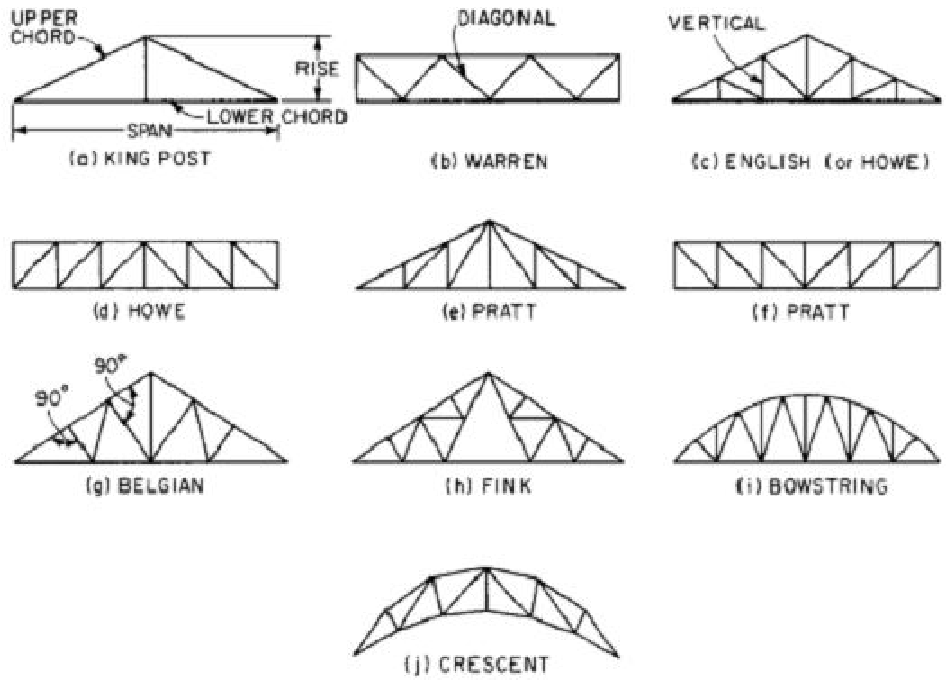
Introduction

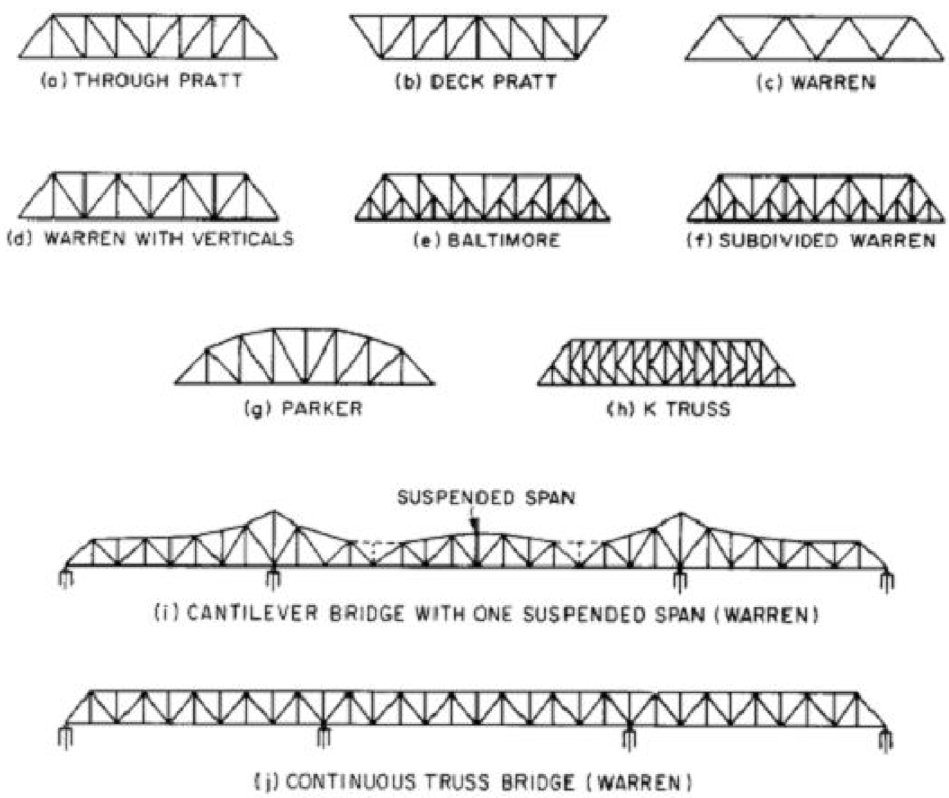
A **truss** is a structural system constructed of linear members forming triangular patterns. The members are assumed to be straight and connected to one another by frictionless hinges. All loading is assumed to be concentrated at these connections (joints or panel points). By virtue of these properties, truss members are subject only to axial load. In reality, these conditions may not be satisfied; for example, connections are never frictionless, and hence some moments may develop in adjoining members. In practice, however, assumption of the preceding conditions is reasonable. If all the members are coplanar, then the system is called a **planar truss.** Otherwise, the structure is called a **space truss.** The exterior members of a truss are called **chords,** and the diagonals are called **web members.**

