Al-Mansour University College

قسم الهندسة المدنية

Civil Eng. Dept.

المرحلة الرابعة

4th. Stage

Structural Steel Design

2022 - 2023

Lec.1

تصميم حديد

Lec. Hayder Qais

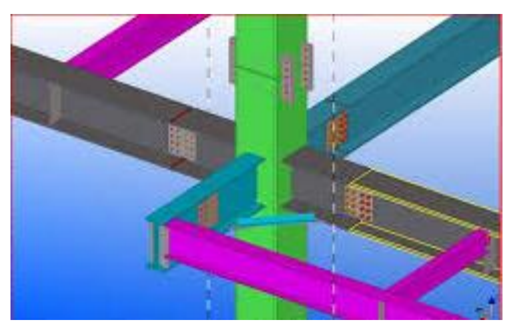
Structural Steel Design

Al Mansour University College

Civil Engineering Department

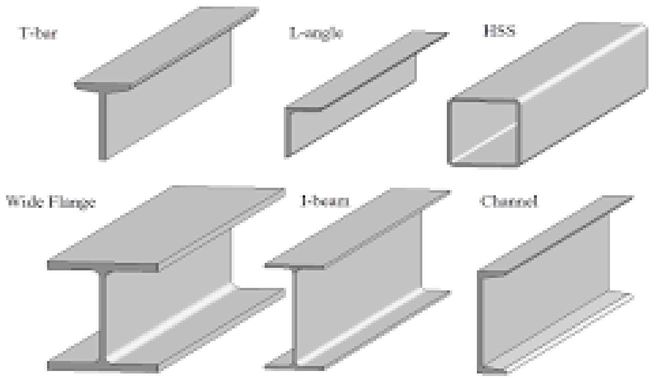
4^{th} year

Ass. Lect. Haider Qais



Contents

Chapter 1
Introduction of Steel Structure



STRUCTURAL DESIGN:

Structural design of buildings

- Structural concrete Design shall conform to ACI code.
- Structural steel design and fabrication shall conform to AISC manual

❖ **Types of steel structures:**

1- Buildings
- public buildings
- industrial buildings
- residential buildings

2- Bridges
- rail roads
- mono rail (overground)
- highwys & pedestrins

3- Other structures
- power trnsmision towers, for
- radars & t.v installiations
- water supply tanks.
- ships & air planes.

Steel material types:

- Carbon steel [0.2% to 1.5% carbon] + Fe (A36, A53, A500, A501, A529, A570), ideal type for structural applications because of cheap production cost and high ductility

- Low carbon steel [~ 0.2% carbon] uses: sheets, wires, pipes, rebar

- Mild carbon steel [0.3% to 0.7% carbon] uses: rails, boilers, plates, axles, structures.

- High Carbon Steel [0.7% to 1.5% carbon] Uses: Surgical instruments, razor blades, cutlery, spring, construction.

Advantages of steel as structural material:

1. High strength:
   - The high strength of steel per unit of weight means that the weight of structures will be small warning before failure take place.
2. Toughness
   - Steel has both strength and ductility.
   - A steel member loaded until it has larger deformations will still be able to withstand large forces. For example, steel member can be

subjected to larger deformation during fabrication without fracture (steel member can be bend, hammered, sheared, and holes punched)

Disadvantages of steel as a structural material:

1. Maintenance cost

    - Steel structures are disposed to corrosion when exposed to air, water, and humidity. They must be painted periodically.

2. Fireproof cost

    - The strength of steel members is reduced at high temperatures.

3. Buckling

    - For most structures the use of steel column bjected to cyclic loading.

Some types of steel structures:

Steel structures can be divided into three general categories:

1. Framed structures: for example (multistory buildings and bridges) Framed structures usually consist of beams, columns, and diagonal

bracing to provide stability.

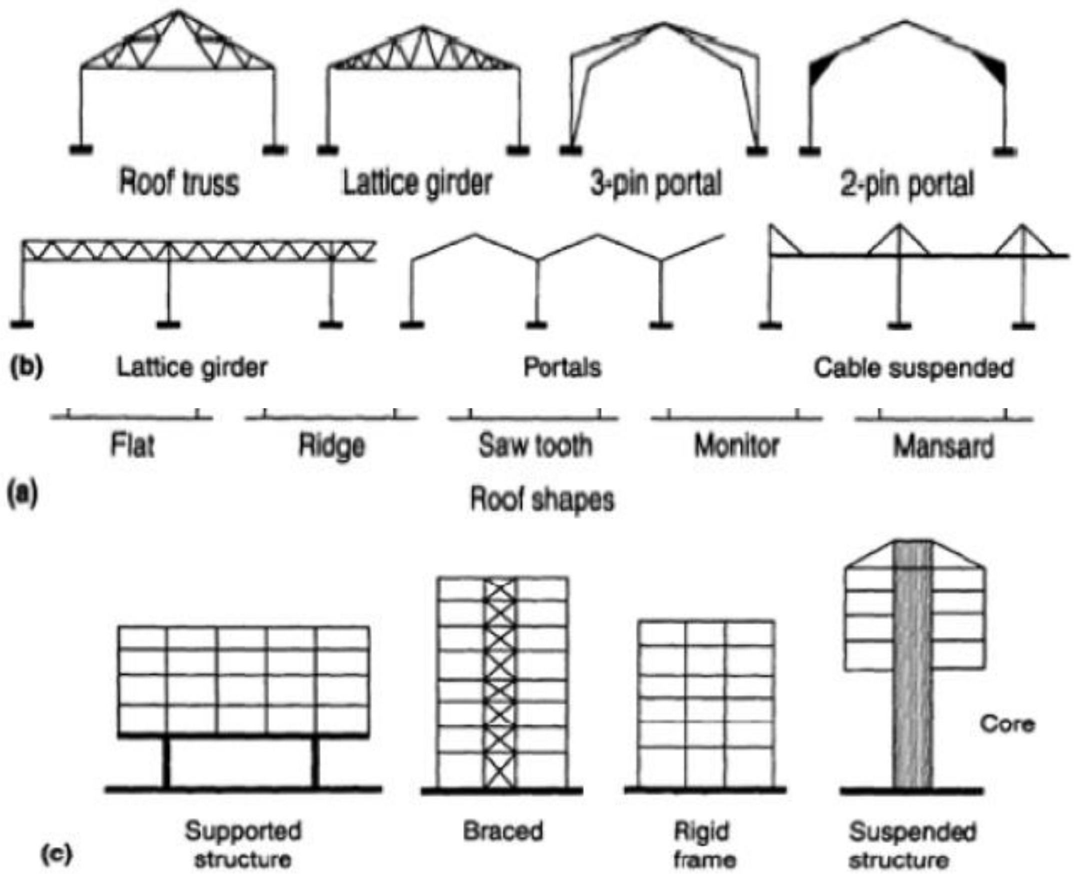
2. Shell structures: for example water tanks

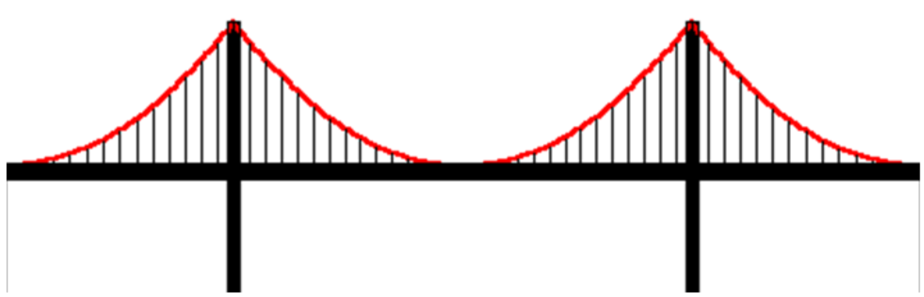
3. Suspension structures: for example (suspension bridges, and cable state bridges

Column

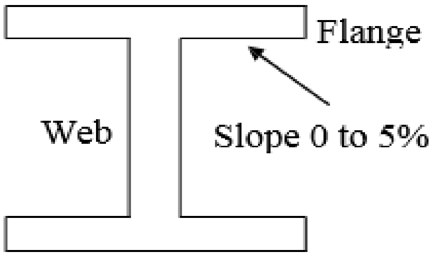
Bracing

Beam

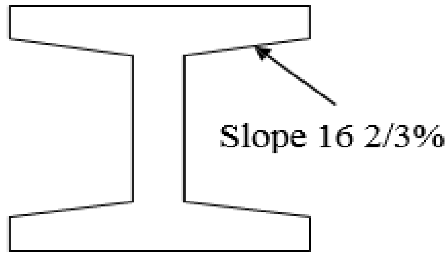




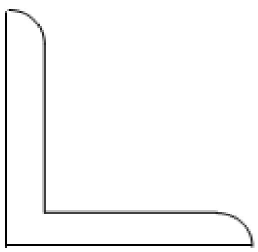
Steel sections:



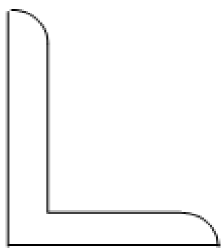
W section
Wide flange beam



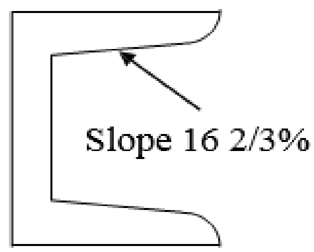
S section
American Standard beam



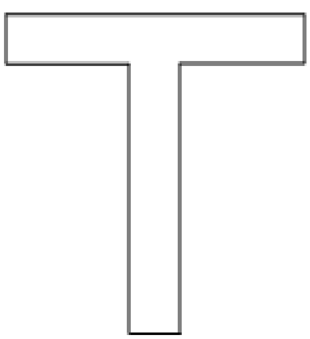
Equal leg angle



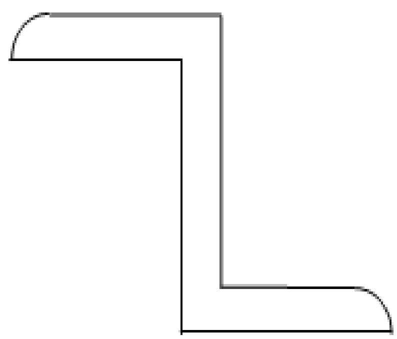
Unequal leg angle



Channel



Tee



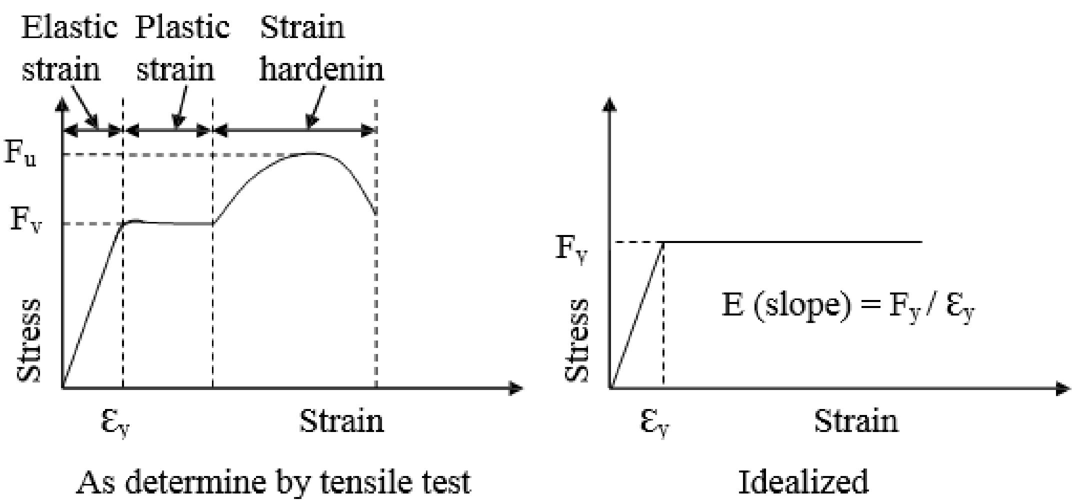
Zee

Notes:

W_{27x117}: is a W section approximately 27 in deep weighing 117 Ib/ft.

$L_{6x6x1/2}$: is an equal leg angle, each leg is 6 in long and ½ in thick.

Stress – strain relationship in structural steel:



F_u: Ultimate Stress, F_y: Yield Stress

Stress = P / A, Strain = ∆L / L

- If a piece of ductile structural steel is subjected to a tensile force, it will begin to elongate. If the tensile force i te rate without a corresponding increase in the stress

- Structural steel is usually grouped into several major ASTM classifications. As shown in the table (2-3 and 2-4) in the AISC Manual.

Loads:

- The most important and most difficult task forced by the structural engineer is the accurate estimation of the loads that may be applied to a structure during its life.

- The next important task is to determine the worst possible combinations of these loads that might occur at one time.

- The objective of a structural engineer is to design a structure that will be able to withstand all the loads to which it is subjected while serving its purpose throughout its intended life span.

- Loads can be classified into three categories: dead loads, live loads, and environmental loads (e.g. Snow loads, Rain loads, Wind loads, and Earthquake loads)

TYPES OF LOADS:

- Dead loads.
- Live Loads.
- Wind Loads.
- Impact Loads.
- Fatigue.
- Earthquake Loads.
- Snow Loads.
- Other Loads.

Dead Loads: are loads of constant magnitude that remain in one position. The weight of the structure (beams, columns, slabs, wall, finishing, plastering etc) is considered dead load as well as attachments to the structure such as: pipes, air-conditioning, heating ducts, roof and floor covering, etc.

- Weight of people.
- Furniture.
- Machinery & goods.
- Dynamic forces resulting from moving loads.

- Wind loads.
- Forces resulting from temperature change.
- Pressure of liquids.
- Earthquakes.

Type of Building		LL (psf)
Apartment houses	Apartments	40
	Public rooms	100
Dining rooms and restaurants		100
Garages (passenger cars only)		50
Gymnasiums, main floors, and balconies		100
Office buildings	Lobbies	100
	Offices	50
Schools	Classrooms	40
	Corridors first floor	100
	Corridors above first flo	80
Storage warehouses	Light	125
	Heavy	250
Stores (retail)	First floor	100
	Other floors	75

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

Wind load: are horizontal loads on the building which are exerted on the surface area of the building on windward side. This load is calculated based on the wind zone which provides the maximum wind speed in the given zone. This can be obtained from the wind map of the location. This wind speed is converted into force based on the surface area and orientation of building. wind direction. Shape of the building is or structural member is also considered for calculation.

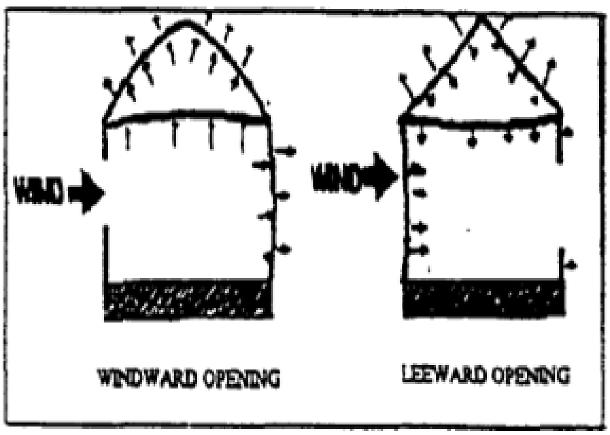
- For building with sloping roofs; aero dynamic effect must be consider as follow

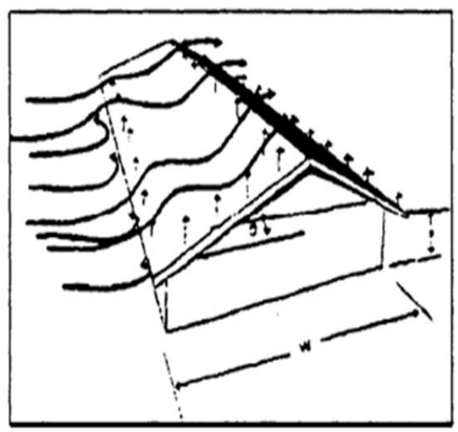
R□□□□R□□□□R□□1.2□□ □D□R□□□□R□□

- Wind pressure can be approximately calculated as

□□□. □2□□²□

Where; (v) is speed of air in mile / hour. (q) is pressure per unit area





Impact load: The term impact refers to the dynamic effect of a suddenly applied load (e.g. trucks, vehicles)

Acceptable methods for designing structural steel member:

- The AISC specification two acceptable methods for designing steel structural members and their connections, which are **Load and Resistance Factor Design (LRFD)** and **Allowable Strength Design (ASD)**.

