10½
25 X 30	5 X 5	7 X 7	10¼ X 10	12¼ X 12	14¼ X 10¼	14 X 14½
25 X 35	5 X 5	8½ X 8½	12¼ X 10	12¼ X 12¼	14 X 14½	14¼ X 14¼
25 X 40	5 X 5	8½ X 8½	12⅛ X 10	12¼ X 12¼	14 X 14½	14¼ X 14¼
30 X 30	5 X 5	8½ X 8½	12¼ X 10	12¼ X 12¼	14 X 14½	14¼ X 14¼
30 X 35	5 X 5	8½ X 8½	12¼ X 12	14 X 14½	14¼ X 14¼	14½ X 14¼
35 X 40	6 X 6	10¼ X 10¼	12¾ X 12	14 X 14½	14¼ X 14¼	14½ X 14¼
35 X 35	6 X 6	10¼ X 10¼	12¼ X 12	14 X 14½	14½ X 14¼	14½ X 14¼
35 X 40	6 X 6	10¼ X 10¼	12¾ X 12¾	14¼ X 14¼	14½ X 14¾	15 X 15%
40 X 40	8 X 8	12¼ X 10	12¾ X 12¾	14¼ X 14¼	14⅛ X 14¾	15 X 15%

FRAMED CONNECTIONS

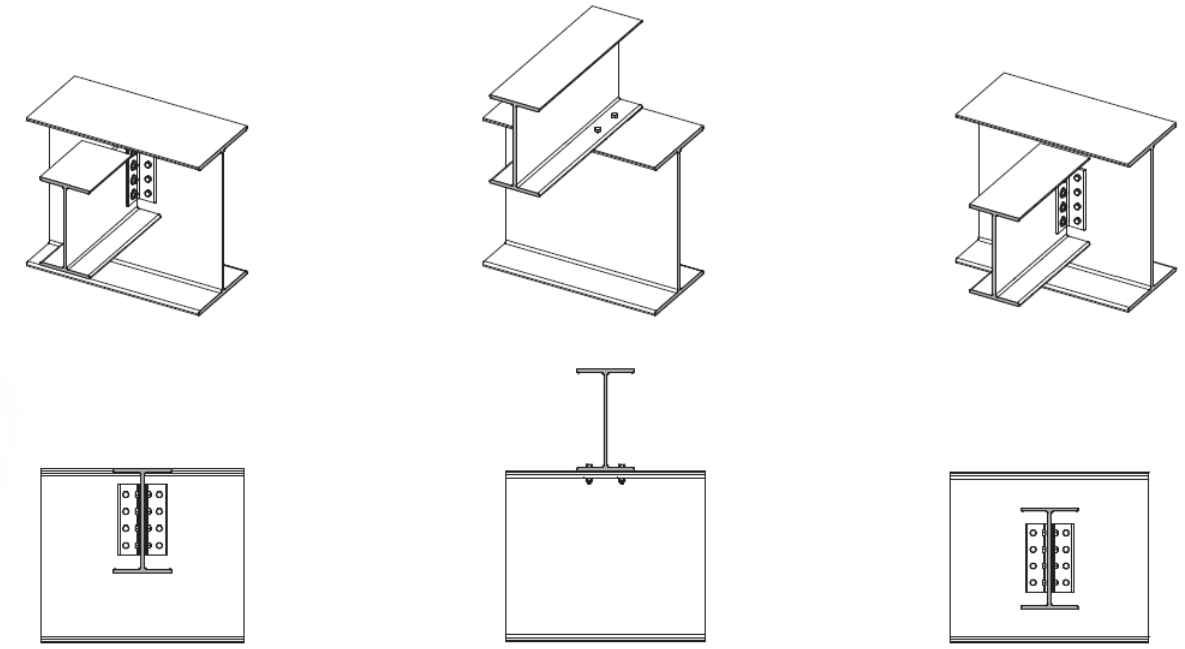
Steel structures are assembled using a basic suite of connection types:

1- Beam -to-Girder Connections
2- Girder- or Beam-to-Column Connections
3- Column Connections

1- Beam -to-Girder Connections

There are three basic ways to frame a beam into a girder.

1- Coped connection (see below figure/ Left)
2- Bearing connection (see below figure/ Middle)
3- Simple framed connection (see below figure / Right) (used where there is no floor element to be supported.

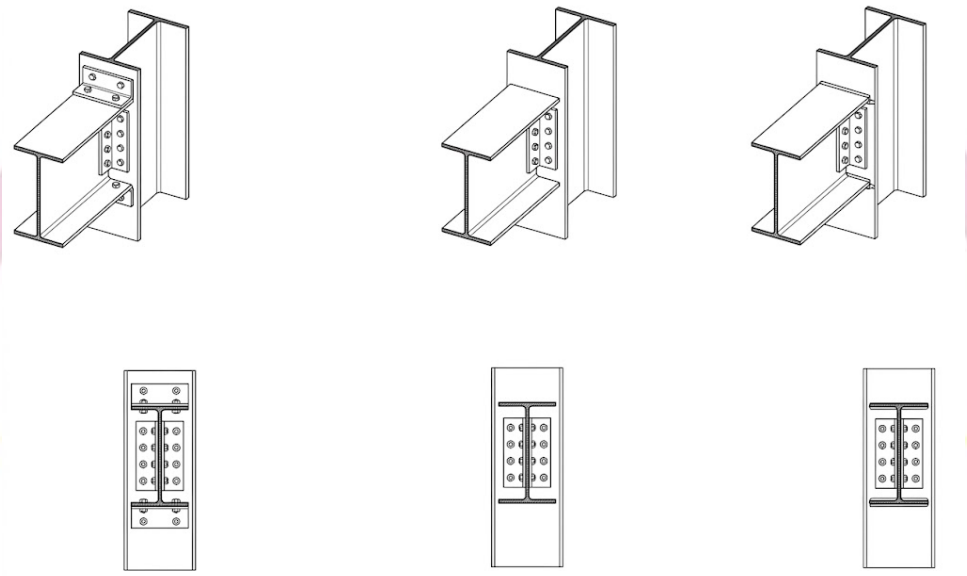


Beam -to-Girder Connections

Left: a coped connection provides a level surface for the installation of the floor deck.
Right: bearing connections

<u>2- Girder- or Beam-to-Column Connections</u>

a. **Seated connection** (Angles are bolted to the column to provide a ledge for the beam during erection (see figure below/left).
b. **Standard framed connection** (the angles are bolted to the web of the beam at the shop and then bolted to the column flange on site. The connection acts as a hinge in that it is only designed to resist shear (see figure below/ middle)
c. **Moment connection:** The connection (on the right) has been reinforced to resist moment. Plates have been welded to the column prior to erection. They are also welded to the flanges of the beam so as to provide resistance to bending at the connection.



These connections can be made either to the flange of the column or to the web, depending on the orientation of the column, which is a function of the structural layout. Columns are generally oriented so that the **dominant wind** load strikes perpendicular to the flange of the column.

3- <u>Column Connections</u>

A: column to foundation connections:

Steel columns are generally welded to a base plate that is used to attach the column to the foundation pier or supporting system.

Left: This simple base connection uses four threaded bolts to anchor the plate (The column member is pin-connected to the base.

Right: A round plate is welded to the base of the round HSS column.

B. Column Splices

The splice is normally placed around 600mm (24 inch) above the floor level.

Column splices can be done as

1- Bolting (see figure below/right)
2- Using matching plage (see figure below/middle)
3- Welding (see figure below/left)

STEEL STRUCTURAL SYSTEMS: FLOOR SYSTEMS, BRACED FRAMES, AND TRUSS SYSTEMS

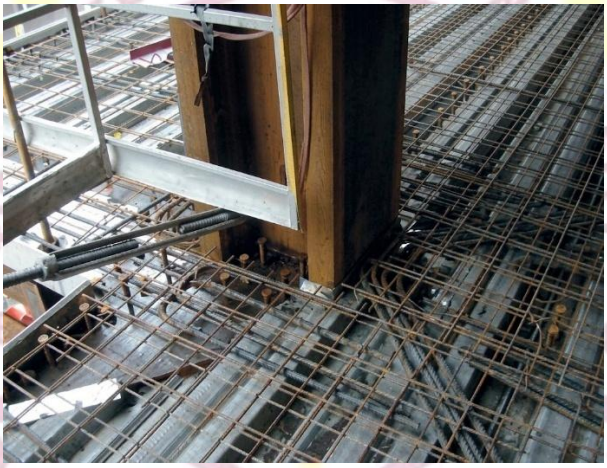
Steel buildings use different structural systems to safely carry gravity and lateral loads. Floor systems transfer loads to beams and columns, braced systems provide lateral stability, and truss systems efficiently support long spans using lightweight structural members.

FLOOR SYSTEMS

The floor deck can consist of:

- profiled steel decking with a concrete topping (deck depth varies from 38 to 91mm)
- hollow-core precast concrete slabs

hollow-core precast concrete slabs



Prior to pouring the concrete floor slab.

BRACED SYSTEMS

Framed and pin connections are inherently unstable. Buildings must have additional means to provide lateral stability to the frame using:

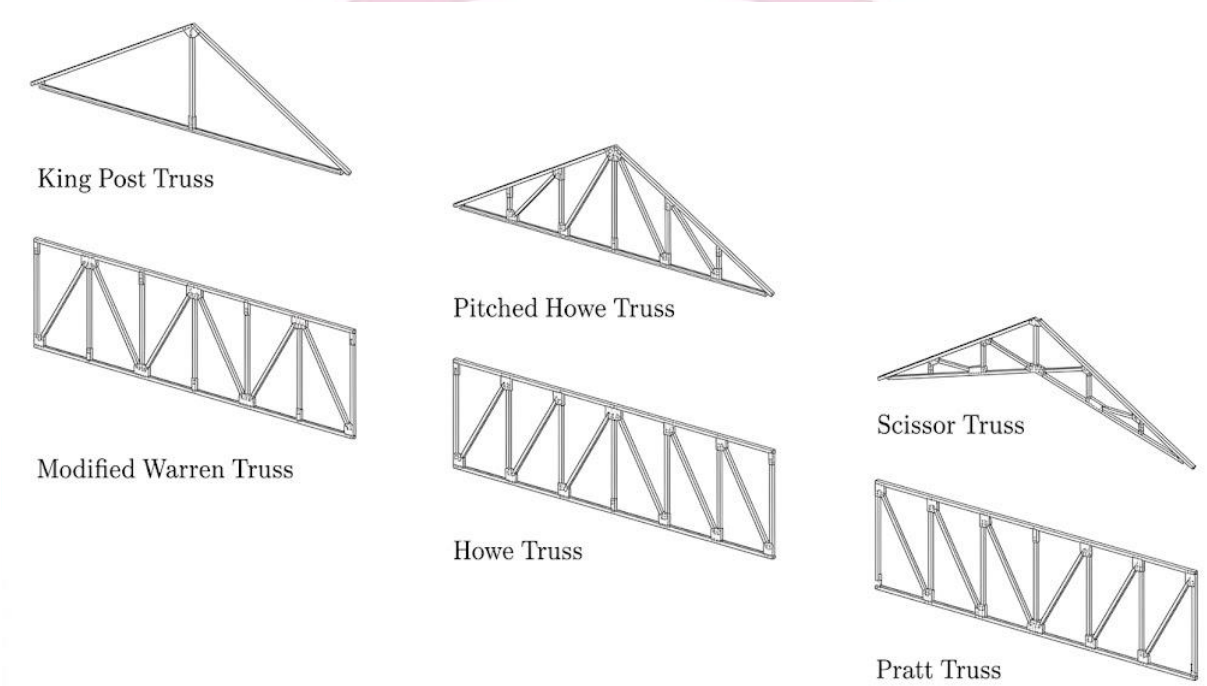
- Reinforcement of the framed connections themselves to provide moment resistance
- Addition of bracing to the frame
- Use of shear walls (either in concrete or steel plate)
- Using concrete stair and elevator cores
- Using concrete floor systems.