Trusses often act as beams. They may be constructed horizontally; examples include roof trusses and bridge trusses. They also may be constructed vertically; examples include transmission towers and internal lateral bracing systems for buildings or bridge towers and pylons.

Trusses often can be built economically to span several hundred feet. Roof trusses, in addition to their own weight, support the weight of roof sheathing, roof beams or purlins, wind loads, snow loads, suspended ceilings, and sometimes cranes and other mechanical equipment. Provisions for loads during construction and maintenance often need to be included. All applied loading should be distributed to the truss in such a way that the loads act at the joints. Figure 1.1 shows some common roof trusses.

Bridge trusses are typically constructed in pairs. If the roadway is at the level of the bottom chord, the truss is a **through truss.** If it is level with the top chord, it is a **deck truss.** The floor system consists of floor beams, which span in the transverse direction and connect to the truss joints; stringers, which span longitudinally and connect to the floor beams; and a roadway or deck, which is carried by the stringers. With this system, the dead load of the floor system and the bridge live loads it supports, including impact, are distributed to the truss joints. Figure 1.2 shows some common bridge trusses.

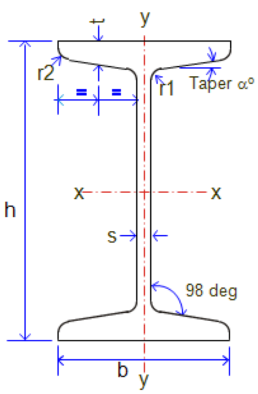




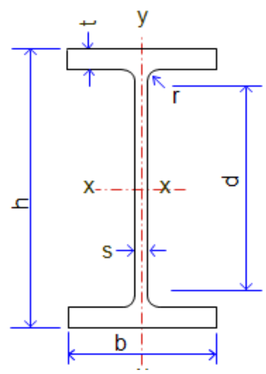
Design of steel structures

Types of steel cross sections

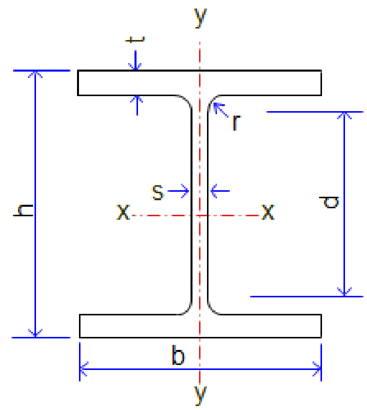
1- I-Sections: (used for beams and columns)



S.I.B

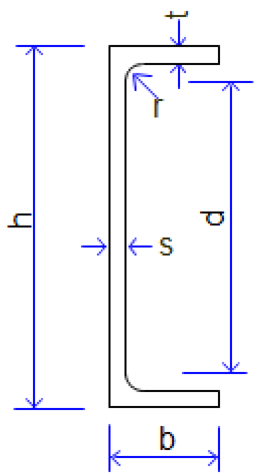


I.P.E

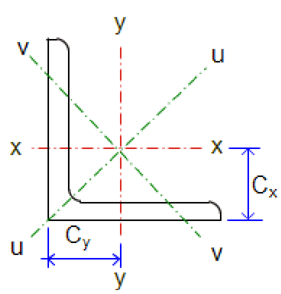


B.F.I.B

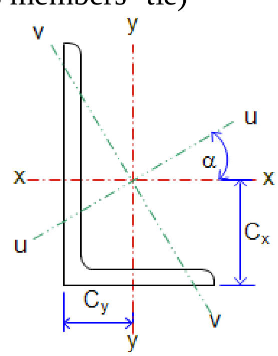
2- Channel section: used for purlins and secondary beams