- Both procedures are based on limit state design principles. The term limit state is used to describe a condition at which a structure or part of a structure ceases to perform its gth of a member is its calculated theoretical strength with no resistance factor (Ø Phi) or safety factor (Ω Omega) applied. A resistance factor usually less than 1.0 is multiplied by the nominal strength of a member to account for variation in material strength, member dimensions and workmanship.

- With both the LRFD and ASD procedures values of the individual loads (dead, live, wind, snow, etc.) are first estimated. These loads are referred to as service or working loads.
- With the LRFD method, possible service load groups are formed and each service load is multiplied by a load factor, normally larger than
- 1.0. The result linear combination of service loads in a group, each multiplied by its respective load factor, is called factored load.

Load calculations (<u>Computing combined loads)</u>

LRFD

R_u or U = 1.4D
R_u or U = 1.2D + 1.6L + 0.5 (Lr or S or R)
R_u or U = 1.2D + 1.6(Lr or S or R) + (0.5L or 0.8W)
R_u or U = 1.2D + 1.6W + 0.5L + 0.5 (Lr or S or R)
R_u or U = 1.2D ± 1.0E+ 0.5 L + 0.2S
R_u or U = 0.9D ± (1.6W or 1.0E)

$$\mathbf{R_u \text{ or } U \leq \emptyset * R_n}$$

ASD

R_u or U = D
R_u or U = D + L
R_u or U = D + (Lr or S or R)
R_u or U = D + 0.75L + 0.75 (Lr or S or R)
R_u or U = D ± (W or 0.7E)
R_u or U = D ± 0.75(W or 0.7E) + 0.75 L + 0.75 (Lr or S or R)
R_u or U = 0.6D ± (W or 0.7E)

$$R_u \text{ or } U \leq^{*} Rn/\ \Omega$$

Where:
U = the design or ultimate load
D = dead load
L = live load due to occupancy
Lr = roof live load
S = snow load
R = nominal load due to initial rainwater or ice
W = wind load
E = Earthquake load

Specifications for design of steel structures:
Many available standards for properties & specifications of steel sections may be followed in design & constructions are published by some institutes & associations such as:

- AISC: American institute of steel construction.
- AASHTO: American associat erican Society for Testing& Material.
- NBC: National Building Code.
- UBC: Uniform Building Code.
- BS: British Standard.
- DIN: Dutch International Norma.
- FIP: French International
- EURO: Euro Code.

American Institute of Steel Construction (AISC), Inc. publishes the AISC Manual of Steel Construction (Steel construction manual, or SCM), which is currently in its 13th edition. Structural engineers use this manual in analyzing, and designing various steel structures. Some of the chapters of the book are as follows:

Part1- Dimensions and properties of various types of steel sections available on the market (W, S, C, WT, HSS, etc.).

Part 2- General design considerations.

Part 3- Design of flexural members.

Part 4- Design of compression members.

Part 5- Design of tension members.

Part 6- Design of members subject to combined loading.

Part 7- Design consideration for bolts.

Part 8- Design considerations for welds.

Part 9- Design of connecting elements.

Part 10- Design of simple shear connections.

Part 11- Design of flexure moment connections.

Part 12- Design of fully restrained (FR) m Design of beam bearing plates, column base plates, anchor rods, and column splices.

Part 15- Design of hanger connections, bracket plates, and crane-rail connections.

Part 16- General nomenclature.

Part 17- Specifications and codes.

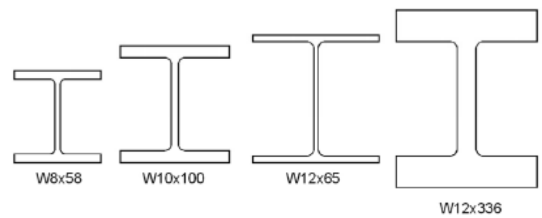
-Commentary on specifications and codes.

-Miscellaneous data and mathematical information.

Steel W-sections for beams and columns

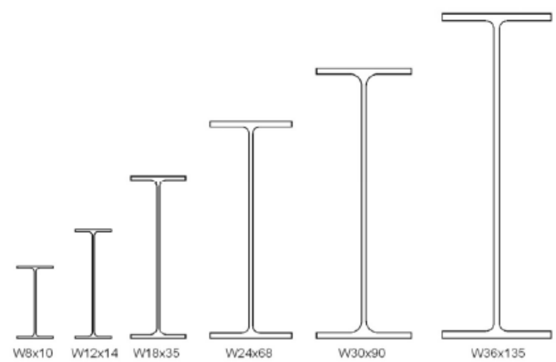
Columns:

Closer to square
Thicker web & flange



Beams:

Deeper sections
Flange thicker than web

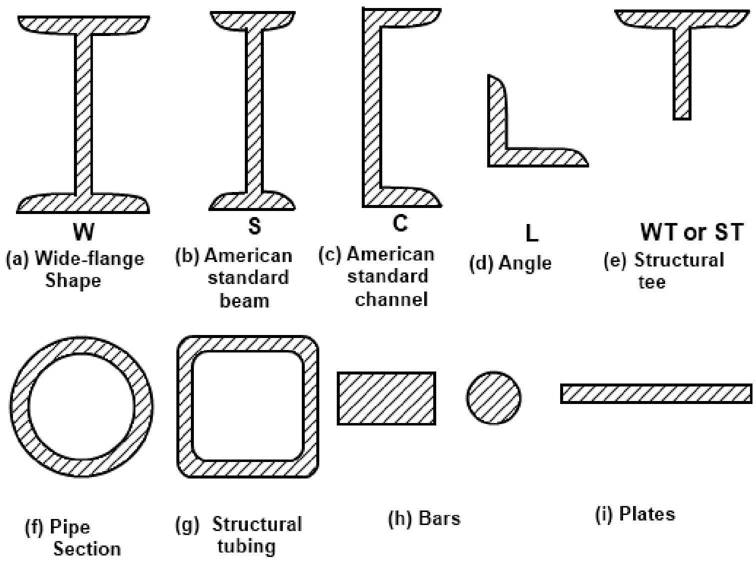


<u>Types of structural steel elements:</u>
According to product method, we have two types as follow:

1- Standard hot rolled steel shapes:

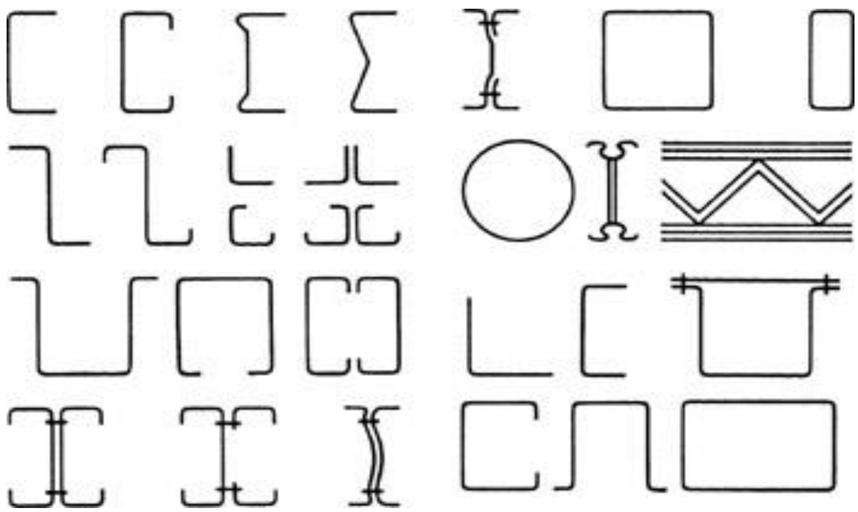
They are formed from hot billet steel by passing through rolls numerous times to obtain the shape required. The steel manual refer to the hot rolled steel shapes as follow:

- Wide flange steel shape referred by WF as W 27x117. This steel shape is the most common structural steel section.
- Standard steel shape referred by S as S 12x30. This has a thick web & narrow flange compared to W steel shapes.
- Standard channels referred by C as C 10x30 or MC 18x58.The second type referee to miscellaneous channels shapes whic is T-shape cut from W shape & ST is T-shape cut from S shape.
- Angles refereed by L as L 4x4x1/2 & they are categorized as equal & non equals legs angles.



2- Cold formed steel shapes:

They are obtained from plates & some of bars having a steel shapes is used for furniture & some of nonstructural works as cladding of gable frames & roof trusses (purlins & side rail, etc.).



<u>Grade of steel:</u>

Refer to AISC-SM Table 2-3 (page 2-39) for a list of appropriate structural steel ASTM designations for various structural shap designations are commonly used:

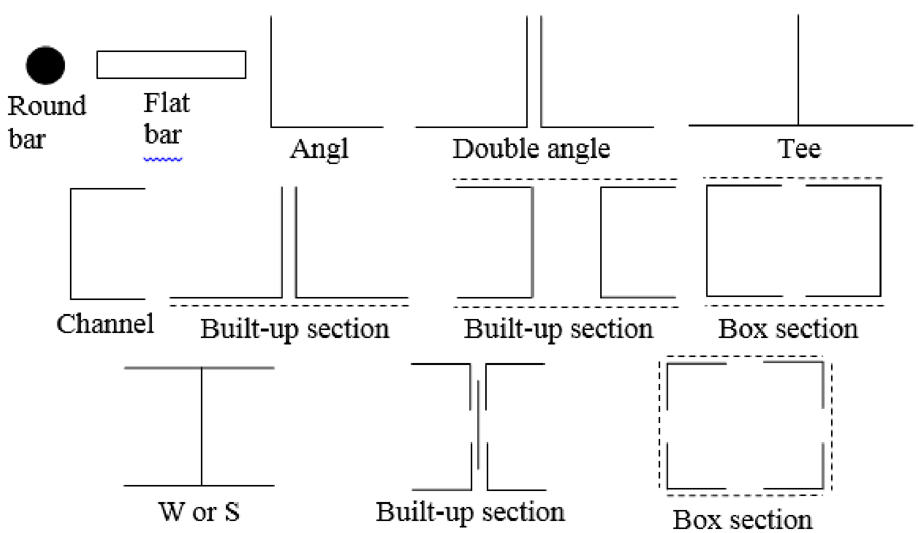
ASTM Designation	**F_y (KSI)**	**F_u (KSI)**	**Applicable structural shapes**
A36	36	58	M, S, C, MC, L, plates
A572	50	65	HP
A992	50	65	W
A53	35	60	Pipe
A500, Gr. B	42	58	HSS round
A500, Gr. B	46	58	HSS rectangular
A588	50	70	Corrosion-resistant for all rolled shapes

Chapter 2
Analysis of Tension Members

Tension members are structural elements that subjected to axial tensile forces, such as:

- Members in trusses
- Cables in cable-stayed and suspension bridges
- Bracing in frames to resist lateral forces from blast, wind, and earthquake.

In the early days of steel structures, tension members consisted of rods, bars, and perhaps cables. Today tension members usually consist of single angles, double angles, tees, channels, W sections, or suctions built up from plates or rolled shapes. Another type of tension section often us , which is very satisfactory for use in transmission towers, signs, foot bridges.



Nominal strength of tension member:

- A ductile steel member without holes and subjected to a tensile load can resist without fracture a load larger than its gross-sectional area times its yielding stress.
- If, on the other hand, we have a tension member with sive elongation of the member)
  $P_n = F_y * A_g$ (AISC equation D2-1)

  $\varnothing_t P_n = \varnothing_t * F_y * A_g$ ($\varnothing_t$=0.9 for the LRFD and Ω_t = 1.67 for ASD)

2. For tensile rupture in the net section, as where bolt or rivet holes are present
   $P_n = F_u * A_e$ (AISC equation D2-2)
   $\varnothing_t P_n = \varnothing_t * F_u * A_e$ ($\varnothing_t$=0.75 for the LRFD and Ω_t = 2 for ASD)

Where:

- A_g = gross area of member, in^2 (mm^2)
- A_e = effective net area, in^2 (mm^2)
  - For Bolted connections: Ae = U x An
  - For welded connection: Ae = U x Ag
- F_y = specified minimum yield stress, ksi (MPa)
- F_u = specified minimum tensile strength, ksi (MPa)

- Values of F_y and F_u are provided in table 2-3 and 2-4 in the AISC manual for the ASTM structural steels on the market today

Gross and Net area determination:

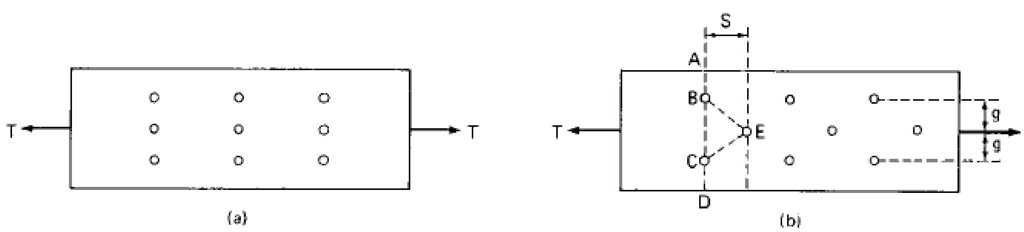
Gross area: the gross area of a member is the total cross-sectional area.

Net area: the net area of a member is the products of the thickness and the net width of each element computed as follows:

1. In computing net area for tension and shear, the wid ing, for each gage space in the chain, the quantity

$$\mathbf{S^2 /4g}$$

- S = longitudinal center to center spacing (pitch) of any two consecutive holes, in.
- g = transverse center to center spacing (gage) between fastener gage lines, in.

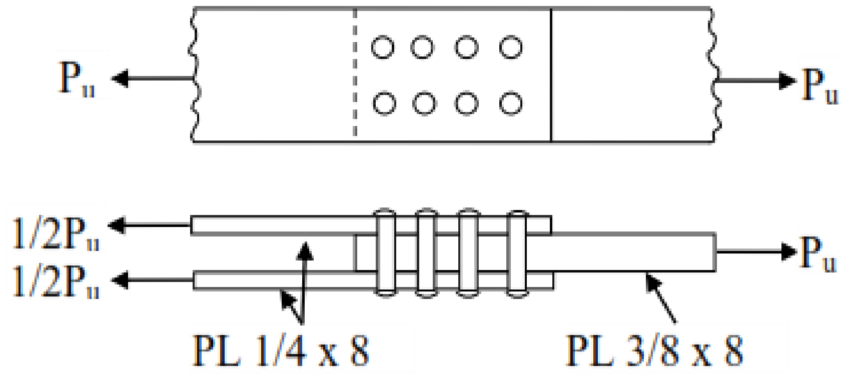


3. In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.
4. Section J4.1 (b) limits A_n to a maximum of 0.85 A_g for splice plates

R – Σ D 1 ² .

❖ <u>Example 1:</u>

Determine the net area of the (3/8 x 8- in) plate shown in the figure below. The plate is connected at its end with two lines of (3/4 - in) standard bolts.



Solution

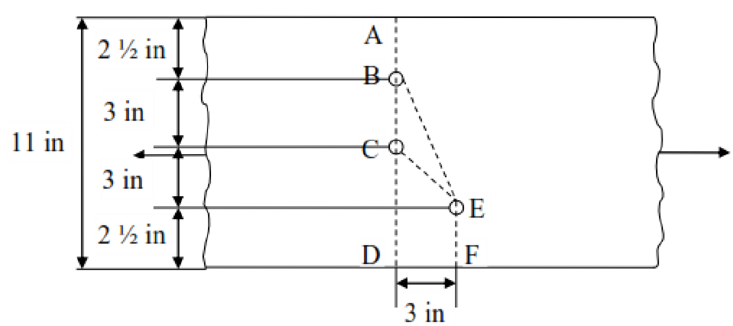
$A_g = \frac{3}{8} * 8 = 3 \text{ in}^2$

$A_n = 3\text{-}2*(\frac{13}{16} + \frac{1}{16}) * \frac{3}{8} = 2.34375$

$= 2.34 \text{ in}^2$

❖ Example 2:

Determine the critical net area of the (1/2 in) thick plate shown in the figure below. The standard holes are punched for (3/4 in) bolts.