TRUSS SYSTEMS

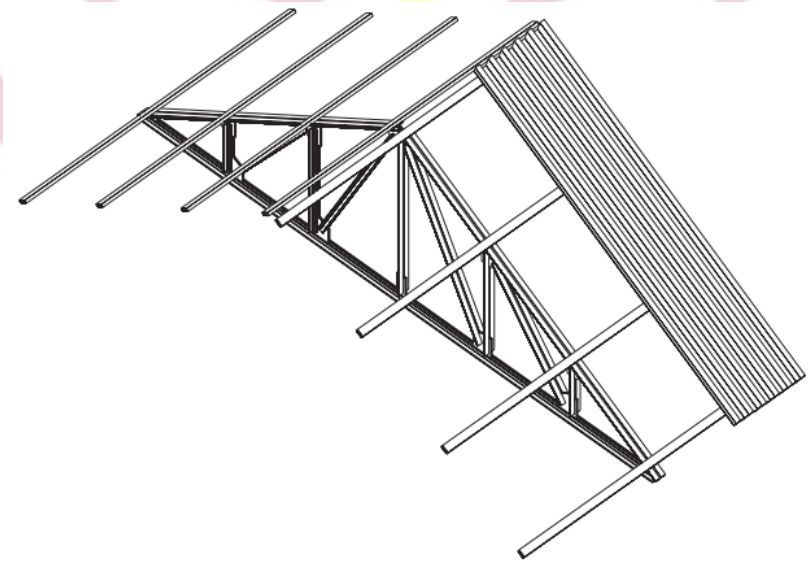
A truss is a structure comprising one or more triangular units constructed with straight members, whose ends are connected at joints referred to as nodes.

Trusses can be:

1- A **planar truss** is two-dimensional, with all of the members lying in essentially a single plane.
2- **Box-type trusses** also span only in one direction but have a three-dimensionality to them that is usually rectangular or triangular.
3- **Space trusses** are also called spaceframes. These systems can span in multiple directions.



Planar Truss Types (the most common ones)



The steel decking for this roof sits on purlins. Purlins are used to span between trusses

Modified Pratt truss.

Left: The long-span Warren trusses (fabricated from square HSS members). Wide-flange sections carry the load of the steel decking to the trusses. Cross bracing is introduced in the plane of the roof.
Right: The trusses are fabricated from wide-flange sections. The incoming beams that support

The skylight support system uses a two-way Vierendeel truss system created from welded square HSS members (uses square HSS sections).

Three-Dimensional Trusses

Three-dimensional truss systems are used as a means to limit the span requirements of the structural members that carry the roof or floor loads to the trusses.

Box-type trusses have a linear span direction.

Space frames can span freely in multiple directions.

triangular trusses that are braced between with lighter round HSS members, giving the structure a space frame-like appearance.

Curved triangular trusses have single round HSS top chords and a pair of round HSS bottom chords that are separated by smaller round HSS web members.

Left: The fabric roof is supported by a single three-dimensional trussed arch.

Right: The truss was divided into sections. High-strength moment-resisting connections were then used to create continuity in the chords. Joints were fabricated by welding plates to the ends of the round HSS members. The connection between the tube and the plate is stiffened by the addition of triangular plates between each bolt hole.

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

- cable supported roofs, ski lifts, and other similar applications. High strength Steel cables are used for suspension bridges,

- Tension members usually consist of single angles, double angles, tees, channels, W sections, or sections built up from plates or rolled shapes, see Fig. 3.1 for more details.

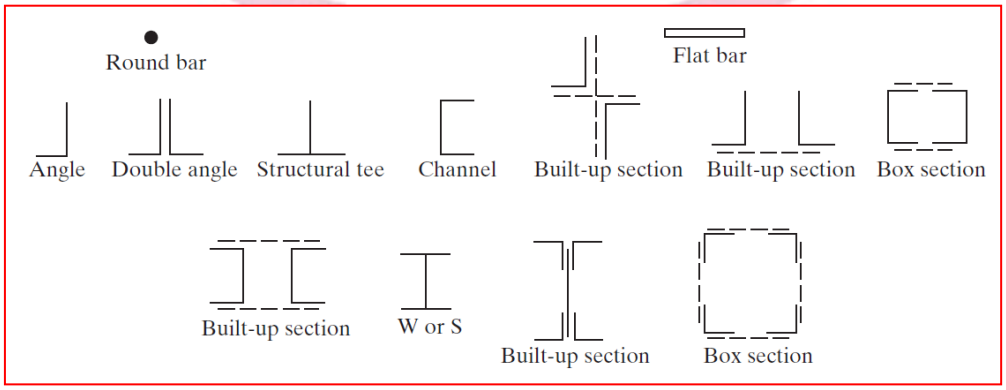
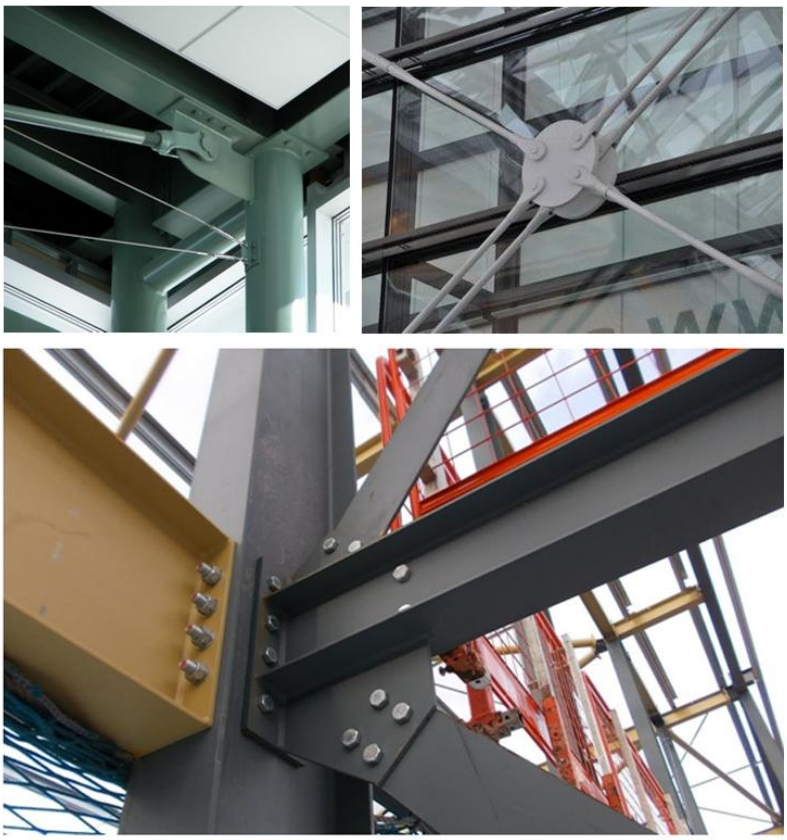


FIGURE 3.1 Types of tension members.



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.

Notice the crack developing at the hole.

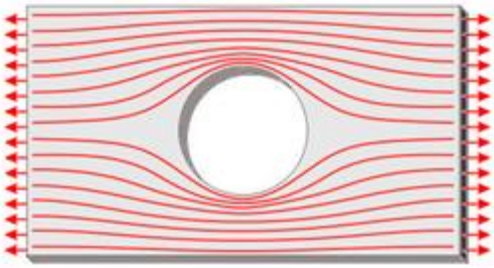
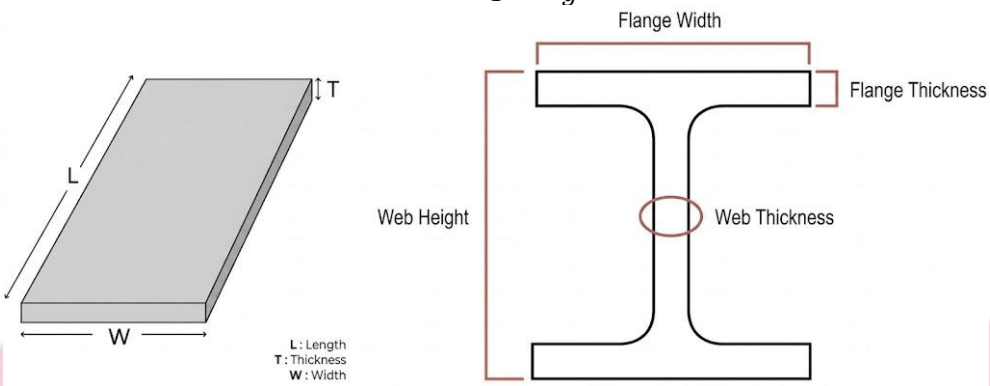


Figure. Stresses around hole

The term "net cross-sectional area," or simply, "net area"(A_n) refers to the gross cross-sectional area of a member, minus any holes cross-sectional area.

$$A_n = A_g - A_h$$

Where A_g is the gross (total) sectional area. for the plates $A_g = W \times t$. For steel sections such as W-section or C-seciton, get A_g form the tables.



and

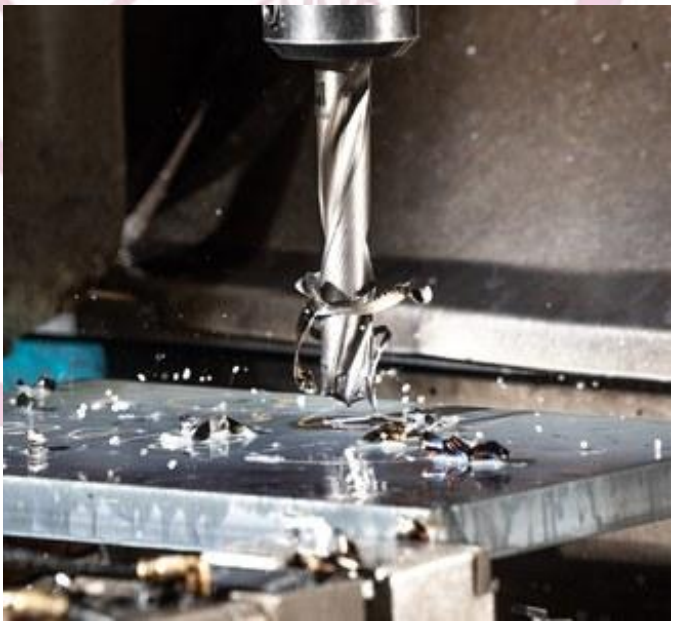
$$A_h = D_h \times t$$

The value of t is either the thickness of the plate, web or the flange

$D_h = D_{bolt} + \frac{1}{8}$ if the holes are punched

$D_h = D_{bolt} + \frac{1}{16}$ if the holes are drilled

Punching machine



Steel plate drilling

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.

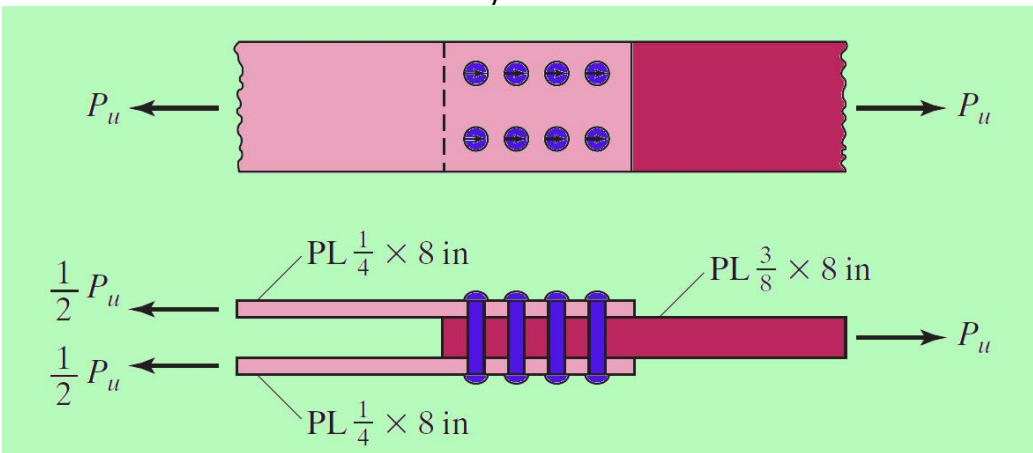


Fig. 3-2 Illustration of Example 3-1

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

EFFECT OF STAGGERED HOLES

Tensile members could fail transversely along line AB in either Fig. 3.4(a) or 3.4(b). the failure could be other than a transverse one. The holes are staggered, and failure along section ABCD is possible unless the holes are a large distance apart.

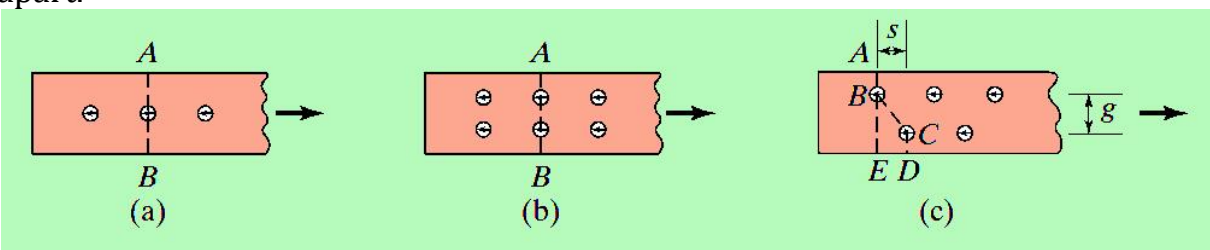


FIGURE 3.4 Possible failure sections in plates.

For staggered failure section, the net area is calcuated as

$$A_n = A_g - A_h + \frac{s^2}{4g} \times t$$

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.

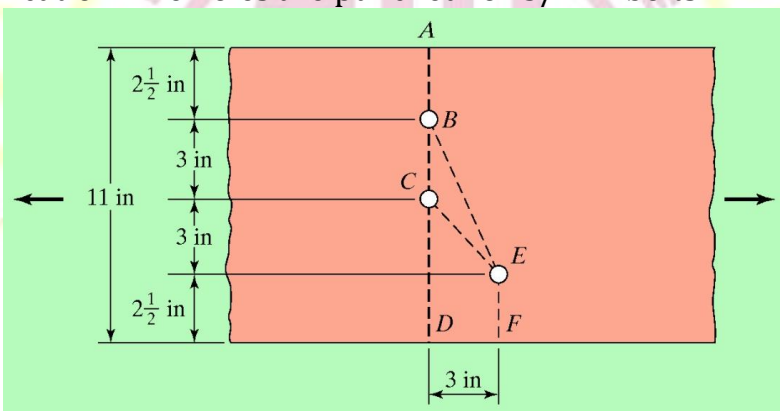


FIGURE 3.5

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 3-4

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.

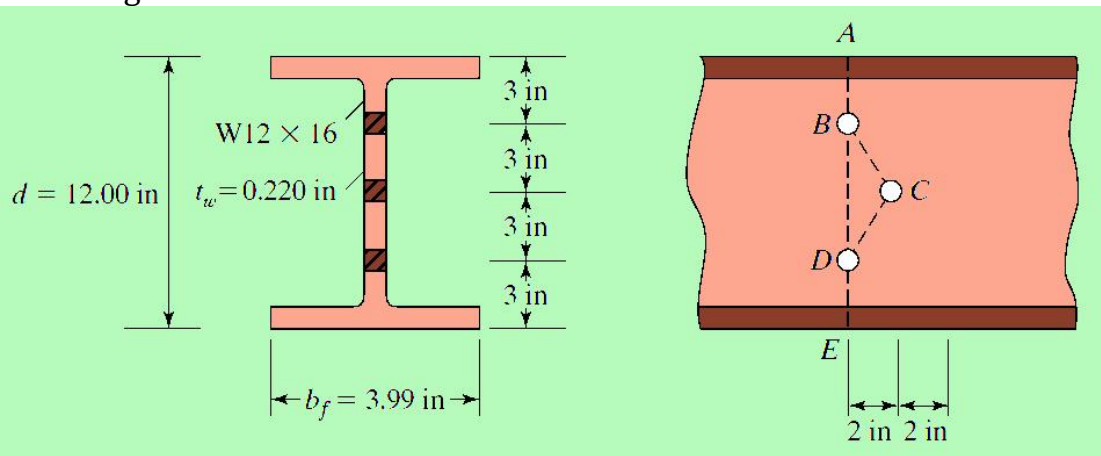


FIGURE 3.7

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

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

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

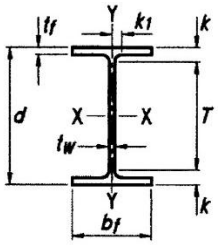


Table 1–1 (continued)

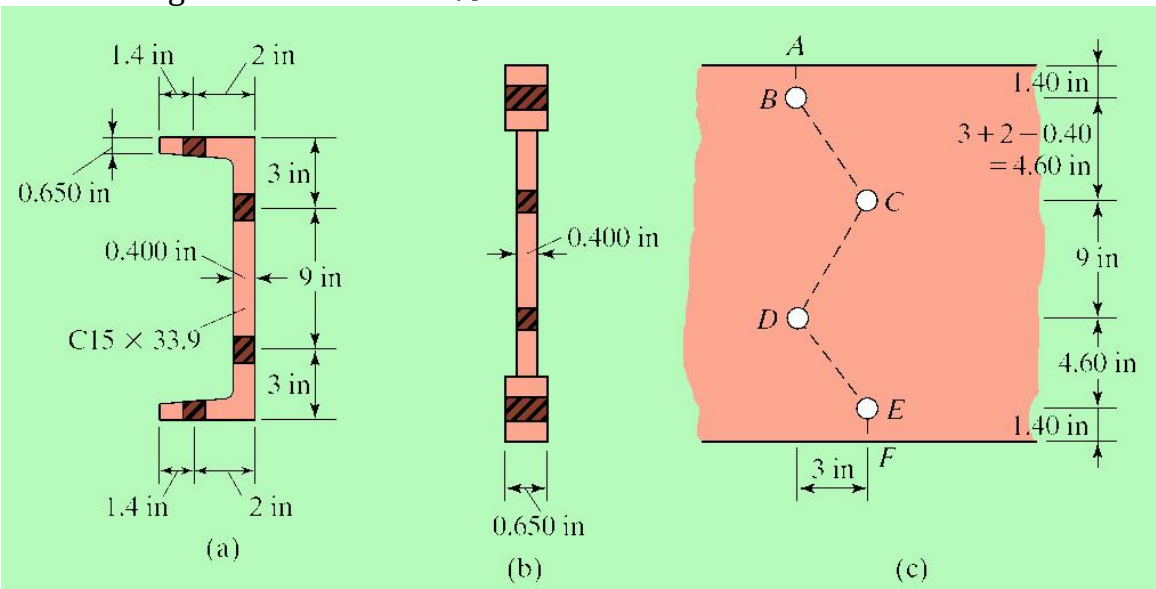
W Shapes

Dimensions

Shape	Area, A	Depth, d		Web 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	12¼	0.360	⅜	3/16	10.0	10	0.640	⅝	1.24	1½	15/16	9¼	5½
×53	15.6	12.1	12	0.345	⅜	3/16	10.0	10	0.575	9/16	1.18	1⅜	15/16	9¼	5½
W12×50	14.6	12.2	12¼	0.370	⅜	3/16	8.08	8⅛	0.640	⅝	1.14	1½	15/16	9¼	5½
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1⅜	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	½	1.02	1⅜	⅞	↓	↓
W12×35[c]	10.3	12.5	12½	0.300	5/16	3/16	6.56	6½	0.520	½	0.820	1 3/16	¾	10⅛	3½
×30[c]	8.79	12.3	12⅜	0.260	¼	⅛	6.52	6½	0.440	7/16	0.740	1⅛	¾	↓	↓
×26[c]	7.65	12.2	12¼	0.230	¼	⅛	6.49	6½	0.380	⅜	0.680	1 1/16	¾	↓	↓
W12×22[c]	6.48	12.3	12¼	0.260	¼	⅛	4.03	4	0.425	7/16	0.725	15/16	⅝	10⅜	2¼[g]
×19[c]	5.57	12.2	12⅛	0.235	¼	⅛	4.01	4	0.350	⅜	0.650	⅞	9/16	↓	↓
×16[c]	4.71	12.0	12	0.220	¼	⅛	3.99	4	0.265	¼	0.565	13/16	9/16	↓	↓
×14[c,v]	4.16	11.9	11⅞	0.200	3/16	⅛	3.97	4	0.225	¼	0.525	¾	9/16	↓	↓

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

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

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

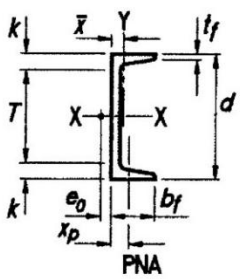


Table 1–5
C Shapes
Dimensions

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	3¾	0.650	⅝	1 7/16	12⅛	2¼	1.17	14.4
×40	11.8	15.0	15	0.520	½	¼	3.52	3½	0.650	⅝	1 7/16	12⅛	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	⅜	3/16	3.40	3⅜	0.650	⅝	1 7/16	12⅛	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	½	¼	3.17	3⅛	0.501	½	1⅛	9¾	1 3/49	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 3/49	1.00	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, see Fig. 3.9(a).

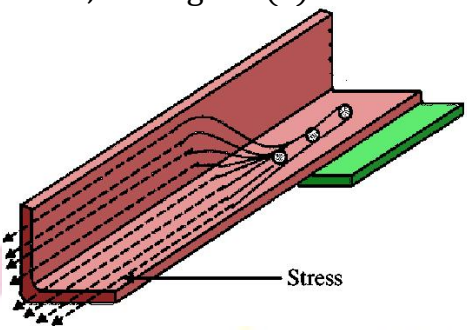


FIGURE 3.9 Shear lag.