Solution

$A_g = 1/2 * 11 = 5.5 \text{in}^2$

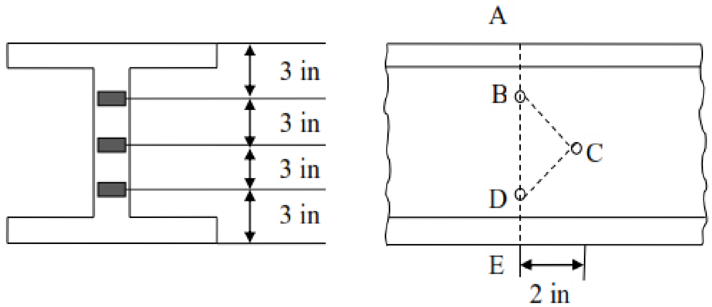
$A_n \text{ (ABCD)} = 5.5\text{-}2*(\frac{13}{16} + \frac{1}{16}) * \frac{1}{2} = 4.625 \text{ in}^2$

$A_n \text{ (ABEF)} = 5.5\text{-}2*(\frac{13}{16} + \frac{1}{16}) * \frac{1}{2} + \frac{3^2}{4*6} * \frac{1}{2} = 4.8125 \text{ in}^2$

$A_n \text{ (ABCEF)} = 5.5\text{-}3*(\frac{13}{16} + \frac{1}{16}) * \frac{1}{2} + \frac{3^2}{4*3} * \frac{1}{2} = 4.5625 \text{in}^2$ (controls)

❖ Example 3:

Determine the net area of the W12x16 shown in the figure below. Assume that the standard holes are for 1 in bolts.



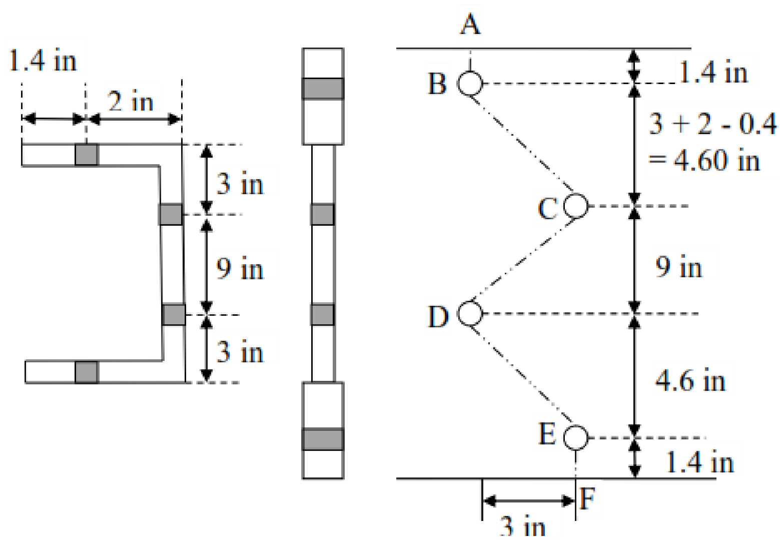
Solution

A_n (ABDE) = $4.71-2*(1\frac{1}{16}+\frac{1}{16})*0.22$=4.215 in^2

A_n (ABCDE) = $4.71-2*(1\frac{1}{16}+\frac{1}{16})*0.22+2*\frac{2^2}{4*3}*0.22$ =4.1142 in^2 (controls)

❖ Example 4:

Determine the net area along route ABCDEF for the C 15x33.9 shown in the figure below. Assume that the standard holes are for 3/4 in bolts.



Solution

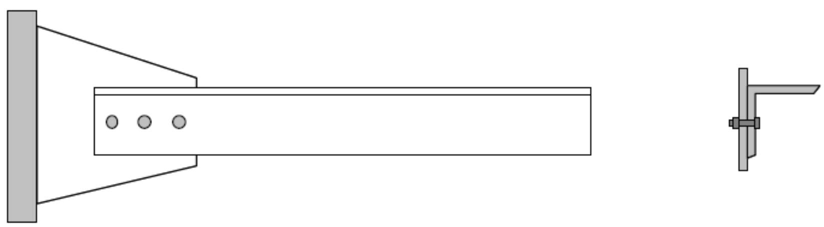
From AISC manual For C 15x33.9: A_g = 10 in^2, t_w = 0.4 in, t_f = 0.65 in

A_n (ABCDEF) = $10-2*(\frac{13}{16}+\frac{1}{16})*0.65+2*\left(\frac{13}{16}+\frac{1}{16}\right)*0.4+\ 2*\frac{3^2}{4*9}*$

$0.4+\ 2*\frac{3^2}{4*4.6}*\frac{10.65+0.4}{2}$ =8.77 in^2

Effective net area:

1. When a member other than a flat plate or bar is loaded in axial tension until failure occurs across its net section, its actual tensile failure stress will probably be less that the tensile strength of the steel, unless all of the various elements which make up the section are connected so that stress is transferred uniformly across the section
2.



To account for the non-uniformity, the effective net area of tension member shall be determined as follows:

$A_e = A_n$ U (AISC equation D3-1)

U = the shear leg factor, is determined as shown in table D3.1

3. The above equation logically applies for both fastener connections having holes and for welded connections.
4. For welded connections, the net area equal the gross area since there are no hole
5. Whenever the tensile load is transmitted by bolts, rivets, or welds through some but not all of the cross-sectional elements of the members, the load carrying efficiency is reduced and U will be less than (1)

- For bolted member: the following equation can be used to estimate the shear leg factor or reduction coefficient U

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

= Distance from the plain of the connection to the centroid of the area of the whole section

- In order to calculate U for W section connected by its flange only, we will assume that the section is split into two structural tees. Then the will be the distance from the outside edge of the flange to the C.G of the structure Tee
- For welded members: when tension loads are transferred by welds, the rules from AISC table D3.1 that are to be used to determine values of A and U are as follows

  1. Should the load be transmitted only by longitudinal welds to other than a plate member, or by longitudinal welds in combination with transverse welds, A is to equal the gross area of the member Ag.
  2. Should a tension load be transmitted only by transverse welds, A is to equal the area of the directly connected elements U is to equal 1.0 (case 3 in Table D3.1)
  3. Test has shown that when flat plates or bars connected by longitudinal fillet welds are used as tensio n members, they may fail prematurely by shear lag at the corners if the welds are too far apart. Therefore, the AISC specification states that when such situations are e ective net area. For such situations the value of U to be used (case 4 in Table D3.1) are as follows:

When $l \geq 2w$ (U = 1.0)

When $2w \geq l \geq 1.5w$ (U = 0.87)

When $1.5w \geq l \geq w$ (U = 0.75)

Where

l = weld length, in

w = plat width (distance between weld), in

Connecting elements for tension members: when splice or gusset plates are used as statically loaded tensile connecting elements, their strength shall be determined as follows

1. For tensile yielding of connecting elements

$$R_n = F_y A_g \;\; (AISC\ Equation\ J4-1)$$

$$\emptyset_t R_n = \emptyset_t F_y A_g \; (\emptyset_t = 0.9)$$

2. For tensile rupture of connecting elements

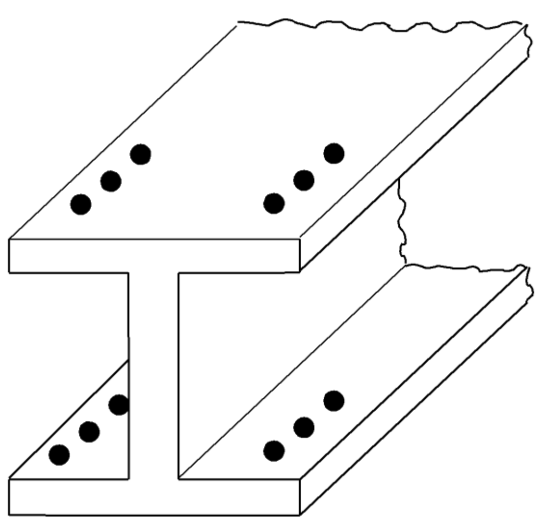
$$R_n = F_u A_e \;\; (AISC\ Equation\ J4-2)$$

$$\emptyset_t R_n = \emptyset_t F_u A_e \; (\emptyset_t = 0.75)$$

3. The net area $A = A_n$, to be used may not exceed 85 percent of A_g.

❖ Example 5:

Determine the LRFD tensile design strength for a W 10x45 with two lines of (3/4in diameter) bolts in each flange using A572 Grade 50 steel and the AISC specification. There are assumed to be at least three bolts in each line (4in on center), and the bolts are not staggered with respect to each other.



Solution

From table 2-6 of AISC manual F_y = 50 ksi, F_u = 65 ksi for A572 Grade 50 steel. From table 1-1 (W shapes Dimensions): for W 10x45 (Ag = 13.3 in2, d = 10.1 in, b_f = 8.02 in, t_f = 0.62 in)

- Cross sectional yielding
  $P_n = F_y A_g$
  $P_n = 50 * 13.3 = 665k$
  $\varnothing_t P_n = \varnothing_t * F_y * A_g$
  $0.9*665 = 598.5k$

- Tensile rupture strength

$A_n = 13.3 - 4* (\frac{13}{16} + \frac{1}{16}) * 0.062 = 11.13 \text{ in}^2$

From table 1-8 (WT shapes Dimensions): for WT 5x22.5 (which is half of a W10x45)   = 0.907

L = 4 + 4 = 8in

U = 1-   /L

= 1-0.907/8 = 0.8866

b_f = 8.02in > (2/3)d =(2/3)10.1 = 6.7333in

U = 0.9 from table (D3-1 case 7) ….. control

$A_e = UA_n$

$= 0.9 * 11.13 = 10.017 \text{in}^2$

$P_n = F_u A_e$

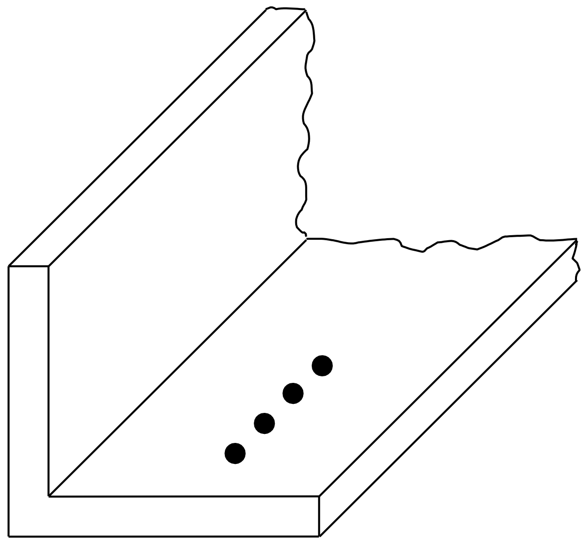
$= 65 * 10.017 = 651.105k$

$\varnothing_t P_n = \varnothing_t * F_u * A_e$

$= 0.75 * 651.105 = 488.3588k$ ….. (Control)

❖

connected at its ends with one line of four (7/8in) diameter bolts in standard holes (3in) on center in one leg of the angle.



<u>Solution</u>

From table 2-6: F_y =36 ksi, F_u = 58 ksi for A36 steel.
From table 1-7 (L shapes dimensions): $A_g = 4.38in^2$, $=$ $= 1.62in$

- Cross sectional yielding
  $P_n = F_y A_g$
  $P_n = 36 * 4.38 = 157.68k$
  $\varnothing_t P_n = \varnothing_t * F_y * A_g$
  0.9*157.68 = 141.912k ….. (control)

- Tensile rupture strength

$A_n = 4.38 - (\frac{15}{16} + \frac{1}{16}) * \frac{3}{8} = 4.005\ in^2$

L 3+3+3 = 9in

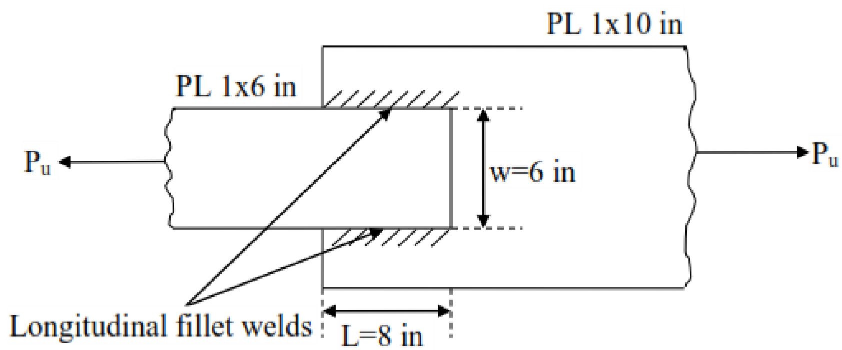
U = 1- /L
 = 1- 1.62/9 = 0.82 (control)

$U = 0.8\ (from\ table\ D3 - 1\ case\ 8)$

$A_e = UA_n$
 $= 0.82 * 4.005 = 3.2841in^2$
$P_n = F_u A_e$
 = 58 * 3.2841= 190.4778 k
$\varnothing_t P_n = \varnothing_t * F_u * A_e$
 = 0.75 * 190.4778 = 142.8584

❖ Example 7

The 1x6 in plate shown in the figure below is connected to a 1x10 in plate with longitudinal fillet welds to transfer a tensile load. Determine the LRFD tensile design strength of the member if Fy = 50 ksi and Fu = 65 ksi.



Solution

- Cross sectional yielding

  $P_n = F_y A_g$

  $P_n = 50 * 1 * 6 = 300$ k

  $\varnothing_t P_n = \varnothing_t * F_y * A_g$

  $0.9*300 = 270$ k

- Tensile rupture strength

$A_n = A_g = 1 * 6 = 6\ in^2$

$1.5w = 1.5 * 6 = 9in$

$1.5w > L > w$

$9 > 8 > 6$

U = 0.75 (from table D 3-1 case 4)

$A_e = UA_n = 0.75 *6 = 4.5\ in^2$

$P_n = F_u\ A_e$

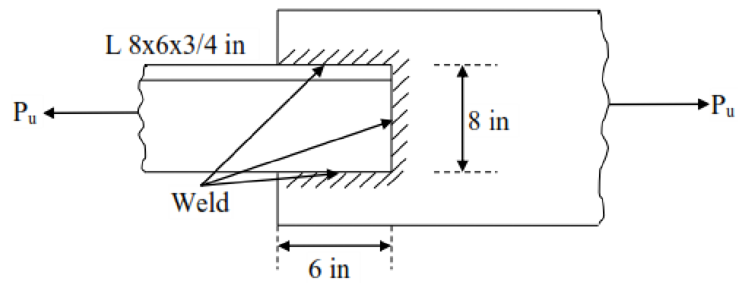
$= 65 * 4.5 = 292.5$ k

$\varnothing_t P_n = \varnothing_t * F_u * A_e$

$= 0.75 * 292.5 = 219.375$….. (Control)

❖ <u>Example 8</u>

Compute the LRFD design strength of the angle (8x6x3/4 in) shown in the figure below. It is welded on the ends and sides of the 8 in leg only. Fy = 50 ksi and Fu = 70 ksi.