In such a situation, the connected cross section is not 100 percent effective.
The effective net area, A_e, is determined by

$$A_e = A_n U$$

where U is a reduction factor that accounts for the nonuniform stress distribution

The value of U is calcauted form Table 3.2

For case 1, $U = 1$
For case 2, the value of U is calcuated as

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

where $\bar{x}$ is the distance measured from the plane of the connection to the centroid of the area of the whole section.
and L is the length of its connection.

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.

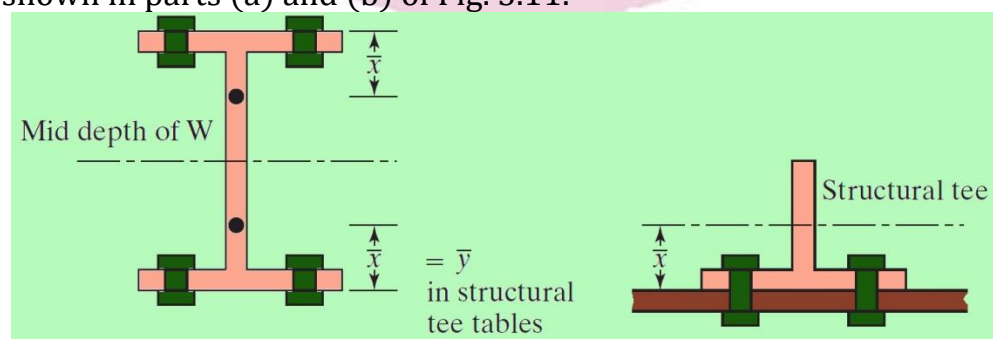
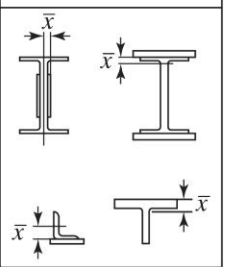
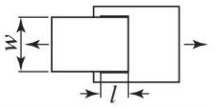
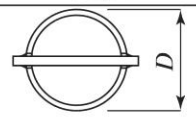
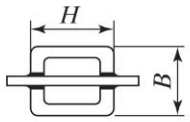
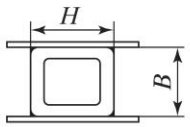


FIGURE 3.11 Values of $\bar{x}$ for different shapes.

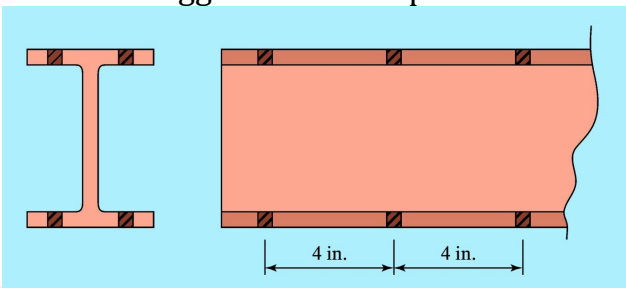
TABLE 3.2 Shear Lag Factors for Connections to Tension Members

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)

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

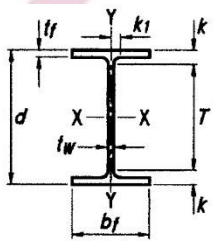


Table 1–1 (continued)
W Shapes
Dimensions

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}	Distance k_1	Distance T	Work-able 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

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

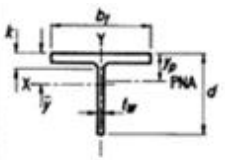


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	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,*]	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	0.685	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,†]	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[†]	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.
[†] 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.
[*] 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: $\frac{b_f}{2t_f}$	Compact Section Criteria: $\frac{h}{t_w}$	Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis X-X: $\bar{y}$	Axis X-X: Z	Axis X-X: y_p	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Axis Y-Y: Z	Q_s $F_y = 50$ ksi	Torsional Properties: J	Torsional Properties: 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	0.875	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

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

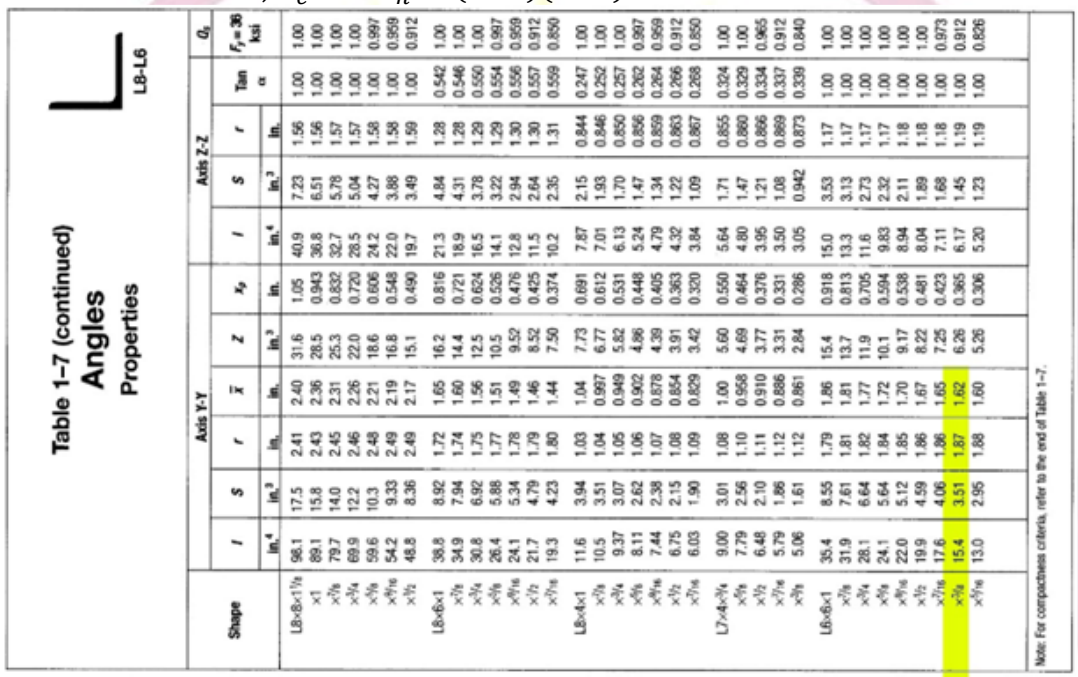


Table 1–7 (continued)
Angles
Properties

L8-L6

Shape	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y $\bar{x}$	Axis Y-Y Z	Axis Y-Y x_p	Axis Z-Z I	Axis Z-Z S	Axis Z-Z 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.

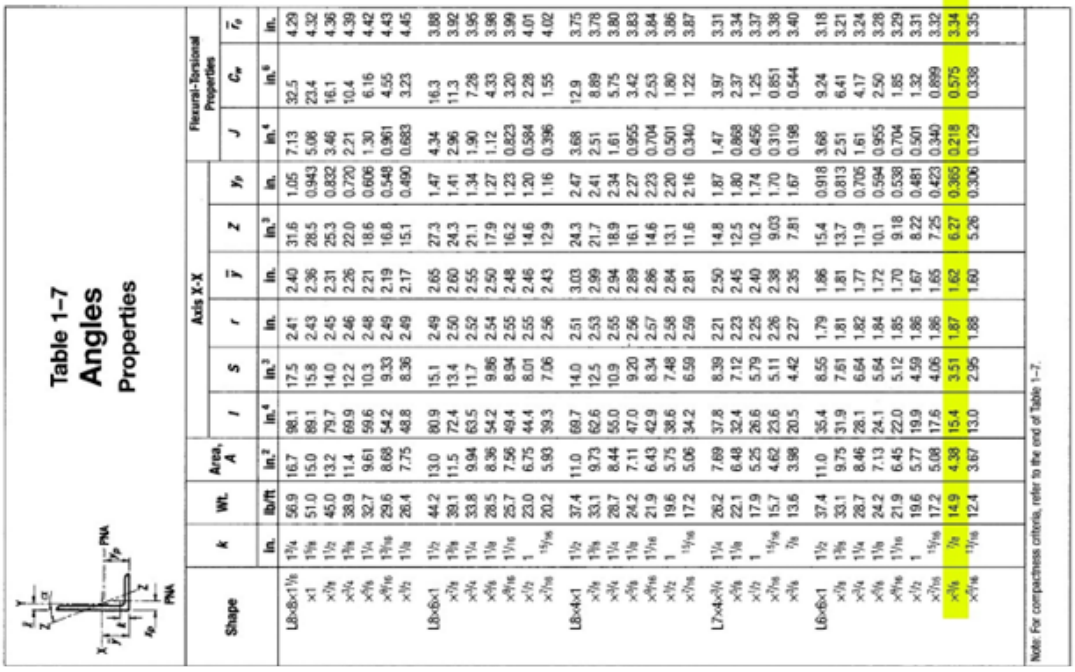


Table 1–7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X $\bar{y}$	Axis X-X 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	1⅛	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.

EFFECTOVE AREA OF WELDED MEMBERS

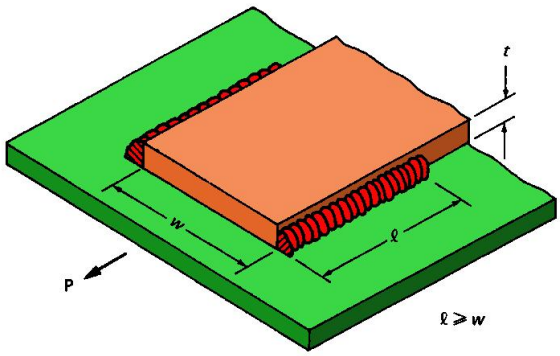
A. Welded Pates

See Table 3.2/ Case 4

$$A_e = A_g . U$$

$$A_g = W . t$$

U is either 1.0, 0.87, or 0.75 based on the lentgh of the wedling and the value of W (see case for details)



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

FIGURE 3.12 a

Example 3-8

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

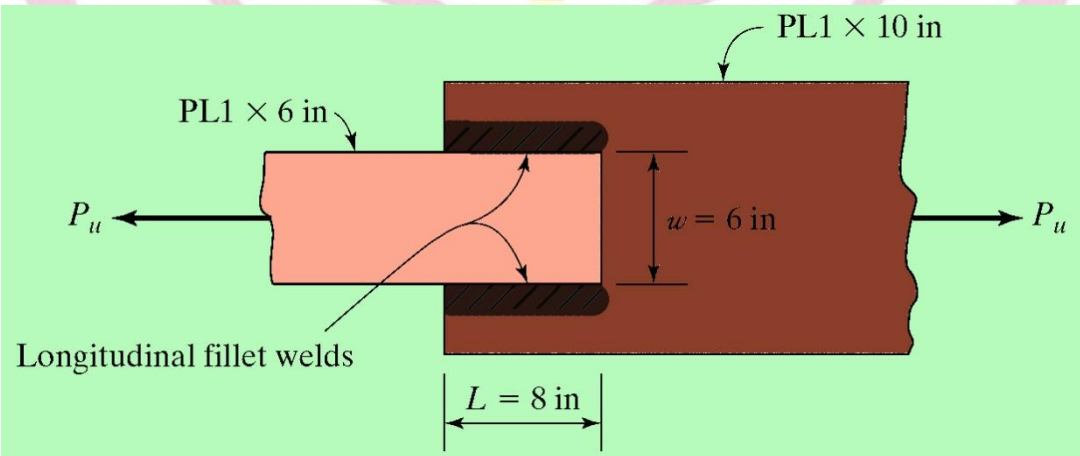


FIGURE 3.13

Solution:

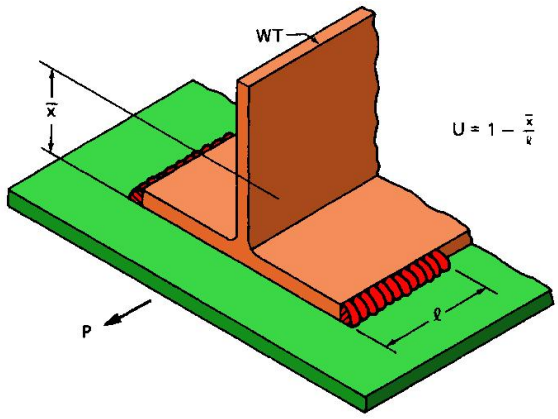
$$(1.5w = 1.5 \times 6 = 9\ in.) > (L = 8\ in.) > (w = 6)$$

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

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

B. Welded Tee section

See Table 3.2/ Case 4

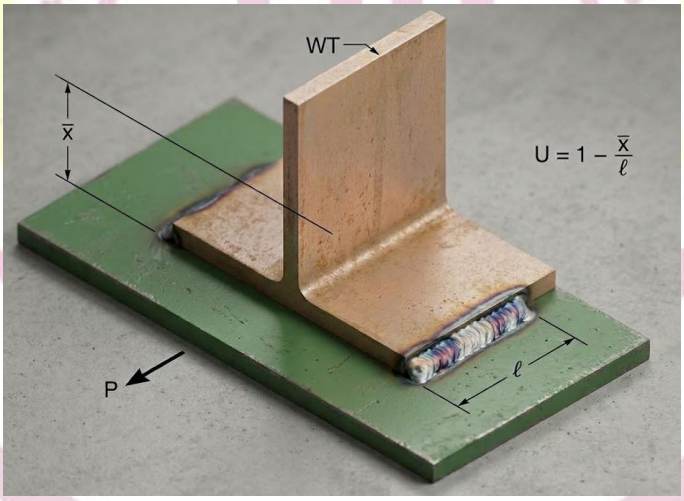


x̄ = distance from the centroid of the shape to the plane of the connection, in.

ℓ = weld length, in.

FIGURE 3.12 b

Example: Determine the effective area of the WT 5 × 15 shown below, given that the length of the weld is 6.0 in.



Solution.

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

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

$l = 6$ in.

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

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

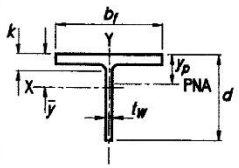


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k		Distance: 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: $\frac{b_f}{2t_f}$	Compact Section Criteria: $\frac{h}{t_w}$	Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis X-X: $\bar{y}$	Axis X-X: Z	Axis X-X: y_p	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Axis Y-Y: Z	Q_s $F_y = 50$ ksi	Torsional Properties: J	Torsional Properties: 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

$$\min A_g = \frac{P_u}{\phi_t F_u U} + \ Area\ of\ holes$$

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

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

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:

$$P_u = 1.4D$$

$$P_u = 1.2D + 1.6L$$

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). $[F_y = 50\ ksi, F_u = 65\ ksi]$.

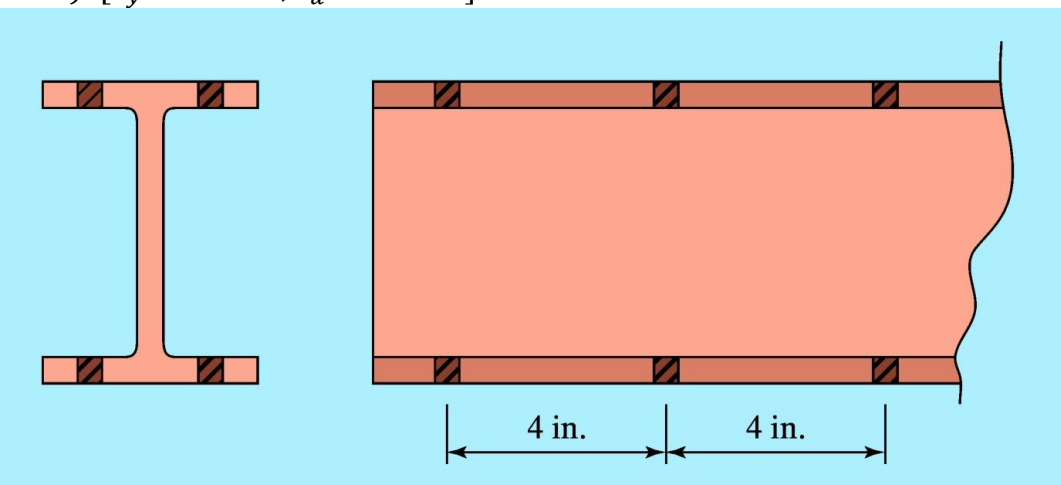


FIGURE 4.1 Cross section of member for Example 4-1.

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 in² 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

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

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

Summary

Design of Tension Members

1. $\min A_g = \dfrac{P_u}{\phi_t F_y}$ $\qquad \phi_t = 0.9$
2. $\min A_g = \dfrac{P_u}{\phi_t F_u U} +$ Area of holes $\qquad \phi_t = 0.75$
3. $\min r = \dfrac{L}{300}$

Checking

(a) Gross section yielding

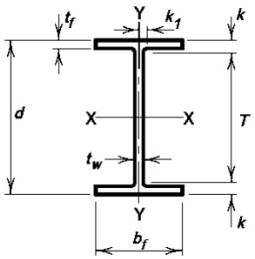
$$P_u < \phi_t P_n$$
$$P_n = F_y A_g \text{ and } \phi_t = 0.9$$

(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

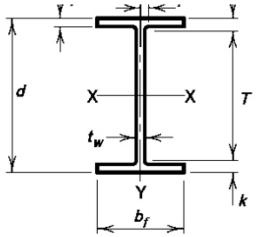
$$\frac{L}{r} < 300$$



W SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance 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 $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Compact Section Criteria F_y'''	X_1	$X_2 \times 10^6$	Elastic Properties Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Plastic Modulus Z_x	Plastic Modulus 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

CHAPTER 5
Introduction to Axially Loaded Compression Members

INTRODUCITON

Steel columns are vertical structural members that primarily carry compressive loads and transfer them from beams and floors to the foundation. They are designed to resist compression, buckling, and, in some cases, bending and tension. Common steel column sections include W-shapes, HSS sections, and built-up sections.

END RESTRAINT AND EFFECTIVE LENGTHS OF COLUMNS

End restraint describes how strongly the ends of a column are prevented from rotation and lateral movement by the connected beams, slabs, or foundations.

Effective length is the equivalent length of an ideal pin-ended column that has the same buckling behavior as the actual column. It is given by:

$$L_e = K.L$$

where L is the actual column length and K is the effective length factor, which depends on the end restraints. See Figure 5.1 for practical K values.

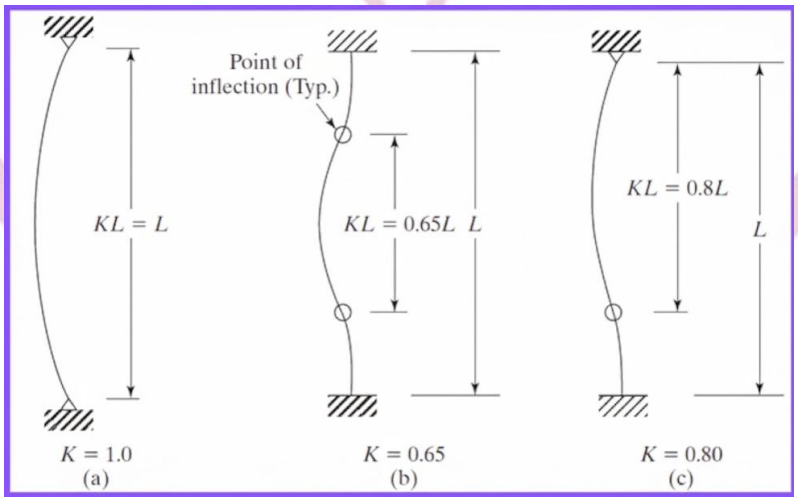


FIGURE 5.1 Effective length (KL) for columns in braced frames (sidesway prevented).

Columns End restaint

COLUMN FORMULAS

LRFD compression strength $\phi_c P_n$ is:

$$\phi_c P_n = \phi_c F_{cr} A_g$$

where F_{cr} is the flexural buckling stress. P_n is the nominal strength, and $[\phi_c = 0.90]$

The value of F_{cr} may be determined

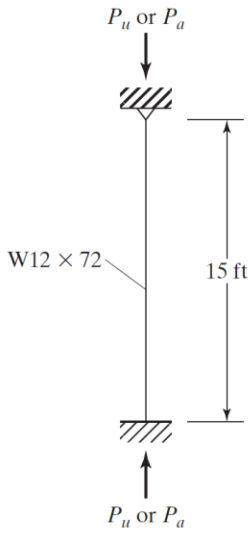
a) If $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ $\Rightarrow F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$

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

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
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

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

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
$\Omega_c = 1.67$	$\phi_c = 0.90$

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

W12

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

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 $\phi_c F_{cr} = 33.85\ ksi$

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

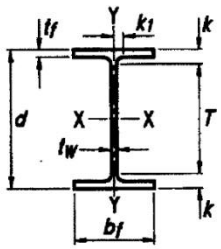


Table 1–1 (continued)

(1) **W Shapes**

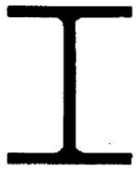
Dimensions

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

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
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200

(1) **Table 4–22 (continued)**

Available Critical Stress for Compression Members

(2)

$F_y = 35$ksi			$F_y = 36$ksi			$F_y = 42$ksi			$F_y = 46$ksi			$F_y = 50$ksi		
$\frac{Kl}{r}$	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

(3)

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 $F_y = 50\ ksi$

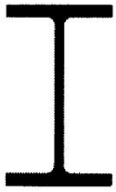
For $\boldsymbol{W8 \times 48}$ we get $\phi_c P_n = 497\ ksi > 492$[OK]

(2) F_y = 50 ksi

Table 4–1 (continued)

Available Strength in Axial Compression, kips

(1) **W Shapes**

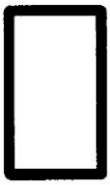


W8

Shape	W8×											
Wt/ft	67		58		48		40		35		31	
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
least 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 $\boldsymbol{HSS\ 12 \times 8 \times 3/8}$ we get $\phi_c P_n = 498\ ksi > 492$[OK]



HSS12

Table 4–3 (continued)

Available Strength in Axial Compression, kips

(2) F_y = 46 ksi

(1) **Rectangular HSS**

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
gyration r_y 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
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
(4) 10	331	498	275	414	204	307	130	196	435	653	359	539
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 $\boldsymbol{HSS\ 10 \times 10 \times 3/8}$ we get $\phi_c P_n = 511\ ksi > 492$[OK]

Table 4–4 (continued)

Available Strength in Axial Compression, kips

F_y = 46 ksi (2)

HSS12–HSS10

(1) Square HSS

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 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
(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
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	
(3) Design	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
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
(4) 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

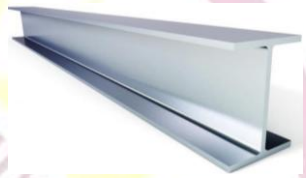
CHAPTER 7
Introduction to Beams

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, and S is section modulus

Example 7.1 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 7.2 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)

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

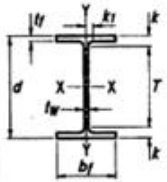


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w	$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k: 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	1 1/16		
×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	21.1	12.3	12 1/4	0.430	7/16	1/4	12.0	12	0.670	11/16	1.27	1 9/16	1 1/16		
×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.[illegible]	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.[illegible]	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.[illegible]	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.[illegible]	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.[illegible]	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.[illegible]	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	19.0	740	116	5.39	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
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	9.00	22.0	597	97.4	5.31	108	195	32.4	3.04	49.2	3.40	11.5	2.94	6540
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

CHAPTER 8
Case Study: Addition To the Royal Ontario Museum (Rom)

CASE STUDY: ADDITION TO THE ROYAL ONTARIO MUSEUM (ROM)

Design Architect: Studio Daniel Libeskind
Local Architects: Bregman and Hamann
Engineers: ARUP London/Halsall Associates
General Contractor: Vanbots Construction Corporation
Steel Fabrication and Erection: Walters Inc.