<u>Solution</u>

From table 1-7 (L shapes Dimensions): for L 8x6x3/4 (A_g = 9.99 in^2 1.56in)

- Cross sectional yielding
  $P_n = F_y A_g$
  P_n = 50 * 9.99 = 499.5 k
  $\varnothing_t P_n = \varnothing_t * F_y * A_g$
  0.9*499.5 = 449.5 k
- Tensile rupture strength

$A_n = A_g$ = 9.99 in^2, L = 6in
U= 1- /L = (1-1.56/6) = 0.74
$P_n = F_u A_e$

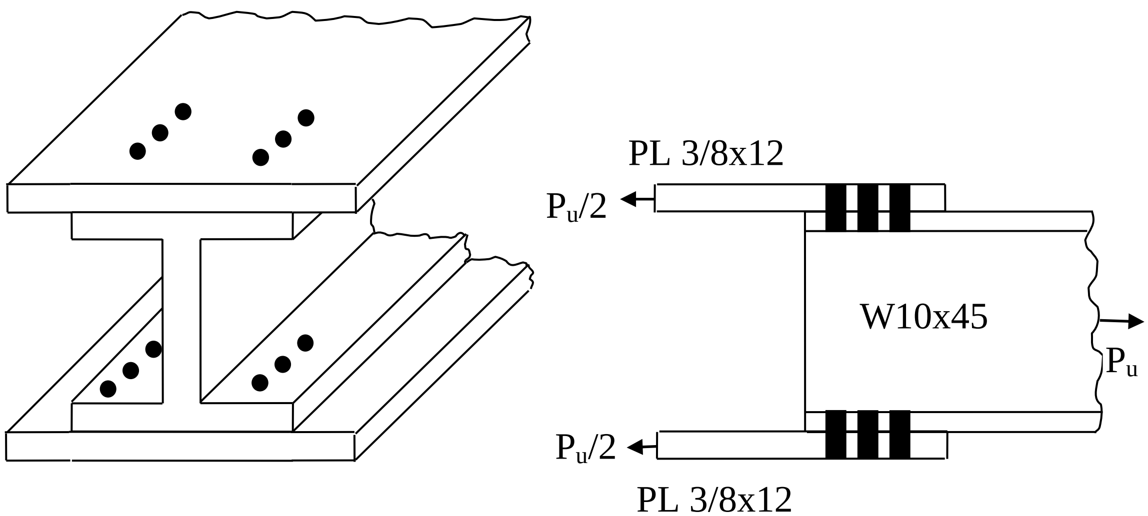
= 70 * 7.3926= 517.48 k

$\varnothing_t P_n = \varnothing_t * F_u * A_e$

= 0.75 * 517.48 = 388.11….. (Control)

❖ <u>Example 9:</u>

The tension member shown in the figure below is assumed to be connected at its ends with two 3/8x12 in plates. If two lines of (3/4 in) bolts are used in each plate determine the LRFD design tensile force that the two plate can transfer.



1. Tensile yielding of   elements:

$$Nominal\ strength\ of\ plates\ R_n = F_y A_g$$

$$R_n = 50 * (\frac{3}{8} * 12 * 2) = 450\ k$$

$$\emptyset_t R_n = \emptyset_t F_y A_g$$

$$\emptyset_t R_n = 0.9 * 450 = 405\ k$$

2. Tensile rupture of connecting elements:

$$A_n of\ 2\ plates = 2 * 12 * \frac{3}{8} - 4 * \left(\frac{13}{16} + \frac{1}{16}\right) * \frac{3}{8} = 7.6875\ in^2 \leq 0.85A_g$$

$$0.85A_g = 0.85 * 2 * 12 * \frac{3}{8} = 7.65\ in^2$$

$$\therefore A_n \quad 7\ 5\ in^2 = A_e$$

$$R_n = F_u A_e$$

$$P_n = 65 * 7.65 = 497.25\ k$$

$$\text{Ø}_t R_n = \text{Ø}_t F_u A_e$$

$$\text{Ø}_t R_n = 0.75 * 497.25 = 372.9375\ k = P_u(controls)$$

❖ Block shear:

- The LRFD design strength of tension members is not always controlled by $\varnothing_t$ P_n or by the strength of the bolts or weld with which they are connected. They may instead be controlled by block shear strength.
- The failure of a member may occur along a path involving tension on one plane and shear on a perpendicular plane.
- The AISC specification (J4.3) states that the block shear design strength of particular member is to be determined by:

1. Computing the tensile fracture strength on the net section in one direction and adding to that value the shear yield strength on the gross area on the perpendicular segment
2. Computing the shear fracture strength on the gross area subject to tension and adding it to the tensile yield strength on the net area subject to shear on the perpendicular segment
3. The expression to apply is the one with larger rupture term.

- The AISC specification (J4.3)states that the available strength R_n for the block shear rupture design strength is as follows:

$$R_n = 0.6\, F_u A_{nv} + U_{bs}\, F_u A_{nt}$$

$$\leq\ 0.6\, F_y A_{gv} + U_{bs}\, F_u A_{nt}\ (AISC\ equation\ J4-5)$$

$$(\emptyset = 0.75)$$

A_{gv} = Gross area subjected to shear, in^2

A_{nv} = Net area subjected to shear, in^2

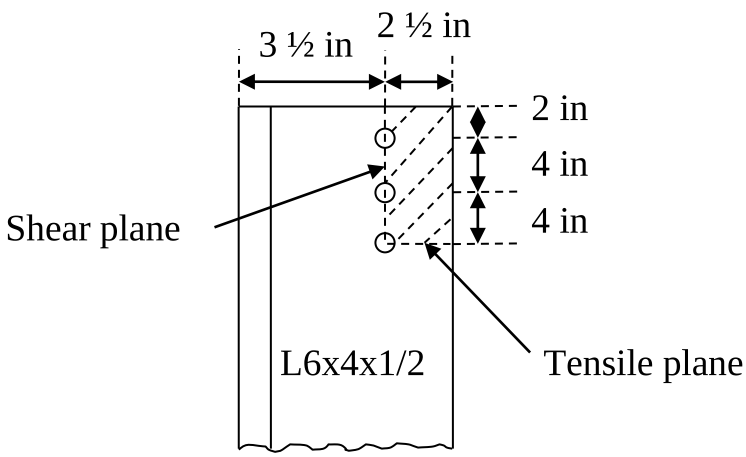
A_{nt} = Net area subjected to tension, in^2

- The reduction factor U_{bs}, is used to account for the fact that stress distribution may not be uniform on the tensile plane for some connections.

1. Should the tensile stress distribution be uniform, U_{bs} will be taken equal to 1.0. The tensile stress is generally considered to be uniform for angles, gusset or connection plates and for coped beam with one line of bolts.
2. Should the tensile stress be nonuniform U_{bs} is to be equal to 0.5, such a situation occurs in coped beams with two lines of bolts. Should the bolts for coped beams be placed at nonstandard distances from beam ends, the same situation of nonuniform tensile stress can occur, and U_{bs} is to be equal to 0.5.

❖ Example 10:

The A572 grade 50 tensile member shown in the figure below is connected with three (3/4 in) bolts. Determine the LRFD block shear rupture strength of the member. Also calculate the LRFD design tensile strength of the member.



From table 2-4: F_y = 50 ksi, F_u = 65 ksi for A572 Grade 50 steel.

From table 1-7 (L shapes Dimensions): for L 6x6x3/8 (A_g = 4.75 in^2, $\bar{x}$ =0.981 in)

1. Block shear strength

$$A_{gv} = 10 * \frac{1}{2} = 5\ in^2$$

$$A_{nv} = 5 - 2.5 * \left(\frac{13}{16} + \frac{1}{16}\right) * \frac{1}{2} = 3.9063\ in^2$$

$$A_{nt} = 2.5 * \frac{1}{2} - 0.5 * (\frac{13}{16} + \frac{1}{16}) * \frac{1}{2} = 1.0313\ in^2$$

$$U_{bs} = 1.0$$

$$R_n = 0.6\, F_u A_{nv} + U_{bs}\, F_u A_{nt} \le\ 0.6\, F_y A_{gv} + U_{bs}\, F_u A_{nt}$$

$$R_n = 0.6 * 65 * 3.9063 + 1 * 65 * 1.0313$$

$$\le\ 0.6 * 50 * 5 + 1 * 65 * 1.0313$$

$$219.3802 > 217.0345$$

$$\therefore R_n = 217.0345\ k$$

$$\emptyset R_n = 0.75 * 217.0345 = 162.7759\ k =\ P_u\ (controls)$$

2. Gross section yielding (nominal or available tensile strength of angle):

$$P_n = F_y A_g$$

$$P_n = 50 * 4.75 = 237.5\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 237.5 = 213.75\ k$$

3. Tensile rupture strength:

$$A_n = 4.75 - \left(\frac{13}{16} + \frac{1}{16}\right) * \frac{1}{2} = 4.3125\ in^2$$

$$L = 4 + 4 = 8\ in$$

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

$$U = 1 - \frac{0.981}{8} = 0.8774 \ \ (controls)$$

$$U = 0.6 \ (from\ table\ D3 - 1\ case\ 8)$$

$$A_e = UA_n$$

$$A_e = 0.8774 * 4.3125 = 3.7838\ in^2$$

$$P_n = F_u A_e$$

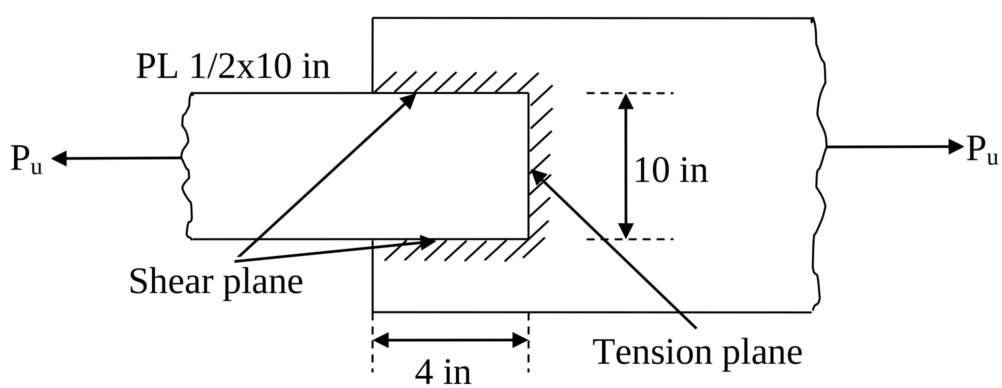
$$P_n = 65 * 3.7838 = 245.947\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 245.947 = 184.4603\ k$$

❖ Example 11:

Determine the LRFD design strength of the A36 (F_y=36 ksi, F_u=58 ksi) plates shown in the figure below. Include block shear strength in the calculations.



1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 36 * 10 * 1/2 = 180\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 180 = 162\ k\ =\ P_u\ (controls)$$

2. Tensile rupture strength:

$$A_n = A_g = 10 * \frac{1}{2} = 5\ in^2$$

$$U = 1\ (table\ D3.1\ case\ 1)$$

$$A_e = UA_n$$

$$A_e = 1 * 5 = 5\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 58 * 5 = 290\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 290 = 217.5\ k$$

3. Block shear strength

$$A_{gv} = \frac{1}{2} * 4 * 2 = 4\ in^2$$

$$A_{nv} = 4\ in^2$$

$$A_{nt} = \frac{1}{2} * 10 = 5\ in^2$$

$$U_{bs} = 1.0$$

$$R_n = 0.6\, F_u A_{nv} + U_{bs}\, F_u A_{nt} \leq\ 0.6\, F_y A_{gv} + U_{bs}\, F_u A_{nt}$$

$$R_n = 0.6 * 58 * 4 + 1 * 58 * 5 \leq\ 0.6 * 36 * 4 + 1 * 58 * 5$$

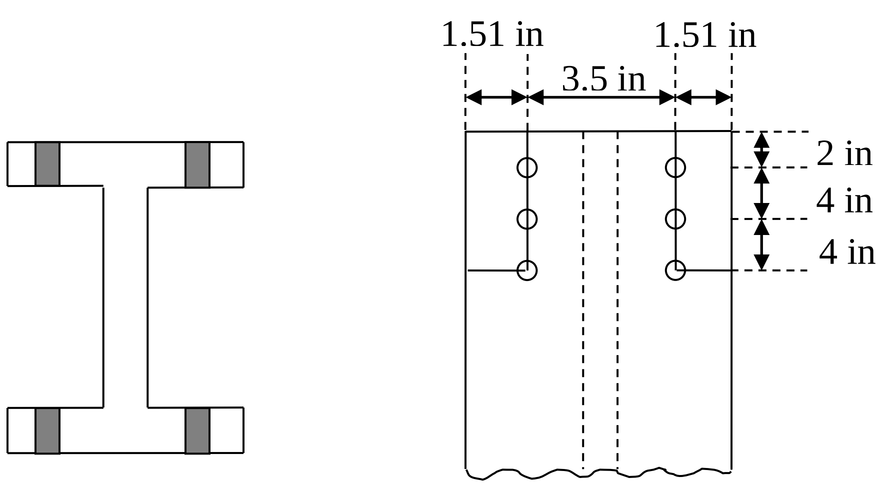
$$429.2 > 376.4$$

$$\therefore R_n = 376.4\ k$$

$$\emptyset R_n = 0.75 * 376.4 = 282.3\ k$$

❖ Example 12:

Determine the LRFD tensile design strength of the W12x30 (F_y=50 ksi, F_u=65 ksi) shown in the figure below if 7/8 in bolts are used in the connection. Include block shear calculation for the flanges.



From table 1-1 (W shapes Dimensions): for W 12x30 (A_g = 8.79 in^2, d = 12.3 in, b_f = 6.52 in, t_f = 0.44 in, t_w = 0.26 in)

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 50 * 8.79 = 439.5\ k$$

$$\text{Ø}_t P_n = \text{Ø}_t F_y A_g$$

$$\text{Ø}_t P_n = 0.9 * 439.5 = 395.55\ k$$

2. Tensile rupture strength:

$$A_n = 8.79 - 4 * \left(\frac{15}{16} + \frac{1}{16}\right) * 0.44 = 7.03\ in^2$$

From table 1-8 (WT shapes Dimensions): for WT 6x15(which is half of a W12x30) $\bar{x} = 1.27$

$$L = 4 + 4 = 8\ in$$

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

$$U = 1 - \frac{1.27}{8} = 0.8413$$

$$b_f = 6.52\ in < \frac{2}{3} d = \frac{2}{3} * 12.3 = 8.2\ in$$

$$\therefore U = 0.85\ (\ from\ table\ D3 - 1\ case\ 7)(controls)$$

$$A_e = UA_n = 0.85 * 7.03 = 5.9755\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 65 * 5.9755 = 388.4075\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 388.4075 = 291.3056\ k = P_u(controls)$$

3. Block shear strength

$$A_{gv} = 10 * 0.44 * 4 = 17.6\ in^2$$

$$A_{nv} = 17.6 - 4 * (2.5 * \left(\frac{15}{16} + \frac{1}{16}\right) * 0.44) = 13.20\ in^2$$

$$A_{nt} = 4 * 1.51 * 0.44 - 4 * 0.5 * (\frac{15}{16} + \frac{1}{16}) * 0.44 = 1.7776\ in^2$$

$$U_{bs} = 1.0$$

$$R_n = 0.6\ F_u A_{nv} + U_{bs}\ F_u A_{nt} \leq\ 0.6\ F_y A_{gv} + U_{bs}\ F_u A_{nt}$$

$$R_n = 0.6 * 65 * 13.2 + 1 * 65 * 1.7776$$

$$\leq\ 0.6 * 50 * 17.6 + 1 * 65 * 1.7776$$

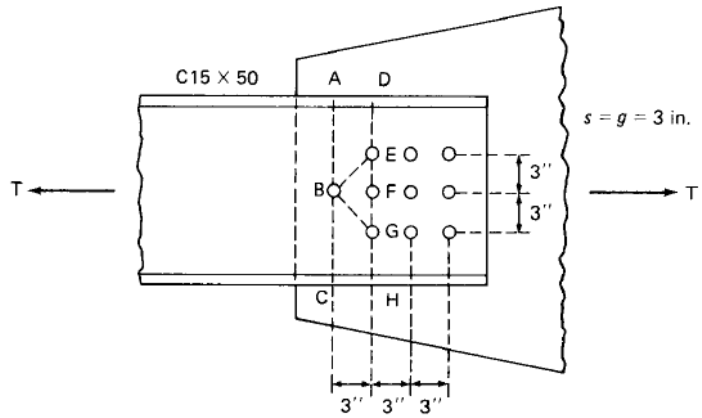
$$630.344 < 643.544$$

$$\therefore R_n = 630.344\ k$$

$$\emptyset R_n = 0.75 * 630.344 = 472.758\ k$$

❖ <u>Example 13</u>

Find the maximum allowable tensile load T for a single channel section C15x50 connected to a 1"gusset plate to the web as by 10 bolts of 1" diameter as shown below. Use A572G50 steel material.