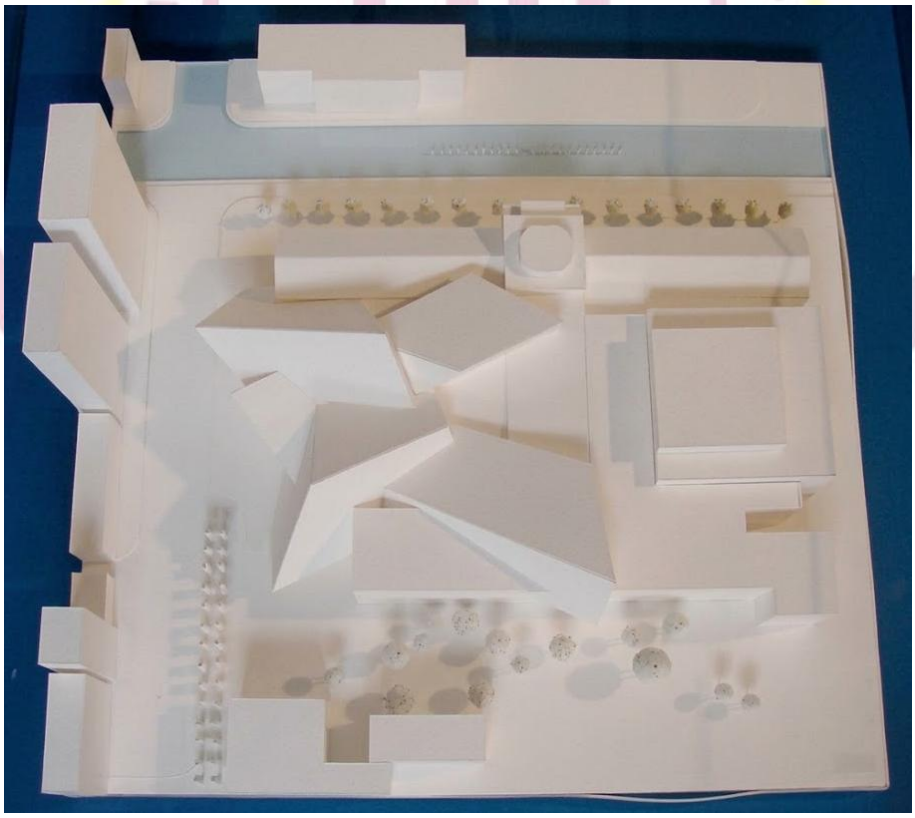
The initial design

The initial design idea for the ROM addition began as a quick and informal "napkin sketch."

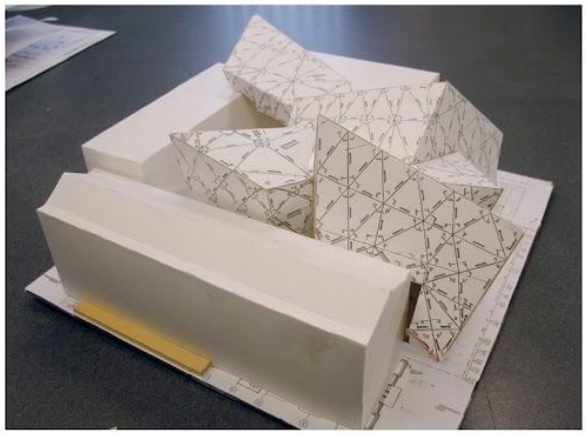
Physical and Digital Models

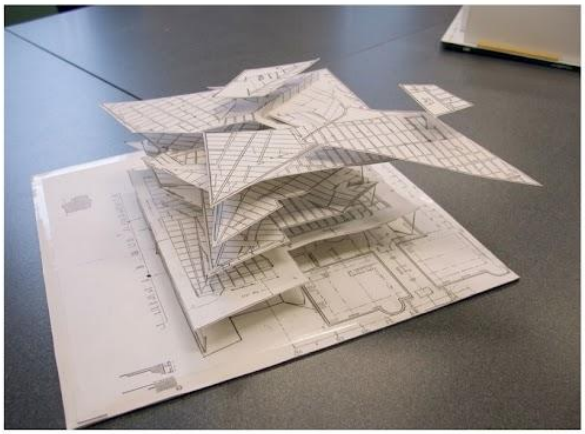
Digital and physical Drawings and models are very important means of communication among team members.

- Models of the traditional finely crafted type complete with interior lights, people and cars used for client-related display purposes.
- Three-dimensional physical models were also used to visualize the **steel structure**, showing the floor framing layers or the steel diagrids.

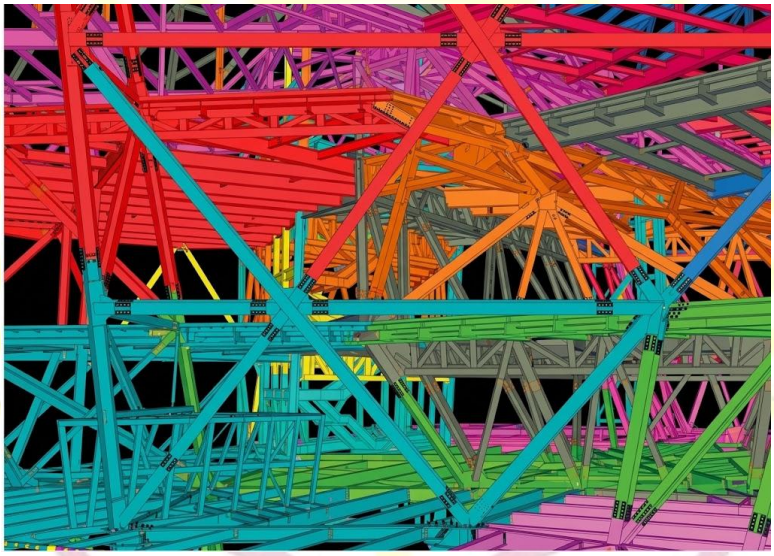


One of the massing study models created by the architects.





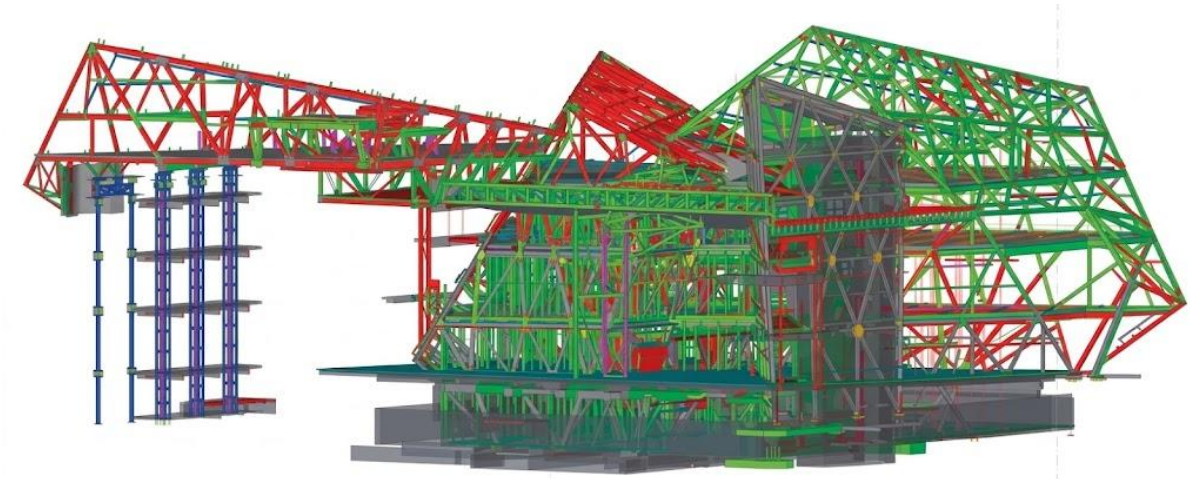
Left: A model used by the fabricator to examine the steel on the faces of the crystals. Right: This paper model uses the drawings for the steel floor framing to create a sense of the "floating" floor plates themselves, as independent of the sloped diagrid walls.



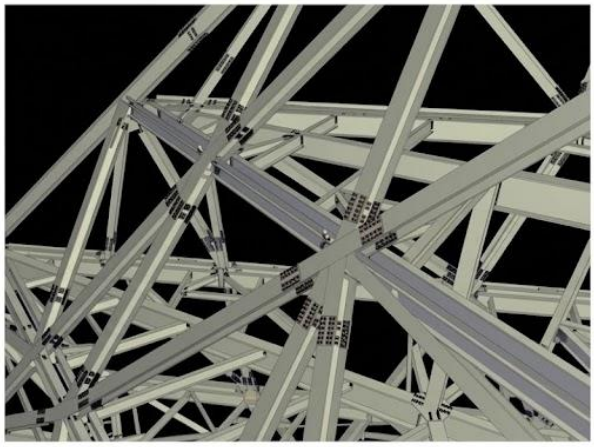
The color coding on this illustration reveals the groups of members that comprise a particular part of the erection sequence.

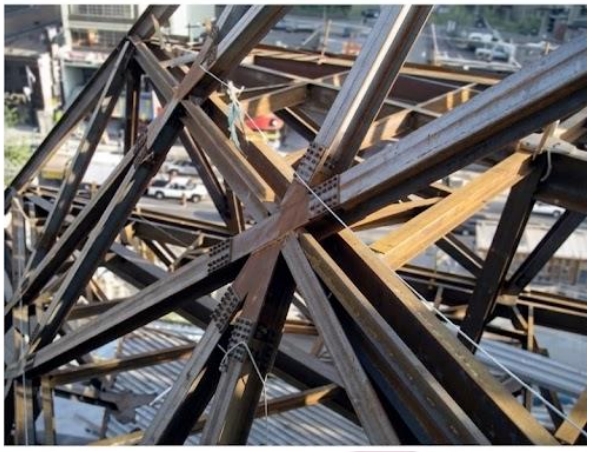
Structural Stability Systems

- The triangulation gives stability to many of the inclined planes.
- Moment connection systems are also used to increase lateral stability, particularly for large truss members or skylight enclosures or cantilevering.



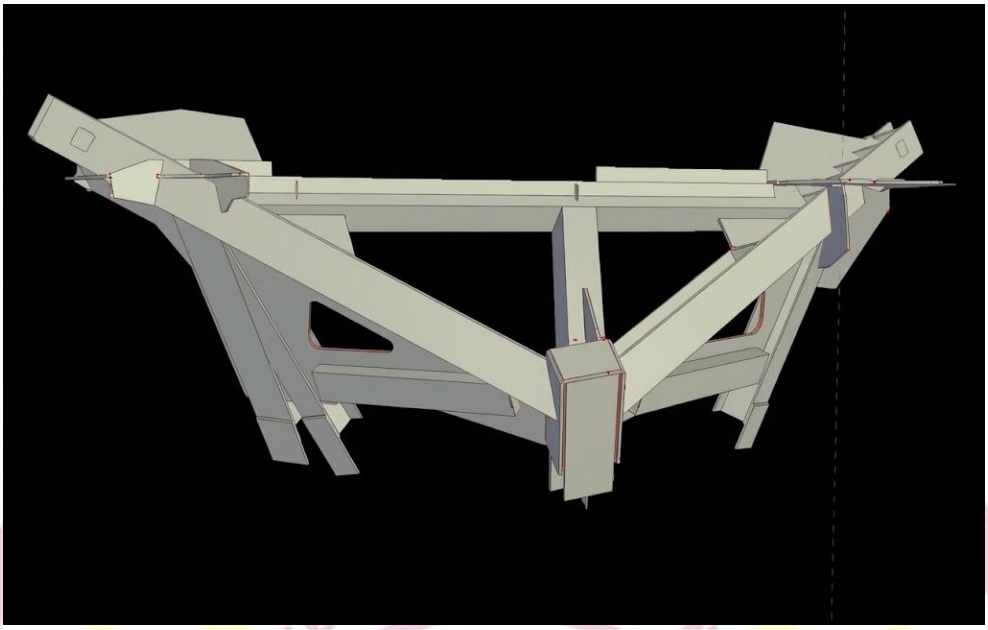
3D modeling software was used of illustrations and showing connection details.



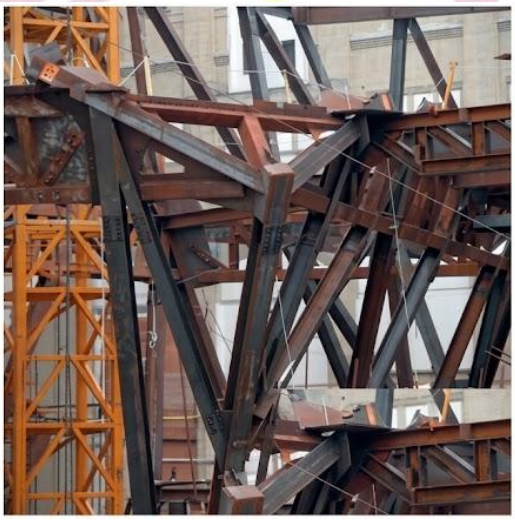


Left: Digital model.
Right: Photograph taken at the same angle as the digital model, showing how precisely the two images align.

Because the ROM's steel structure was too irregular to use a normal grid system, the project team identified steel members by vertex and sometimes a memorable nickname such as **"The Owl."**



- Drawing of the element as extracted from the digital model.
- It was to be shop-fabricated so that the most complex pieces that required welding could be completed in the shop.
- The element was sized to fit on a **flatbed truck for transport**.
- The joints of site connection were to be **bolted**, all bolt holes were drilled in the shop.



Left: "The Owl" nearing completion. All of the smaller steel pieces have been cut based on the drawing sheet.
Right: "The Owl" is in place in the structure on site.

Transportation and Site Issues:

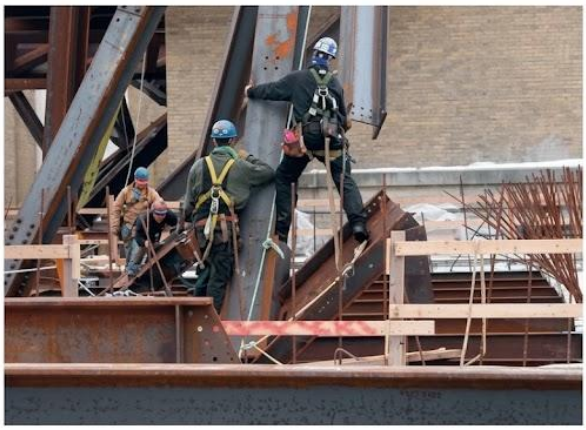
Individual pieces of steel cannot be any larger than can be transported to the site.
The fabricator must know the overhead clearance of every overpass as well as critical turning radii for the carrier for the route of travel between the fabrication shop and the construction site. Timing is also important to avoid peak traffic hours can alleviate some congestion.

Left: Delivery and Storage of Steel Members.
Right: Ironworkers are assembling a number of smaller pieces to create a larger entity to erect.

Erecting the Steel

99% of the erection of the ROM was achieved with the use of a single tower crane located at the center of the project.

The location of the single-tower crane



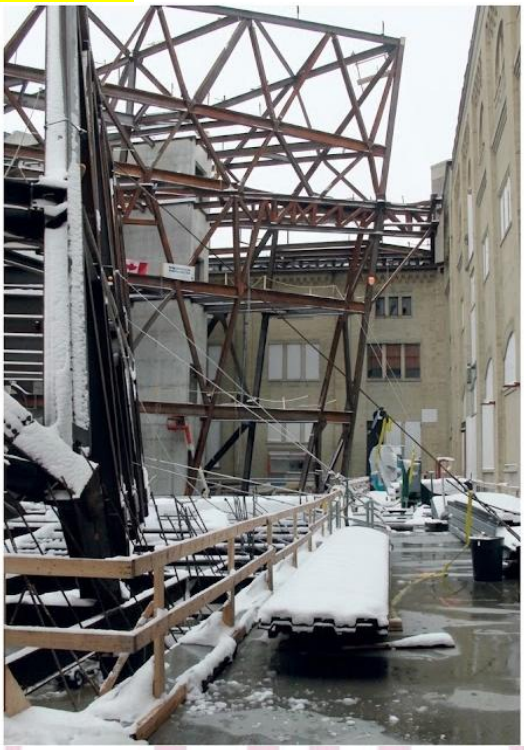
Left: The crane attachment for a truss element
Right: The ironworkers grab hold of the incoming diagrid element.

Left: Ironworkers guide a large steel beam into position.
Right: The lead ironworkers resolving a steel connection issue

In order to use the crane efficiently, not all of the bolts are inserted as the piece is erected, just sufficient to hold it in place. The lead ironworkers are given the task of handling the erection. A second, following crew of ironworkers fill in the remaining bolts and tighten them snug-tight to specification. These men are using a small lift to access the connection.

The Effects of Weather and Climate on Erection

Climate and temperature impact steel erection. Although severely cold or hot temperatures are not as problematic as for a concrete structure, where the curing of the concrete may be negatively affected, steel does expand and contract significantly, which can make fitting the pieces difficult.



Work proceeds on the ROM during the winter months

Providing Permanent Stability for the Frame

There are various ways to provide lateral stability to a steel-framed building (as explained earlier). The early phase of construction began by erecting the steel around the concrete core for the stairs and elevator (see the figure below)

Coordination with Other Systems

The design and detail of the structural steel for the building must also be coordinated with the mechanical and electrical systems.
In traditional buildings these systems are often concealed in a suspended ceiling.
Buildings with unusual layouts will have more difficulty in working.
Building Information Modeling (BIM) systems allow the various members of the team to more easily troubleshoot a conflict-free situation.

Left: Some of the duct runs in the ROM project are massive.
Right: The vertical distribution risers for the air handling system in the ROM had to be worked into the planning of the steel structure.

Left: A preview of the finished building prior to installation of the exhibits.
Right: Exterior view of the completed museum.

CHAPTER 9
Architecturally Exposed Structural Steel (AESS)

Architecturally Exposed Structural Steel (AESS):

Architecturally Exposed Structural Steel (AESS) refers to structural steel that is intentionally left visible as part of the architectural expression of a building.
The connections, welds, bolts, surfaces, and fabrication quality become part of the architectural design.
Steel is therefore both a structural element and an architectural feature.
AESS requires much closer cooperation among:

- **Architect**
- **Structural engineer**
- **Steel fabricator**
- **Steel erector**

Steel Connections

Different projects use different connection strategies depending on their architectural objectives:

1. Bolted connections: can be intentionally exposed to create an industrial, rugged, or technical appearance.
2. Welded connections: can provide a smoother and cleaner appearance.
3. Combination of bolting and welding: allows designers to balance architectural appearance with fabrication and erection requirements.

Viewing Distance and Level of Detailing

The distance from the observer is an important factor in AESS design.

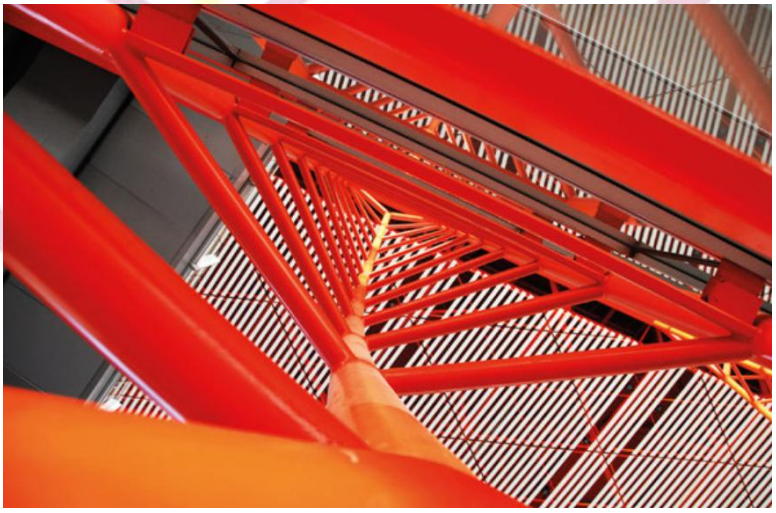
- Steel within approximately 6 m of normal viewing distance may require carefully finished surfaces and highly refined connections.
- Steel located far above the observer or beyond normal viewing distance generally does not require the same level of finishing.
- Appropriate detailing according to viewing distance can produce significant cost savings.

Case Studies

Heathrow Airport Terminal 5 in London, England, is an example of very high-end custom fabrication and detailing. The connections make use of bolting and welding as appropriate to fabrication and erection and allow the process of the assembly to be seen.

The detailing of the Canadian Museum for Human Rights in Winnipeg, MN, Canada, makes use of a combination of custom-fabricated elements and more standard sections and detailing. Bolting is proactively used for its rugged aesthetic as part of the story told by the materiality of this museum.