<u>Solution</u>

From table of AISC manual Fy = 50ksi, Fu 65kis, Ag =14.7in^2, Tw =0.716"

For standard-size holes (STD), the size of hole should be 1/8 in. greater than the diameter of the fastener.

Size of hole for 1" bolt = 1+1/8=9/8"

Reduction factor: U = 0.85 (J 4.1)

At section ABC: An1 = Ag – n tw (D+1/8) = 13.89 in2

At section DEBGH: An2 = $\text{Ag} - \text{n tw}\left(\text{D} + \frac{1}{8}\right) + \text{ n tw } \frac{\text{s2}}{\text{4g}}$=13.36 in2 .

At section DEFGH: An3 = Ag – n tw (D+ 1/8) = 12.28 in2

The controlled net area is the minimum value = 12.28 in2

Maximum allowable tensile load based on the yield criterion:

T = 0.60 Fy Ag = 0.60(50)(14.7) = 441 kips

Maximum allowable tensile load based on the fracture criterion:

T = 0.50 Fu Ae = 0.50(65)(0.85)(12.28) = 339.26 kips

Therefore, the maximum allowable tensile load is controlled by T = 339.26 kips

Al-Mansour University College

قسم الهندسة المدنية

Civil Eng. Dept.

المرحلة الرابعة

4th. Stage

Structural Steel Design

2022 - 2023

Lec.2

تصميم حديد

Lec. Hayder Qais

Chapter 3

Design of Tension Members

Selection of Members:

- This chapter deals with the design of tension members for external loads.
- In general the design of tension members have the following properties:

1. Compactness
2. Dimensions that fit into the structure with reasonable relation to other member dimensions.
3. Connection to as many part of the section nd leg

- The choice of member type is often affected by the type of connections used for the structure. Some steel section are not very convenient to bolt together with the required gusset or connection plates, while the same section may be welded together with little difficulty.

1. Tension members consisting of angles, channels, and W or S sections will probably be used when the connections are made with bolts
2. Plates, channels and structural tees might be used for welded structures.

2 • The slenderness ratio of a member is the ratio of its unsupported

length to its least radius of gyration. For tension members other than rods, the maximum value of slenderness ratio is suggested le deflections or vibrations.

- The area needed for a particular tension member can be estimated with the LRFD or the ASD equations. If the LRFD equations are used, the design strength of a tension member is the least of $\emptyset_t F_y A_g$, $\emptyset_t F_u A_e$, or its block shear strength $\emptyset_t R_n$. in addition, the slenderness ratio should not exceed 300.

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

$$\text{Min } A_g = \frac{P_u}{\emptyset_t F_y}$$

2. To satisfy the second of these expressions, the minimum value of Ae must be at least equal to:

$$\text{Min } A_e = \frac{P_u}{\emptyset_t F_u}$$

And since $A_e = UA_n$ for bolted,

$$\text{Min } A_n = \frac{\text{Min } A_e}{U} = \frac{P_u}{\emptyset_t F_u U}$$

And since A_g = Min An + estimated area of holes,

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

3

3. The third expression can be evaluated, once a trial shape has been

selected and other parameters related to the block shear strength are known.

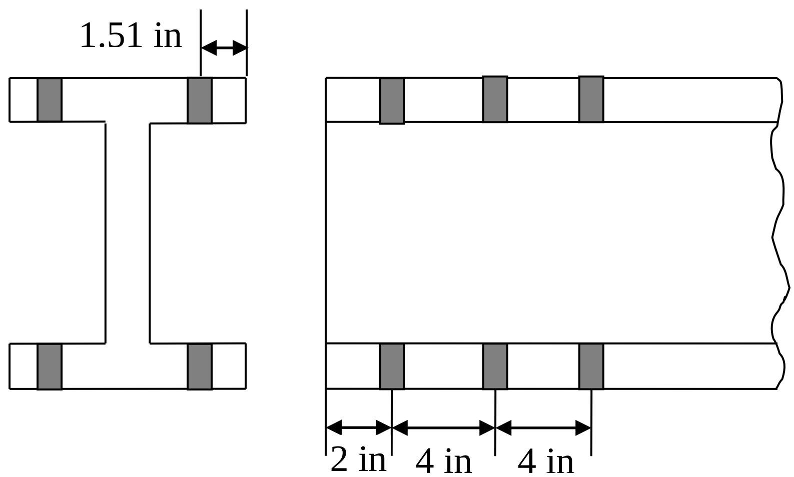
- The designer can substitute into equations 1 and 2, taking the larger value of Ag so obtained for an i

$$\text{Min r} = \frac{L}{300}$$

4

❖ Example 1:

Select a 30 ft long W12 section of A992 steel to support a tensile service dead load P_D = 130 k and a tensile service live load P_L = 110 k. 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).



From table 2-4: F_y = 50 ksi, F_u = 65 ksi for A992 steel.

a) Considering the necessary load combinations:

$$P_u = 1.4\,D = 1.4 * 130 = 182\ k$$

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

b) Computing the minimum A_g required using LRFD equations:

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

$$\min A_g = \frac{332}{0.9 * 50} = 7.3778\ in^2$$

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

$$assume\ U = 0.9\ (from\ table\ D3 - 1\ case\ 7)$$

$$assume\ t_f = 0.38\ in$$

After looking at W12 sections in the AISC manu or more.

$$\min A_g = \frac{332}{0.75 * 65 * 0.9} + 4 * \left(\frac{15}{16} + \frac{1}{16}\right) * 0.38 = 9.087\ in^2$$

c) Preferable minimum r:

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

$$\min r = \frac{30 * 12}{300} = 1.2\ in$$

Try W12x35 (A_g = 10.3 in^2, d = 12.5 in, b_f = 6.56 in, t_f = 0.52 in, r_y = 1.54 in, r_x = 5.25 in)

d) Checking:

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 50 * 10.3 = 515\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 515 = 463.5\ k > 332\ k\ \therefore ok$$

2. Tensile rupture strength:

$$A_n = 10.3 - 4 * \left(\frac{15}{16} + \frac{1}{16}\right) \;\; 0 = 8.22\ in^2$$

From table 1-8 (WT shapes Dimensions): for WT 6x17.5 (which is half of a W12x35) $\bar{x} = 1.3$

$$L = 4 + 4 = 8\ in$$

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

$$U = 1 - \frac{1.3}{8} = 0.8375$$

$$b_f = 6.56\ in < \frac{2}{3}d = \frac{2}{3} * 12.5 = 8.3333\ in$$

$$\therefore U = 0.85\ (\ from\ table\ D3 - 1\ case\ 7)(controls)$$

$$A_e = UA_n = 0.85 * 8.22 = 6.987\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 65 * 6.987 = 454.1\ \text{Ø}_t P_n = \text{Ø}_t F_u A_e$$

$$\text{Ø}_t P_n = 0.75 * 454.155 = 340.6163\ k > 332\ k \therefore ok$$

3. Block shear strength

$$A_{gv} = 10 * 0.52 * 4 = 20.8\ in^2$$

7

$$A_{nv} = 20.8 - 4 * (2.5 * \left(\frac{15}{16} + \frac{1}{16}\right) * 0.52) = 15.6\ in^2$$

$$A_{nt} = 4 * 1.51 * 0.52 - 4 * 0.5 * (\frac{15}{16} + \frac{1}{16}) * 0.52 = 2.1008\ in^2$$

$$U_{bs} = 1.0$$

$$R_n = 0.6 * 65 * 15.6 + 1 * 65 * 2.1008$$

$$\leq\ 0.6 * 50 * 20.8 + 1 * 65 * 2.1008$$

$$744.952 < 760.552$$

$$\therefore R_n = 744.952\ k$$

4. Slenderness ration

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

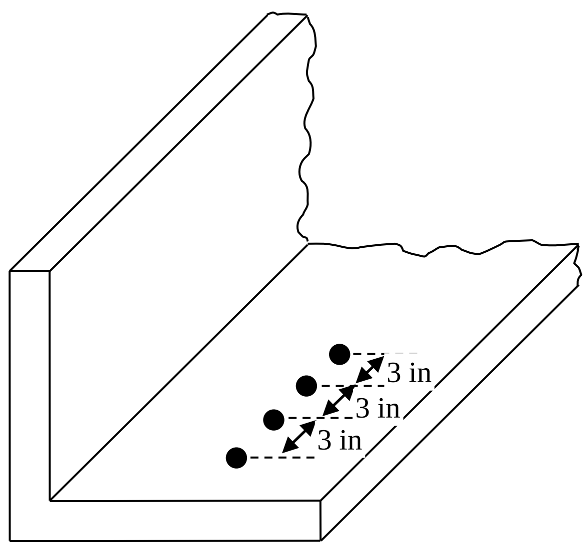
$$\frac{30 * 12}{1.54} = 233.7662\ in < 300\ \therefore ok$$

e) Checking for lighter section:

Try W12x30 (A_g = 8.79 in^2, d = 12.3 in, b_f = 6.52 in, t_f = 0.44 in, r_y = 1.52 in, r_x = 5.21 in)

❖ Example 2:

Design a 9 ft long 1/2 in thick single angle tension member to support a dead tensile working load of 30 k and a live tensile working load of 40 K. The member is to be conne



a) Considering the necessary load combinations:

$$P_u = 1.4\,D = 1.4 * 30 = 42\,k$$

$$P_u = 1.2\,D + 1.6L = 1.2 * 30 + 1.6 * 40 = 100\,k\ (controls)$$

b) Computing the minimum A_g required using LRFD equations:

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

9

$$\min A_g = \frac{100}{0.9 * 36} = 3.0864 \, in^2$$

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

$$assume\ U = 0.8\ (\ from\ table\ D3 - 1\ case\ 8)$$

$$the\ thickness\ of\ the\ angle\ (t) = 1/2\ in$$

$$\min A_g = \frac{100}{0.75 * 58 * 0.8} + 1 * \left(\frac{15}{16} + \frac{1}{16}\right) * \frac{1}{2} = 3.3736\ in^2$$

c) Preferable minimum r:

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

$$\min r = \frac{9 * 12}{300} = 0.36\ in$$

Try L 4x31/2 x1/2 (A_g = 3.5 in^2, r_y = 1.04 in, r_x = 1.23 in, r_z = 0.716 in, $\bar{x} = 0.994\ in$)

d) Checking:

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 36 * 3.5 = 126\ k$$

$$3.4\ k > 100\ k \therefore ok$$

2. Tensile rupture strength:

10

$$A_n = 3.5 - 1 * \left(\frac{15}{16} + \frac{1}{16}\right) * 1/2 = 3\ in^2$$

$$L = 3 + 3 + 3 = 9\ in$$

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

$$U = 1 - \frac{0.994}{9} = 0.8951\ (controls\ )$$

$$U = 0.8\ (\ from\ table\ D3 - 1\ case\ 8)$$

$$A_e = UA_n = 0.8951 * 3 = 2.6853\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 58 * 2.6853 = 155.7493\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 155.7493 = 116.812\ k > 100\ k\ \ \therefore ok$$

3. Slenderness ration

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

$$\frac{9 * 12}{0.716} = 150.837\ in < 300\ \ \therefore ok$$

e) Checking for lighter section:

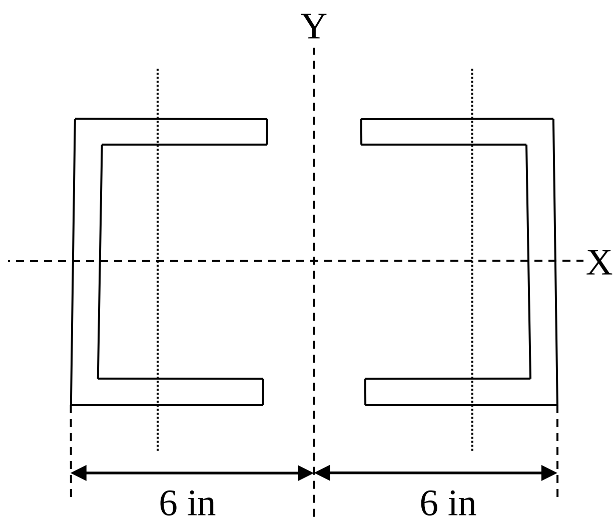
Try L 4x3x1/2 (A_g = 3.25 in^2, r_y = 0.873 in, r_x = 1.24 in, r_z = 0.633 in,

$\bar{x} = 0.822\ in$)

11

❖ Example 3:

A welded tension member is to support the service load $P_D = 150$ k and $P_L = 300$ k and is to consist of two channels placed 12 in out to out, with the flanges turned in. select the lightest standard channels available. Assume that U=0.87. The member is to be 30 ft long. Use A36 steel.



a) Considering the necessary load combinations:

$$P_u = 1.4\,D = 1.4 * 150 = 210\ k$$

$$P_u = 1.2\,D + 1.6L = 1.2 * 150 + 1.6 * 300 = 660\ k\ (controls)$$

b) Computing the minimum A_g required using LRFD equations:

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

$$\min A_g = \frac{660}{0.9 * 36} = 20.3704\ in^2\ (controls)$$

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

12

$$\min A_g = \frac{660}{0.75 * 58 * 0.87} + 0 = 17.4396\ in^2$$

c) Preferable minimum r:

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

$$\min r = \frac{30 * 12}{300} = 1.2\ in$$

Try C 15x40 (A_g = 11.8 in^2, I_y = 9.17 in^4, I_x = 348 in^4, $\bar{x} = 0.778\ in$)

$$A_g = 2 * \ \text{n}^4$$

$$I_y\ for\ the\ built-up\ section = \left(\overline{I_y} + Ad^2\right) * 2$$

$$I_y = (9.17 + 11.8 * (6 - 0.778)^2) * 2 = 661.9\ \text{in}^4\ \text{(controls)}$$

$$r_y = \sqrt{\frac{I_y}{A_g}}$$

$$r_y = \sqrt{\frac{661.9}{23.6}} = 5.2959\ in > \min r = 1.2\ in\ \therefore ok$$

d) Checking:

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 36 * 23.6 = 849.6\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

13

$$\emptyset_t P_n = 0.9 * 849.6 = 764.64\ k > 660\ k\ \therefore ok$$

2. Tensile rupture strength:

$$A_n = A_g = 23.6\ in^2$$

$$A_e = UA_n = 0.87 * 23.6 = 20.532\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 58 * 20.532 = 1190.856\ k$$

$$\emptyset_t P_n = 0.75 * 1190.856 = 893.142\ k > 660\ k \therefore ok$$

3. Slenderness ration

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

$$\frac{30 * 12}{5.2959} = 67.977\ in < 300\ \therefore ok$$

e) Checking for lighter section:

Try C 15x33.9 (A_g = 10 in^2, I_y = 8.07 in^4, I_x = 315 in^4, $\bar{x} = 0.788\ in$)

$$A_g = 2 * 10 = 20\ \text{in}^2 < \min A_g = 20.3704\ \therefore \text{not ok}$$

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 36 * 20 = 720\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

14

$$\emptyset_t P_n = 0.9 * 720 = 648\ k < 660\ k\ \ \therefore not\ ok$$

$$\therefore Use\ 2\ \mathrm{C}\ 15\mathrm{x}40$$

15

- …etween those elements ≤ 24 t of thinner thickness. Or 12” if the member is painted or not subjected to corrosion causes.
- And in continuous contact, S should be ≤ 14 t of the thinner thickness or 7” when the …

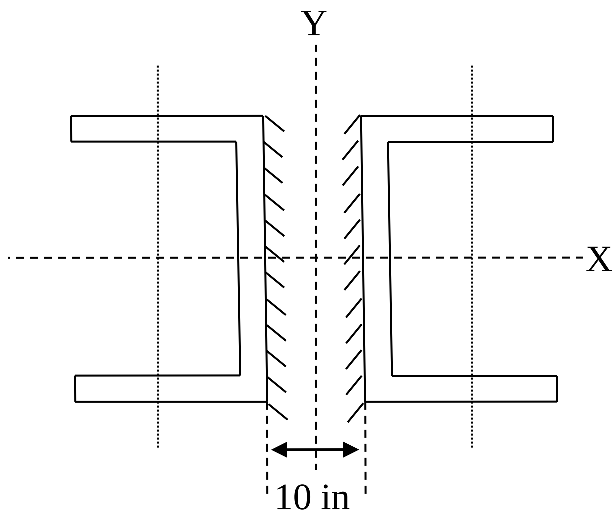
- …ngth of tie plate (Ltp) should be ≥ (2/3) distance between connectors & it is …

- The thickness of tie plate should be ≥ (1/ 50) Ltp.

16

❖ <u>Example 4:</u>

Select a pair of C8 for the condition shown. Use A572 grade 50 steel and assume transverse welds at the ends along the web only. L = 24 ft. The service load $P_D = 100$ k and $P_L = 110$ k.



a) Considering the necessary load combinations:

$$P_u = 1.4\,D = 1.4 * 100 = 140\,k$$

$$P_u = 1.2\,D + 1.6L = 1.2 * 100 + 1.6 * 110 = 296\,k\ (controls)$$

b) Computing the minimum A_g required using LRFD equations:

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

$$\min A_g = \frac{296}{0.9 * 50} = 6.5778\,in^2$$

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

$$\min A_g = \frac{296}{0.75 * 65 * 1} + 0 = 6.0718\ in^2$$

c) Preferable minimum r:

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

$$\min r = \frac{24 * 12}{300} = 0.96\ in$$

Try C 8x11.5 (A_g = 3.37 in^2, I_y = 1.31 in^4, I_x = 32.5 in^4, $\bar{x} = 0.572\ in,$

d = 8 in, t_w = 0.22 in)

$$A_g = 2 * 3.37 = 6.74 \text{ in}^2 > \min A_g = 6.5778 \text{ in}^2 \therefore \text{ok}$$

d) Checking:

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 50 * 6.74 = 337\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 337 = 303.3\ k > 296\ k\ \therefore ok$$

2. Tensile rupture strength:

$$\text{Ø}_t P_n = \text{Ø}_t F_u A_e$$

$$\text{Ø}_t P_n = 0.75 * 228.8 = 171.6\ k < 296\ k\ \therefore not\ ok$$

e) Checking for larger section:

1. Try C 8x13.75 (A_g = 4.03 in^2, d = 8 in, t_w = 0.303 in)

$$A_g = 2 * 4.03 = 8.06 \text{ in}^2 > \min A_g = 6.5778 \text{ in}^2 \therefore \text{ok}$$

Gross section yielding:

$$P_n = F_y A_g = 50 * 8.06 = 403\ k$$

$$\text{Ø}_t P_n = \text{Ø}_t F_y A_g$$

$$\text{Ø}_t P_n = 0.9 * 403 = 362.7\ k > 296\ k\ \therefore ok$$

Tensile rupture strength:

2. Try C 8x18.75 (A_g = 5.51 in^2, I_y = 1.97 in^4, I_x = 43.9 in^4, $\bar{x} =$ 0.565 in, d = 8 in, t_w = 0.487 in)

$$A_g = 2 * 5.51 = 11.02 \text{ in}^2 > \min A_g = 6.5778 \text{ in}^2 \therefore \text{ok}$$

Gross section yielding:

19

$$P_n = F_y A_g = 50 * 11.02 = 551\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 551 = 495.9\ k > 296\ k\ \ \therefore ok$$

Tensile rupture strength:

$$A_n = d * t_w * 2 = 8 * 0.487 * 2 = 7.792\ in^2$$

$$A_e = UA_n = 1.0 * 7.792 = 7.792\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 65 * 7.792 = 506.48\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 506.48 = 379.86\ k < 296\ k\ \ \therefore ok$$

$$I_x\ for\ the\ built-up\ section = 2 * 43.9 = 87.8\ \mathrm{in}^4$$

$$I_y\ for\ the\ built-up\ section = \left(\overline{I_y} + Ad^2\right) * 2$$

$$I_y = (1.97 + 5.51 * (5 + 0.565)^2) * 2 = 345.221\ \mathrm{in}^4\ \text{(controls)}$$

$$r_y = \sqrt{\frac{I_y}{A_g}}$$

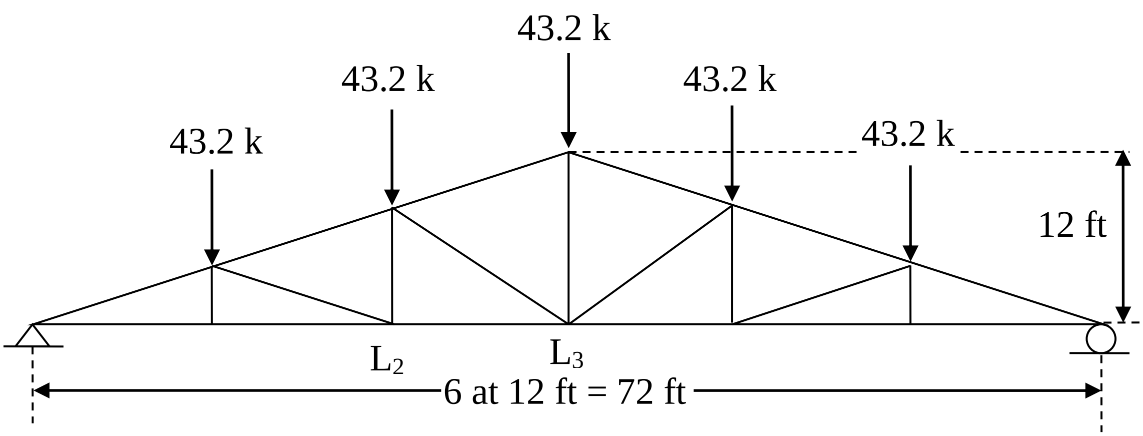
$$r_y = \sqrt{\frac{87.8}{11.02}} = 2.823\ in > \min r = 0.96\ in\ \ \therefore ok$$

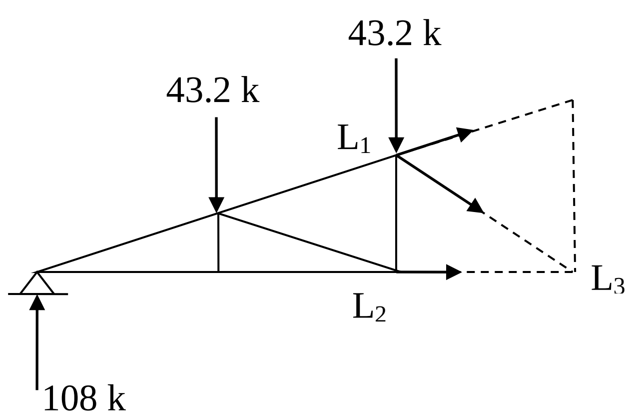
Slenderness ration

$$\frac{L}{r} < 300 \rightarrow \frac{24 * 12}{2.823} = 102.019\ in < 300\ \ \therefore ok$$

20

❖ Example 5:





assume that U=0.85.

a) Considering the necessary load combinations:

$$P_u = 1.4\,D = 1.4 * 20 = 28\ k$$

$$P_u = 1.2\,D + 1.6(L_r\ or\ S\ or\ R) + (0.5L\ or\ 0.8W)$$

$$= 1.2 * 20 + 1.6 * 12 = 43.2\ k\ (controls)$$

$$\sum F_y = 0\ \rightarrow R_A = R_B = \frac{43.2 * 5}{2} = 108\ k$$

$$\sum M_{L1} = 0$$

$$F_{L2L3} * 8 + 43.2 * 12 - 108 * 24 = 0$$

$$\therefore F_{L2L3} = 259.2\ k\ (Tension)$$

b) Computing the minimum A_g required using LRFD equations:

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

$$\min A_g = \frac{259.2}{0.9 * 36} = 8\ in^2$$

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

$$U = 0.85$$

$$\min A_g = \frac{259.2}{0.75 * 58 * 0.85} + 4 * \left(\frac{13}{16} + \frac{1}{16}\right) * t$$

$$= 7.0101 + 3.5t\ in^2$$

t in	E.H.A. in^2	Ag min	Lightest pair available
3/8	1.3125	8.3226	2L6x6x3/8, Ag=8.76 in^2, Wt.=14.9 Ib/ft
7/16	1.5313	8.5414	2L6x6x7/16, Ag=10.2 in^2, Wt.=17.2 Ib/ft
1/2	1.75	8.7601	2L5x5x1/2, Ag=9.58 in^2, Wt.=16.2 Ib/ft

5/8	2.1875	9.1976	2L4x4x5/8, Ag=9.22 in^2, Wt.=15.7 Ib/ft
3/4	2.625	9.6351	2L4x4x3/4, Ag=10.9 in^2, Wt.=18.5 Ib/ft
7/8	3.0625	10.0726	2L5x5x7/8, Ag=16 in^2, Wt.=27.2 Ib/ft

c) Preferable minimum r:

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

$$\min r = \frac{12 * 12}{300} = 0.48\ in$$

2L 6x6x3/8 (A_g = 8.76 in^2, r_y = 2.60 in, r_x = 1.87 in, r_z = 1.19 in, I_y = 15.4 in^4, $\bar{x} = 1.62\ in$)

d) Checking:

1. Gross section yielding:

$$P_n = F_y A_g$$

$$P_n = 36 * 8.76 = 315.36\ k$$

$$\emptyset_t P_n = \emptyset_t F_y A_g$$

$$\emptyset_t P_n = 0.9 * 315.36 = 283.824\ k > 259.2\ k\ \therefore ok$$

2. Tensile rupture strength:

$$A_n = 8.76 - 4 * \left(\frac{13}{16} + \frac{1}{16}\right) * 3/8 = 7.4475\ in^2$$

$$U = 0.85$$

$$A_e = UA_n = 0.85 * 7.4475 = 6.3304\ in^2$$

$$P_n = F_u A_e$$

$$P_n = 58 * 6.3304 = 367.1618\ k$$

$$\emptyset_t P_n = \emptyset_t F_u A_e$$

$$\emptyset_t P_n = 0.75 * 347.1618 = 275.3713\ k > 259.2\ k\ \therefore ok$$

3. Slenderness ration

$$I_y = \left(15.4 + 4.38 * \left(\frac{3}{8}/2 + 1.62\right)^2\right) * 2 = 59.4194\ \mathrm{in}^4$$

$$r_y = \sqrt{\frac{I_y}{A_g}}$$

$$r_y = \sqrt{\frac{59.4194}{8.76}} = 2.60\ in$$

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

$$\frac{12 * 12}{1.19} = 121.008\ in < 300\ \therefore ok$$

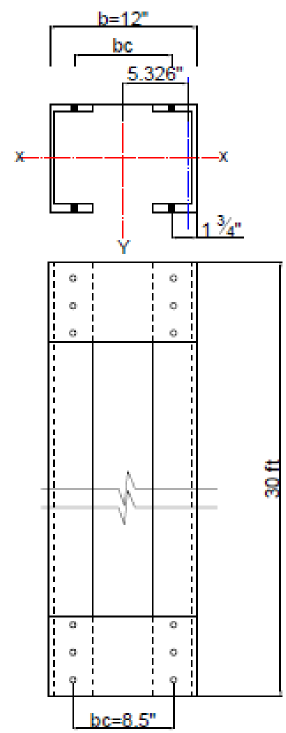
$$\therefore Use\ L6x6x\frac{3}{8}\ in$$

24

2C12 D. 12 . 2 r d

connection have one line of 3x7/8” bolts in a gage line of 1.75” from back edge of channel & 3” distance in between , use A36 steel material to check the adequacy of

d .



D . r r

. 1 ^{2} d 12

d

R D	D
1.2 D. 1. . 2 .	D. . .

r

- r d 2 . .

LRFD Ø=0.9	ASD Ω=1.67
. . 2 ---- .	Ω . 1. . . ---- .

[illegible]

[illegible]

[illegible]

[illegible]

LRFD Ø=0.75	ASD Ω=2
[illegible]	[illegible]

[illegible]

[illegible]

[illegible]

rx = √(324/17.62) = 4.29 in < ry = √(510/17.62) = 5.38 in

Smaller value control….so Lx / rx = 12x30/4.29 = 83.9 < 300 O.

[illegible]

[illegible]

Minimum length of tie plate = (2/3) 8.5 = 5.57” say 6”

Minimum thickness of tie plate = (1/50) 8.5 = 0.17” say 3/16”.

Minimum width of tie plate = 8.5 + 2 (1 ½) = 11.5 say 12”.

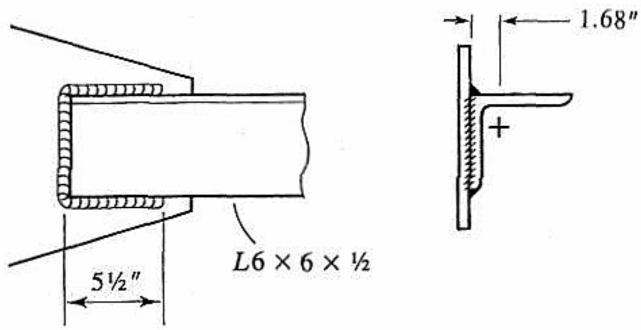
[illegible]

[illegible]

[illegible]

Therefore use tie plate of 3/16” x 12” x 6”

Example 6: Consider the welded single angle L 6x 6 x ½ tension member made from A36 steel shown below. Calculate the tension design strength.



Sol. Form the AISC manual L6x6x1/2, Ag = 5 in^2, $\dot{y} = 1.68$
Gross yielding design strength = Øt Fy Ag = 0.9 x 36 x 5.0 = 162 kips

Net section fracture strength = Øt Fu Ae
For welding: An = Ag = 5.0 in2 So Ae = U Ag

$U = 1 - \frac{\dot{x}}{L} = 1 - \frac{1.68}{6} = 0.72 \leq 0.9\ O.K$ See notes in prg. 2 At table (D3-1).

Net section fracture strength = Øt Fu Ae = 0.75 x 58 x 0.72 x 5.0 = 156.6 kips

Design strength = 156.6 kips (net section fracture strength governs)

Al-Mansour University College

قسم الهندسة المدنية

Civil Eng. Dept.

المرحلة الرابعة

4th. Stage

Structural Steel Design

2022 - 2023

Lec.3

تصميم حديد

Lec. Hayder Qais

Chapter 4: Analysis of Compression Members

❖ Introduction:

-

- Mode of Failures for Columns

1. Flexural Buckling (also called Euler buckling) is the primary type of buckling. Members are subject to flexure or bending when they become unstable.
2. Local Buckling: This type occurs when some part or parts of the cross section of a column are so thin that they buckle locally in compression before the other modes of buckling can occur. The susceptibility of a column to local buckling is measured by the width-thickness ratio of the parts of the cross section
3. Flexural Torsional Buckling may occur in columns that have certain cross-sectional configurations. These columns fail by twisting (torsion) or by a combination of torsional and flexural buckling.