Beijing Capital International Airport Terminal 3 in Beijing, China, is an example of high-end AESS that looks for a very smooth, all-welded solution. Connections that are more technical in character and might use bolting are hidden above the white slatted suspended ceiling.



Edmonton City Hall Atrium: demonstrates that steel located far above the normal viewing distance does not necessarily require highly refined connections, allowing substantial cost savings.

THE CATEGORY SYSTEM

The categories of AESS 1 to AESS 4 reflect differences in finish and detailing that respect differences in viewing distance, function and cost.

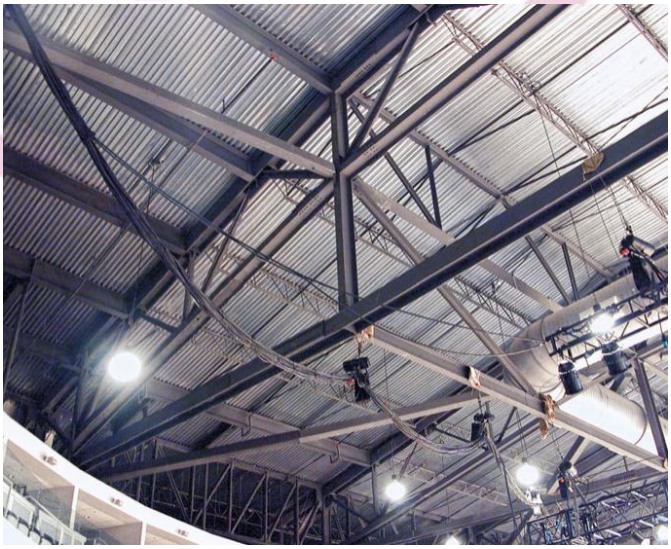
Architecturally Exposed Structural Steel will follow all of the same **structural** requirements and be subject to additional requirements as defined by the assigned AESS category (1, 2, 3, 4 or Custom) and the specific set of characteristics associated with a specific AESS category.

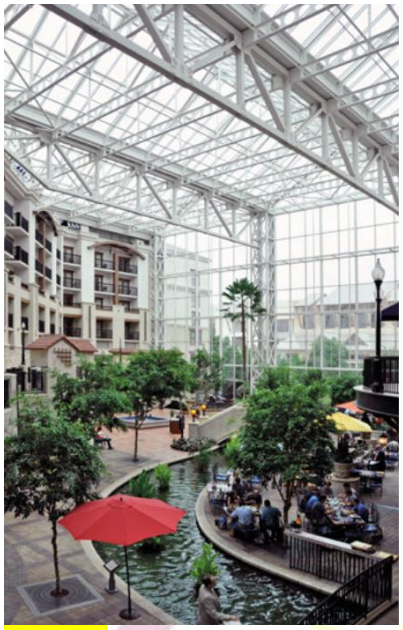
AESS 1 – BASIC ELEMENTS

This type of application would be suitable for "basic" elements that require enhanced workmanship. Exposed structures of this kind could be found in roof trusses for arenas, warehouses, big-box retail stores.

The Ricoh Coliseum in Toronto, ON, Canada, uses trusses for its roof.

A limited level of precision in fabrication would be required, given the building's use.





This steel truss system spans the central atrium of the Gaylord Texan Resort in Dallas, TX, USA.
Its distance to view allows for the use of slightly enhanced workmanship, predominantly standard steel sections and bolted connections.

Lighting must be taken into account when designing with AESS and selecting the appropriate category. The organization and patterning of the steel system is more evident at night than during daytime.
Sunlight and self-shadowing from the structural elements can mask details, whereas the blackness of the night sky in contrast with uplighting can make the structure stand out.

AESS 2 – FEATURE ELEMENTS (with a View Distance > 6m/20ft)

- This category includes structures that are intended to be viewed at a distance larger than 6m/20ft such as the steel in high-level roof or ceiling applications.
- The process requires good fabrication with enhanced treatment of weld, connection and fabrication details.

The trusses are fabricated to AESS 2 standards. They use a combination of W and L-shapes as well as welded and bolted connections. The column supports were fabricated to AESS 3 standards.

The International Terminal at Calgary International Airport in Calgary, AB, Canada, uses AESS 2 standards for the trusses that span the hall.
They are fabricated from a combination of round and square HSS.
The columns and struts are AESS 3 due to their proximity and specific connection detailing.

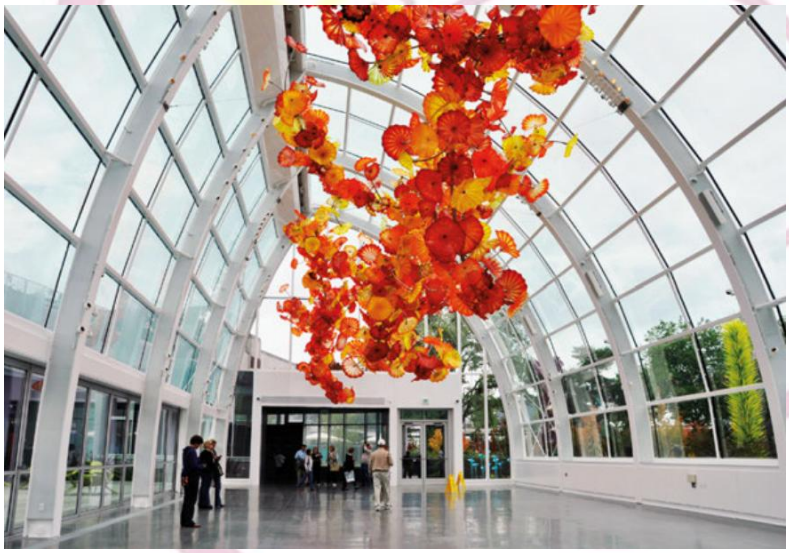
A closer view of the truss shows that a variety of sizes of HSS have been used, dimensioned to suit the loading as well as to allow for ease of welded connections between members.

The triangular arched truss that supports the fabric roof of the Bank of America Pavilion in Boston, MA, USA, would be classed as AESS 2.
The truss segments are bolted together.

AESS 3 – FEATURE ELEMENT with a View Distance ⩽ 6m/20ft

This type of structure can be found in airports, shopping centers, hospitals or lobbies as this structure is frequently be subject to touch by the public, therefore requiring a smoother, more uniform and more durable finish and appearance and the welds should be generally smooth.

Steel detailing at the Palais de Congrès in Montreal, QC, Canada, combines structural requirements with architectural appearance. Larger truss splices use plates and bolts, while the intersection of the diagonal W (wide-flange) section with the bottom chord is ground smooth. Curved plates are carefully cut and welded inside the cutouts to achieve the high-quality workmanship required for AESS 3.

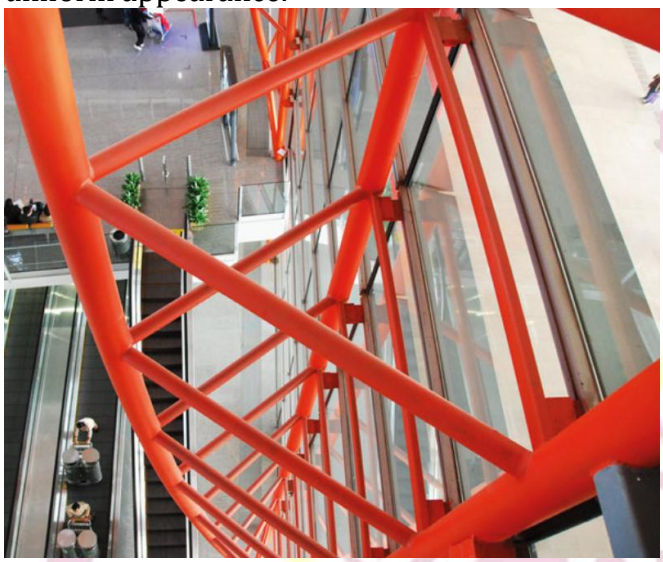


The Chihuly Pavilion in Seattle, WA, USA, has special requirements on the steel due to the continuity of the members from close to distant view.
AESS 3 would be appropriate to govern the choice with deference to close view.
Some savings can be made in the design of the high-level connections that are out of view range.

The pedestrian bridge at the Museum of Flight in Seattle, WA, USA, is served by AESS 3 type steel due to the close view maintained even to the height of the bridge. The added complexity of the curvature and welding of the round HSS "hoops" adds to the cost of this structure.

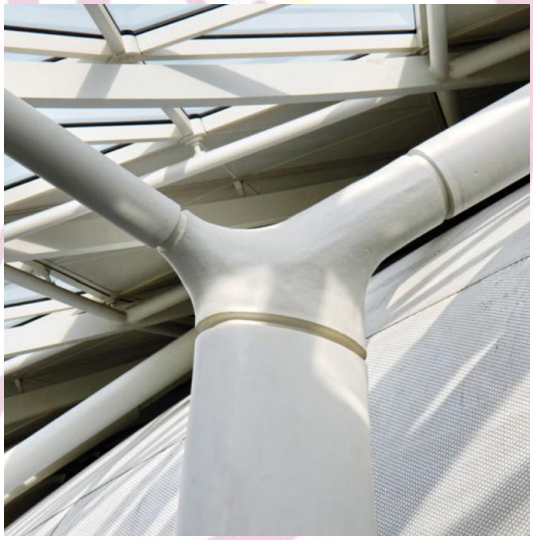
AESS 4 – SHOWCASE ELEMENTS

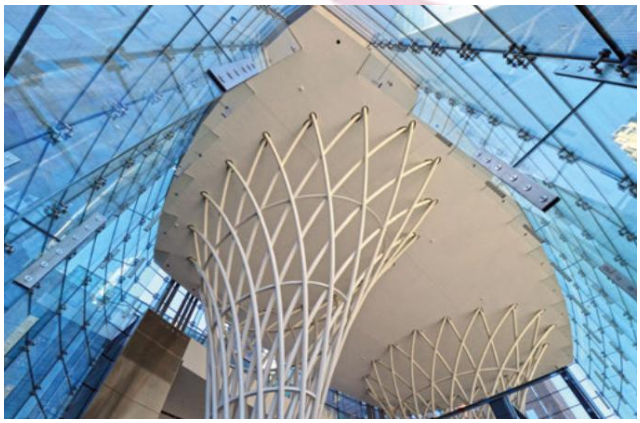
This category represents the highest level of quality and appearance requirements. Where, many columns and spanning members are custom-fabricated. The normal weld seam in the HSS members should not be visible. Welds are carefully contoured and blended, while steel surfaces are filled, ground, and sanded to achieve a smooth, uniform appearance.



Beijing Capital International Airport in Beijing, China, could be considered as AESS 4 in its widespread use of all-welded connections for the curved round HSS trusses that brace its front wall.
The use of curved steel adds to the complexity and cost of the structure.

King's Cross Station in London, England. The use of large custom castings will generally call for AESS 4 standards due to the added challenges in design, fabrication, remediation and erection.





The World Financial Center entry pavilion uses custom-bent steel to create this pair of diagrid column supports. The connections are all welded

AESS C – CUSTOM ELEMENTS

There are certain applications of AESS that will require a customized approach to design and specification. With increases in the reuse of steel for sustainably-minded projects, unique sets of criteria will come into play. Requirements will address the presence of finishes, corrosion, inconsistencies between members.