- Slenderness Ratio:

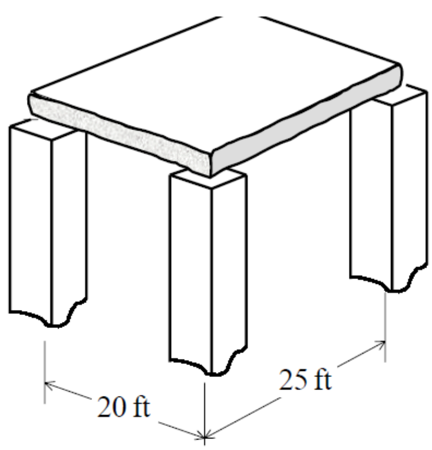
The longer the column becomes for the same cross section, the greater becomes its tendency to buckle and the smaller becomes the load it will carry. The tendency of a member to buckle is usually measured by its slenderness ratio, that is

$$Slenderness\ Ratio = \frac{L}{r}$$

$$Where\ (r) radiusof\ gyration = \sqrt{\frac{I}{A}}$$

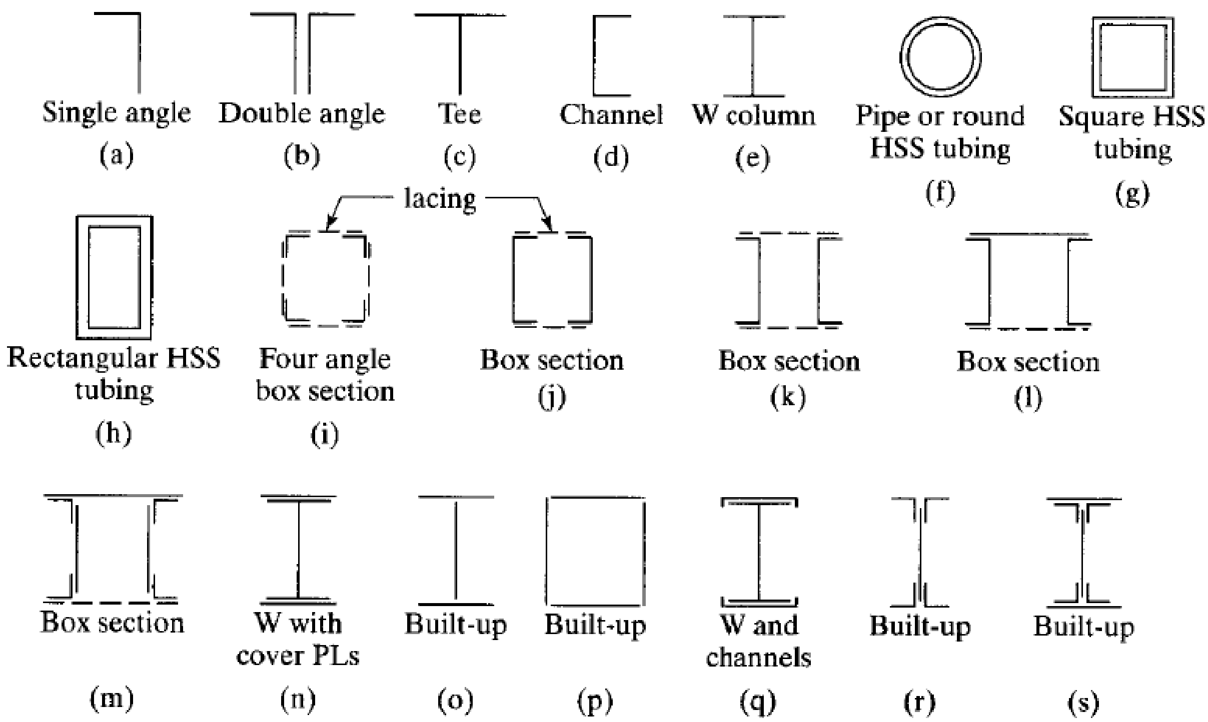
- Column Bay:

  The spacing of columns in plan establishes what is called a *Bay*. For example, if the columns are 20 ft on center in one direction and 25 ft in the other direction, the bay size is 20 ft × 25 ft. Larger bay sizes increase the user's flexibility in space planning.



❖ <u>Section used for column:</u>

-



❖ <u>Development of Column Formulas:</u>

-
- Yield Strength and Length of Column:

1. Short Columns: The yield stresses of the section tested are quite important for short columns as their failure stresses are close to those yield stresses.
2. Columns with Intermediate L/r: The yield stresses are of lesser importance on their effect on failure stresses.
3. Long Slender Columns: The yield stresses are of no significance, but the column strength is very sensitive to end conditions.

- Buckling can be defined as the sudden large deformation of structure due to a slight increase of an existing load under which the structure had exhibited little, if any, deformation before the load was increased.
- Critical buckling load P_{cr}: For a column to buckle elastically, it will have to be long and slender. Its buckling load P_{cr} can be computed with the Euler formula:

$$P_{cr} = \frac{\pi^2 EI}{L^2}$$

Where

E = modulus of elasticity of the material

I = moment of inertia of the cross section

L = length of column

ه

$$Since\ (r) radius of\ gyration = \sqrt{\frac{I}{A}}$$

$$Then\ I = Ar^2$$

Substituting this value into the Euler formula and dividing both sides by the cross sectional area of column the Euler Buckling stress is obtained:

$$\frac{P_{cr}}{A} = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2} = F_e$$

If the value of buckling stress obtained for a particular column exceeds the steel proportional limit, the elastic Euler formula is not applicable.

❖ Example 1:

a.

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

   a. Using W 10x22 (A_g = 6.49 in^2, r_x = 4.27 in, r_y = 1.33 in)

$$Min\ r = r_y = 1.33\ in$$

$$Slenderness\ Ratio = \frac{L}{r} = \frac{15 * 12}{1.33} = 135.34$$

٦

$$Elastic\ or\ buckling\ stress\ F_e = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2} = \frac{\pi^2 * 29000}{(135.34)^2}$$

$$= 15.63\ Ksi < the\ proportional\ limit\ of\ 36\ Ksi$$

$$\therefore ok\ column\ is\ in\ elastic\ range\ and\ Euler\ formula\ \ is\ applicable$$

$$Elastic\ or\ Buckling\ load\ P_{cr} = 15.63 * 6.49 = 101.4\ k$$

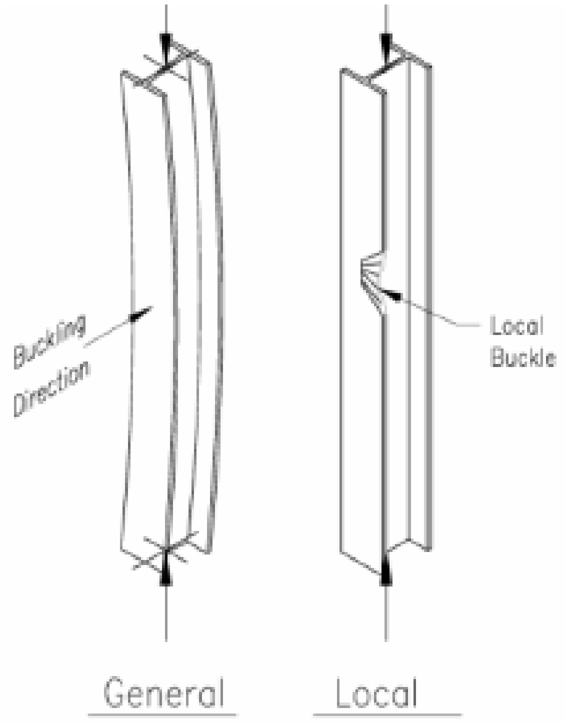
b. Using an 8 ft W 10x22

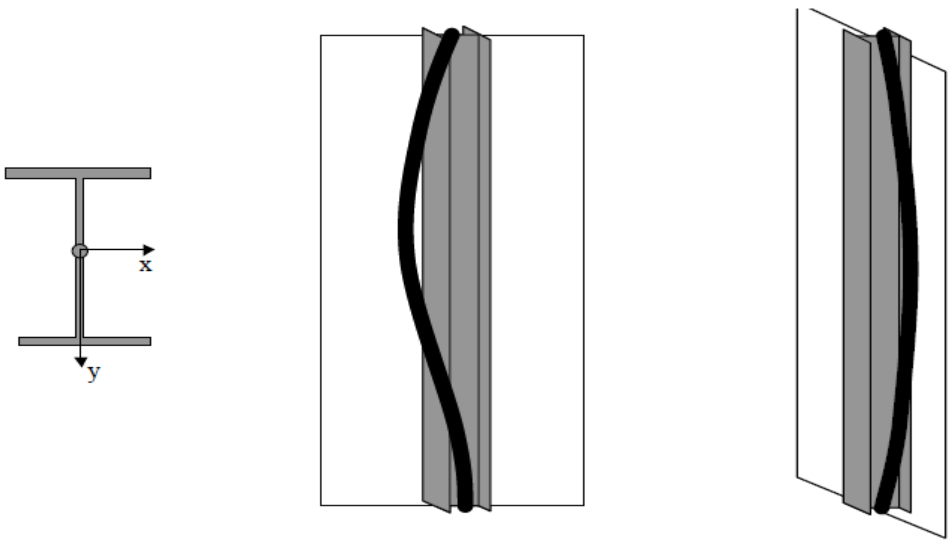
$$Slenderness\ Ratio = \frac{L}{r} = \frac{8 * 12}{1.33} = 72.18$$

$$Elastic\ or\ buckling\ stress\ F_e = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2} = \frac{\pi^2 * 29000}{(72.18)^2}$$

$$= 54.94\ Ksi > the\ proportional\ limit\ of\ 36\ Ksi$$

$$\therefore ok\ column\ is\ in\ inelastic\ range\ and\ Euler\ formula\ \ is\ not\ applicable$$


Buckling
Direction
Local
Buckle
General
Local


x
y

❖ <u>End Restraint and Effective Length of Columns:</u>

- The effective length of a column is defined as the distance between points of zero moment in the column, that is, the distance between its inflection points. In steel specification, the effective length of a column is referred to as KL, where K that one or both ends of a column can move laterally with respect to each other.

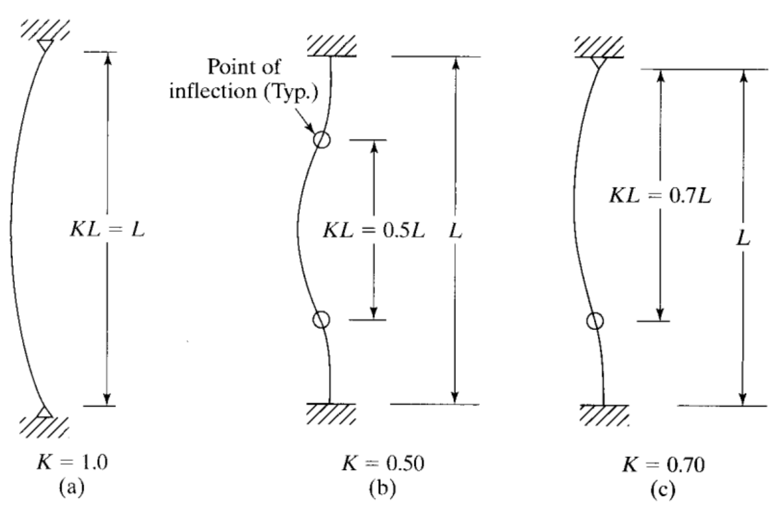
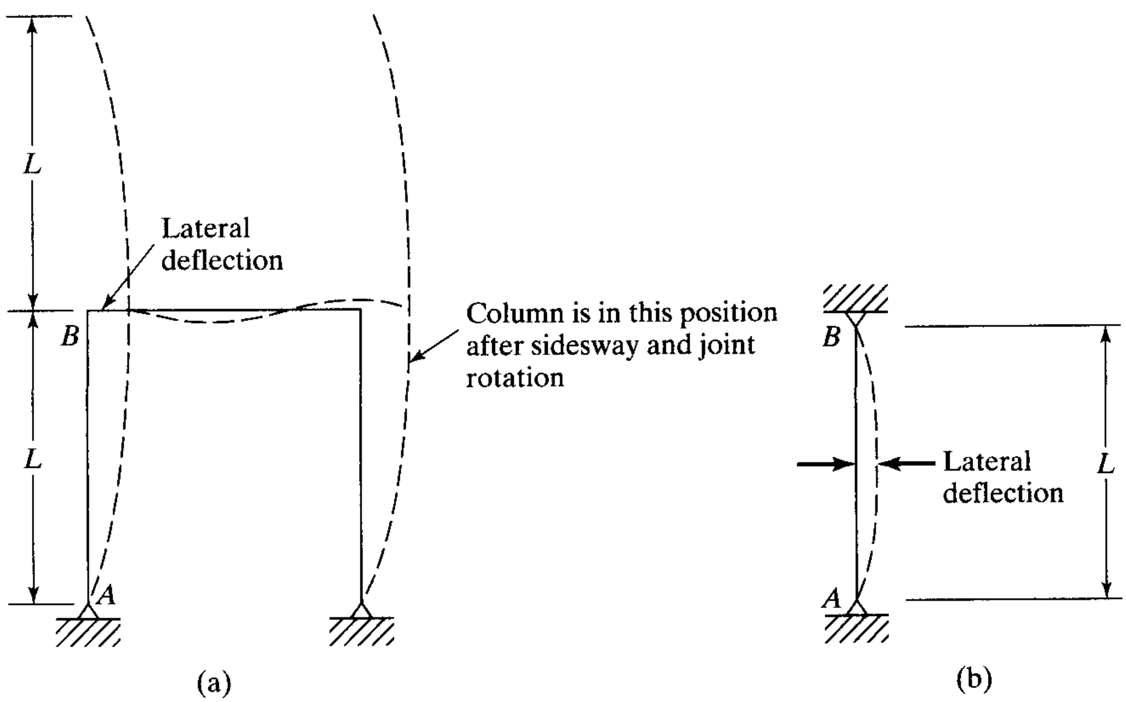


FIGURE 5.4

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

٩

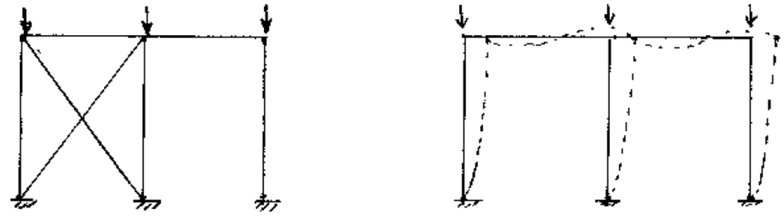
- This discussion would seem to indicate that column effective lengths always vary from an absolute minimum of L/2 to an absolute maximum of L, but there are man heoretically equal 2.0



- Structural steel columns serve as parts of frames, and these frames are sometimes braced and sometime unbraced. A braced frame is one

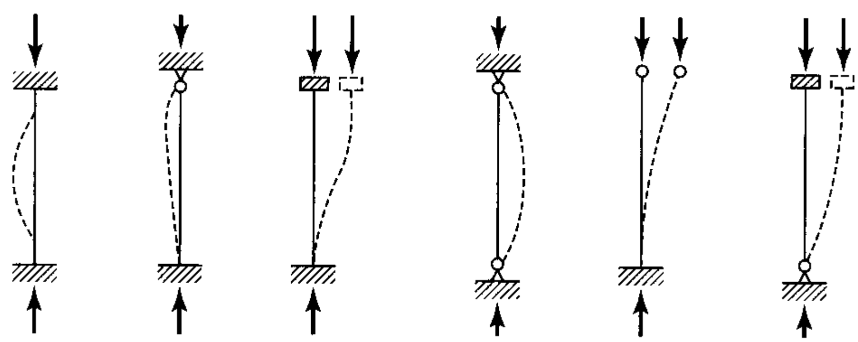
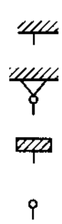
١٠

which sidesway or joint translation is prevented by mean of bracing. For braced frames, K value can never be greater than 1.0, but for unbraced frames, the K values will always be greater than 1.0 because of the sidesway.



Braced frame. Unbraced frame.

TABLE 5.C-C2.2 Approximate Values of Effective Length Factor, *K*

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

١١

❖ Stiffened and Unstiffened Elements:

- It is possible for the thin flanges or webs of a column or beam to buckle locally in compression well before the calculated buckling strength of the whole member is reached. Therefore, the AISC specification (Section B4) provides limiting values for the width-thickness ratio of the individual parts of compression members. Two categories are listed in the AISC manual: stiffened and unstiffened elements.
- An unstiffened element is a projecting piece with one free edge parallel to the direction of the compression force, while, a stiffened element is supported along the two edges in that direction
- For establishing width thickness ratio limits for the elements of compression members the AISC specification divides members into three classifications as follows: compact section, noncompact section, and slender compression elements.
- Compression sections are classified as either a nonslender element or a slender element. A nonslender element is one where the width to thickness ratio of its compression elements does not exceed λ_r, from table 4.1a, when the width to thickness ratio does exceed λ_r; the section is defined as a slender element section.

❖ Column Formulas:

The design strength of compression member is determined as follows,

$$P_n = F_{cr}\, A_g \quad (AISC\ Equation\ E3-1)$$

$$\varphi_c P_n = \varphi_c\, F_{cr}\, A_g = LRFD\ allowable\ compression\ strength\ (\varphi_c = 0.9)$$

Two equations are provided by the LFRD for the critical buckling stress F_{cr},

$$(a) If\ \frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}\ (or\ F_e\ \geq 0.44F_y$$

$$then\ F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \quad (AISC\ Equation\ E3-2)$$

$$(b) If\ \frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}\ (or\ F_e\ < 0.44F_y$$

$$then\ F_{cr} = 0.877F_e \quad (AISC\ Equation\ E3-3)$$

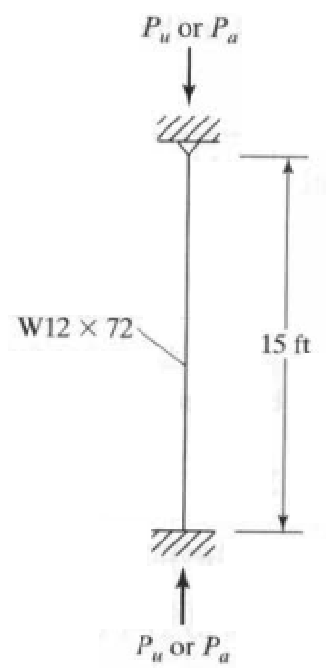
In these expressions, F_e is the elastic critical buckling stress- that is, the Euler stress- calculated with the effective length of the column KL.

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

- To facilitate the design process, the LRFD manual provides computed values of $\varphi_c F_{cr}$ values for steel with F_y = 35, 36, 42, 46, 50 ksi for KL/r from 1 to 200 as shown in table 4-22 of the AISC manual.
- In addition to that, the AISC manual in tables 4-1 to 4-11 has further simplified the needed calculations by computing the LRFD column design strength $\varphi_c P_n$
- For members designed on the basis of compression, the effective slenderness ratio KL/r preferably not exceed 200:

$$\frac{KL}{r} \leq 200$$

❖ <u>Example 2:</u>



1. Using a W 12x72 (A_g = 21.1 in^2, r_x = 5.31 in, r_y = 3.04 in, d =12.3 in, b_f = 12 in, t_f = 0.670 in, k =1.27 in, t_w =0.430 in)

From table B4.1a case 1 and 5:

$$\frac{b}{t} = \frac{\frac{b_f}{2}}{t_f} = \frac{\frac{12}{2}}{0.670} = 8.9552 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29000}{50}} = 13.4866$$

$$\therefore nonslender\ unstiffened\ flange\ element$$

$$\frac{h}{t_w} = \frac{d - 2k}{t_w} = \frac{12.3 - 2 * 1.27}{0.430} = 22.6977 < 1.49\sqrt{\frac{E}{F_y}}$$

$$= 1.49\sqrt{\frac{29000}{50}} = 35.884$$

$$\therefore nonslender\ stiffened\ web\ element$$

$$K = 0.8\ from\ table\ C - A - 7.1$$

$$\frac{KL}{r} = \frac{0.8 * 15 * 12}{3.04} = 47.3684$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 * 29000}{(47.3684)^2} = 127.5617\ Ksi > F_y = 50\ Ksi$$

$$\therefore ok\ column\ is\ in\ inelastic\ range\ and\ Euler\ formula\ is\ not\ applicable$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.4318 > \frac{KL}{r} = 47.3684$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{50}{127.5617}}\right] * 50 = 42.4346\ Ksi$$

$$P_n = F_{cr}\ A_g = 42.4346 * 21.1 = 895.3709\ k$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 0.9 * 895.3709 = 805.8338\ k$$

2. Other way to solve the problem by using table 4-22 of the AISC manual

KL/r	$\varphi_c\ F_{cr}$
47	38.3
47.3684	X
48	38

$$\frac{-0.3684}{-1} = \frac{38.3 - X}{0.3} \rightarrow X = 38.18948\ ksi$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 38.18948 * 21.1 = 805.798\ k$$

3. Other way to solve the problem by using table 4-1 of the AISC manual

$$KL = 0.8 * 15 = 12\ ft$$

$$\varphi_c P_n = 806\ k$$

❖ <u>Example 3:</u>

An HSS 16x16x1/2 with F_y = 46 ksi is used for an 18 ft long column with simple end supports.

Using an HSS 16x16x1/2 (A_g = 28.3 in^2, r_x = r_y = 6.31 in, t_{wall} =0.465 in)

From table B4.1a case 6 12

$$\frac{b}{t}=\frac{16-2*0.465}{0.465}=32.4086\ <1.40\sqrt{\frac{E}{F_y}}=1.40\sqrt{\frac{29000}{46}}$$

$$=35.1518\ \therefore section\ has\ no\ slender\ element$$

b/t also available from table 1-12 of manual

$$K=1.0\ from\ table\ C-A-7.1$$

$$(\frac{KL}{r})_x=(\frac{KL}{r})_y=\frac{1*18*12}{6.31}=34.2314$$

$$F_e=\frac{\pi^2E}{\left(\frac{KL}{r}\right)^2}=\frac{\pi^2*29000}{(34.2314)^2}=244.2578\,Ksi>F_y=46\,Ksi$$

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

$$4.71\sqrt{\frac{E}{F_y}}=4.71\sqrt{\frac{29000}{46}}=118.2608\ >\frac{KL}{r}=34.2314$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{46}{244.2578}}\right] * 46 = 42.5133\ Ksi$$

$$P_n = F_{cr}\ A_g = 42.5133 * 28.3 = 1203.1272\ k$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 0.9 * 1203.1272 = 1082.8145\ k$$

4. Other way to solve the problem by using table 4-22 of the AISC manual

KL/r	$\varphi_c\ F_{cr}$
34	38.3
34.2314	X
35	38.1

$$\frac{-0.2314}{-1} = \frac{38.3 - X}{0.2} \rightarrow X = 38.25375\ ksi$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 38.25375 * 28.3 = 1082.5803\ k$$

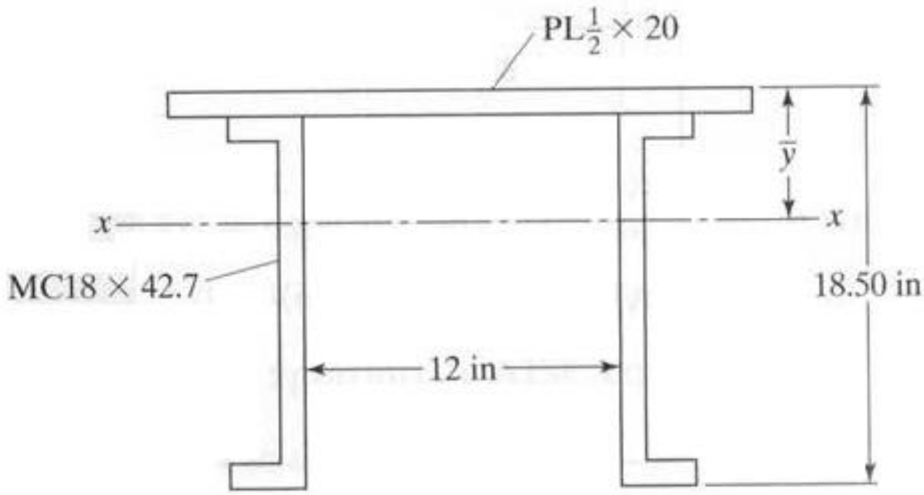
5. Other way to solve the problem by using table 4-4 of the AISC manual

$$KL = 1.0 * 18 = 18\ ft$$

$$\varphi_c P_n = 1080\ k$$

❖ Example 4:

Determine the LRFD design strength $\O_c P_n$ for the axial loaded column shown in the figure below. KL = 19 ft and 50 ksi steel is used.



Using a MC 18x42.7 (A_g = 12.6 in^2, I_x = 554 in^4, I_y = 14.3 in^4, d =18 in, $\bar{x}$ =0.877 in)

$$A_g = 20 * \frac{1}{2} + 2 * 12.6 = 35.2$$

$$\bar{y}\, from\ the\ top = \frac{20 * 0.5 * 0.25 + 12.6 * 9.5 * 2}{20 * 0.5 + 2 * 12.6} = 6.8722\ in$$

$$I_x = 2 * 554 + 2 * 12.6 * (9.5 - 6.8722)^2 + \frac{20 * 0.5^3}{12} + 20 * 0.5$$
$$* (6.8722 - 0.25)^2 = 1720.7580\ in^4$$

$$I_y = 2 * 14.3 + 2 * 12.6 * (6 + 0.877)^2 + \frac{0.5 * 20^3}{12} = 1553.7202\ in^4$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{1720.7580}{35.2}} = 6.9918$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1553.7202}{35.2}} = 6.6438\ in$$

$$\left(\frac{KL}{r}\right)_x = \frac{19 * 12}{6.9918} = 32.6096$$

$$\left(\frac{KL}{r}\right)_y = \frac{19 * 12}{6.6438} = 34.3177$$

From table 4-22 of the AISC manual

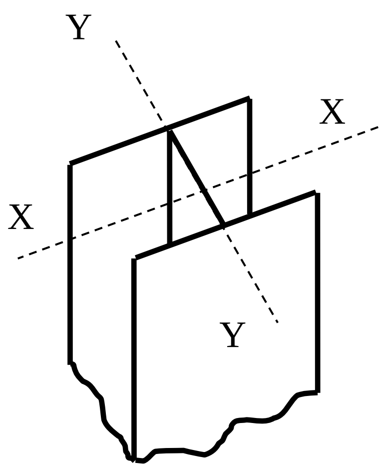
KL/r	$\varphi_c\ F_{cr}$
34	41.4
34.3177	X
35	41.2

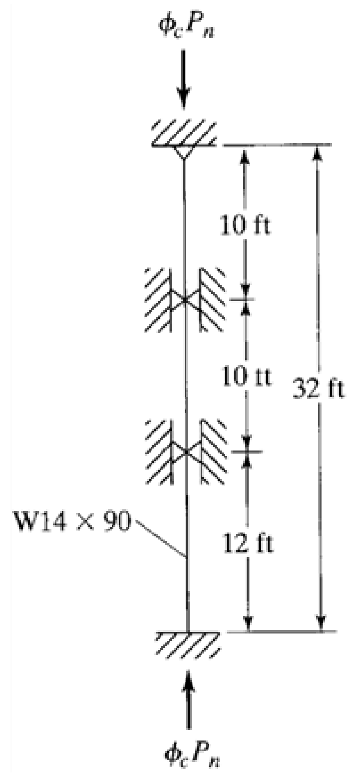
$$\frac{-0.3177}{-1} = \frac{41.4 - X}{0.2} \rightarrow X = 41.33646\ ksi$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 41.33646 * 35.2 = 1455.0434\ k$$

❖ Example 5:

Determine the LRFD design strength $\varphi_c P_n$ for the 50 ksi axially loaded W 14x90m shown in the figure below. The column is braced perpendicular to its weak, or y, axis at the point shown in the figure.





Using a W 14x 90 (A_g = 26.5 in^2, r_x = 6.14 in, r_y = 3.70 in)

Determining effective length:

$$K_x L_x = 0.8 * 32 = 25.6\ ft$$

$$K_y L_y = 1 * 10 = 10\ ft\ \ (controls\ for\ K_y L_y)$$

$$K_y L_y = 0.8 * 12 = 9.6\ ft$$

Computing the slenderness ratio:

$$\left(\frac{KL}{r}\right)_x = \frac{25.6 * 12}{6.14} = 50.0326$$

$$\left(\frac{KL}{r}\right)_y = \frac{10 * 12}{3.70} = 32.4324$$

From table 4-22 of the AISC manual

KL/r	$\varphi_c\ F_{cr}$
50	37.5
50.0326	X
51	37.2

$$\frac{-0.0326}{-1} = \frac{37.5 - X}{0.3} \rightarrow X = 37.48932\ ksi$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 37.48932 * 26.5 = 993.4585\ k$$

To solve the problem using tables 4.1

$$\frac{K_x L_x}{r_x} = \frac{Equivelent\ K_y L_y}{r_y}$$

$$Equivelent K_y L_y = r_y \frac{K_x L_x}{r_x} = \frac{K_x L_x}{\frac{r_x}{r_y}}$$

٢٢

The controlling K_yL_y to be used in the table 4.1is the larger of the real $K_yL_y = 10\ ft$, or the equivalentK_yL_y.

Form table 4-1 for W14x90:

$$\frac{r_x}{r_y} = 1.66$$

$$Equivelent K_yL_y = \frac{25.6}{1.66} = 15.42$$

$$K_yL_y = 10\ ft$$

$$= 10\ ft$$

$$= 9.6\ ft$$

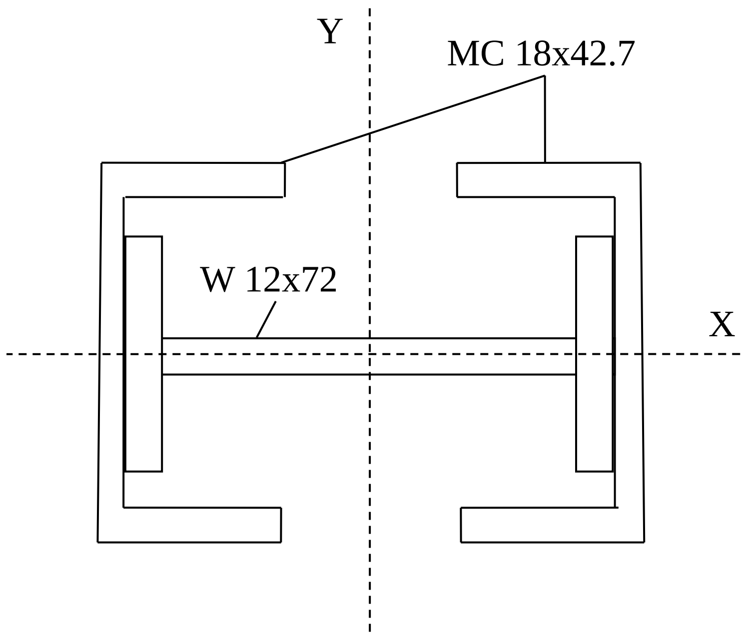
$$= 15.42\ ft\ \ (controls)$$

K_yL_y	$\varphi_c\ P_n$
15	1000
15.42	X
16	979

$$\frac{-0.42}{-1} = \frac{1000 - X}{21} \rightarrow X = 991.18\ k$$

❖ Example 6:

Determine the LRFD design strength $\varnothing_c P_n$ for the axial loaded column shown in the figure below. KL = 18.5 ft and 50 ksi steel is used.



Using a MC 18x42.7 (A_g = 12.6 in^2, I_x = 554 in^4, I_y = 14.3 in^4, d =18 in, $\bar{x}$ =0.877 in, t_w = 0.45 in)

Using a W 12x72 (A_g = 21.1 in^2, I_x = 597 in^4, I_y = 195 in^4, d =12.3 in)

$$A_g = 21.1 + 2 * 12.6 = 46.3\ in^2$$

$$out\ to\ out\ distance\ of\ the\ channels = 12.3 + 2 * 0.45 = 13.2\ in$$

$$I_x = 2 * 554 + 195 = 1303\ in^4$$

$$I_y = 2 * \left(14.3 + 12.6 * \left(\frac{13.2}{2} - 0.877\right)^2\right) + 597 = 1450.1037\ in^4$$

٢٤

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{1303}{46.3}} = 5.305$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1450.1037}{46.3}} = 5.5964\ in$$

$$\left(\frac{KL}{r}\right)_x = \frac{18.5 * 12}{5.305} = 41.8473$$

$$\left(\frac{KL}{r}\right)_y = \frac{18.5 * 12}{5.5964} = 39.6684$$

From table 4-22 of the AISC manual

KL/r	$\varphi_c\ F_{cr}$
41	39.8
41.8473	X
42	39.5

$$\frac{-0.8473}{-1} = \frac{39.8 - X}{0.3} \rightarrow X = 39.54581\ ksi$$

$$\varphi_c P_n = \varphi_c\ F_{cr}\ A_g = 39.54581 * 46.3 = 1830.971\ k$$