3- Angles (used for axially loaded members-trusses members- tie)

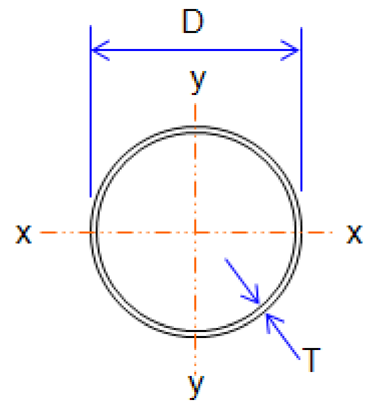


Equal angle



Unequal angle

4- Tubes (pipes): (used for truss members -space trusses-some columns- cables)



5- Plates: (used for gusset plate of connections- strengthing of members- splice plate)

- **Advantages of steel structures:**

1- High strength
2- Easy to make strengthing
3- Ductility
4- Fast in erection
5- Easy to make extension
6- Light weight

- **Disadvantages of steel structures:**

1- Maintenance cost
2- Buckling
3- Less fire resisting
4- Fatigue (in bridges)

- **Design steps of steel structures:**

1- General layout drawing
2- Loads and structural analysis (straining actions)
3- Design of members
4- Design of connections
5- Details and drawing

1- GENERAL LAYOUT

Steps of drawing of layout:

1- Main system (Truss or frame….etc) should be placed in the short direction.
2- Spacing between main system ~ (4 m – 8 m)
3- Depth of main system (H=L/12-16) L=span of main system
4- Slope of upper chord of truss (Z:1= 5:1 to 20:1) h_{min}=1.0 m
5- Panel width a=(1.5-2 m)
6- No. of panel even number (12, 16,…..etc)
7- Inclination of diagonal member (α= 35^o-55^o)
8- Choose H > a > h

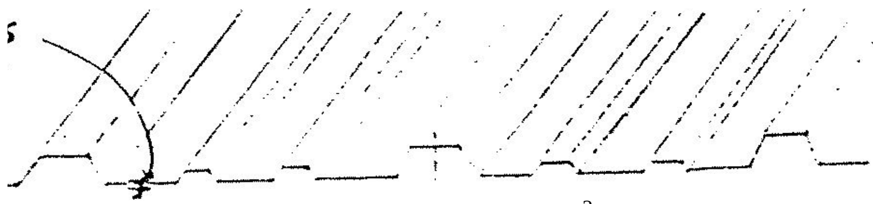
Purlins (secondary beams):

It is the direct support of covering material (steel sheets) and there are two types:

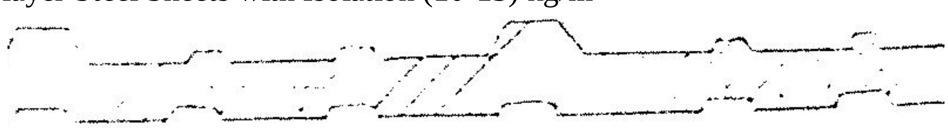
1- Rolled sections:
   wt. ~ (15-30) kg/m
2- Cold formed sections:
   wt. ~ (7- 15) kg/m

Covering material:

Single layer Steel Sheets (5- 8) kg/m^2



Double layer Steel Sheets with Isolation (10-15) kg/m^2



Decking for R.C. slab (5-8) kg/m^2

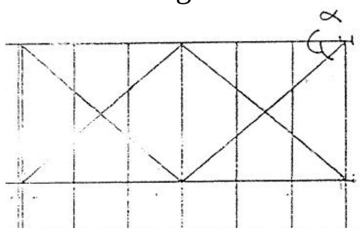
Bracing:

1- Resist any horizontal force in long direction (wind load)
2- Main supporting element during erection
3- Reduce the buckling length of upper chords outside of plane
4- Brace the whole structure in the long direction

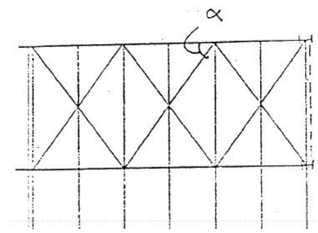
a -Horizontal bracing:

1- Used every (25-30) m in the long direction

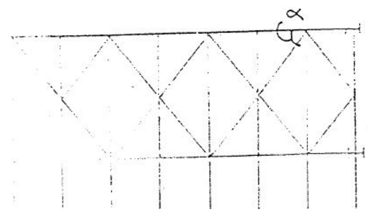
2- Works as a horizontal truss to resist the wind load and transmits to the vertical bracing



Odd/3



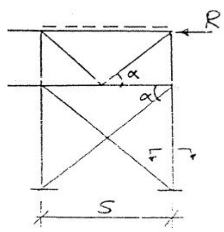
even

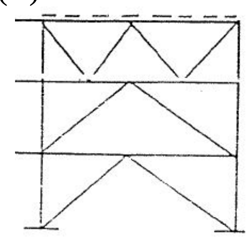


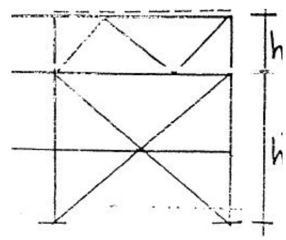
odd

b -Vertical bracing:

1- Used at each horizontal bracing
2- It is the support of horizontal bracing. It acts as a vertical to transmit the reaction of horizontal bracing (R) to the foundation





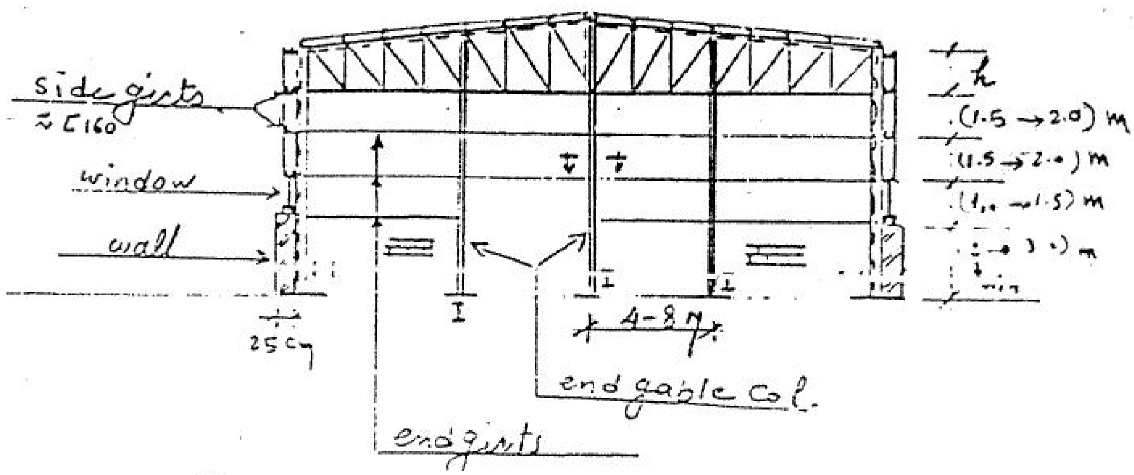


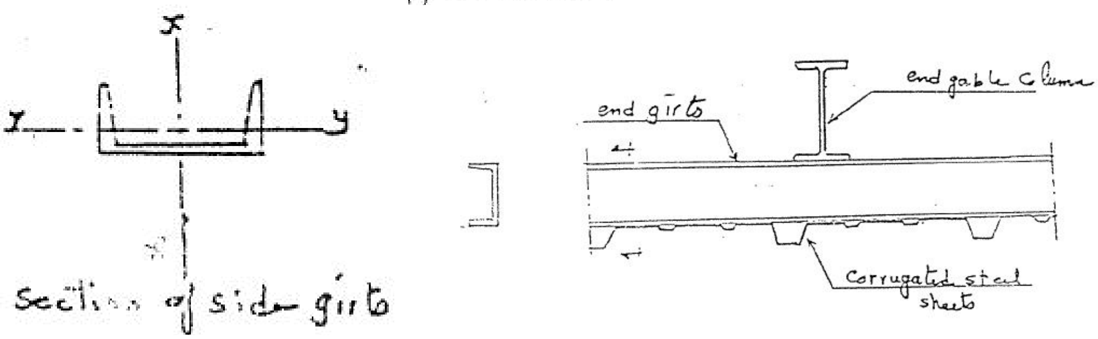
c- Longitudinal bracing:

1- Connect all trusses to work as a space truss to reduce deflection
2- Brace lower chord members out of plane

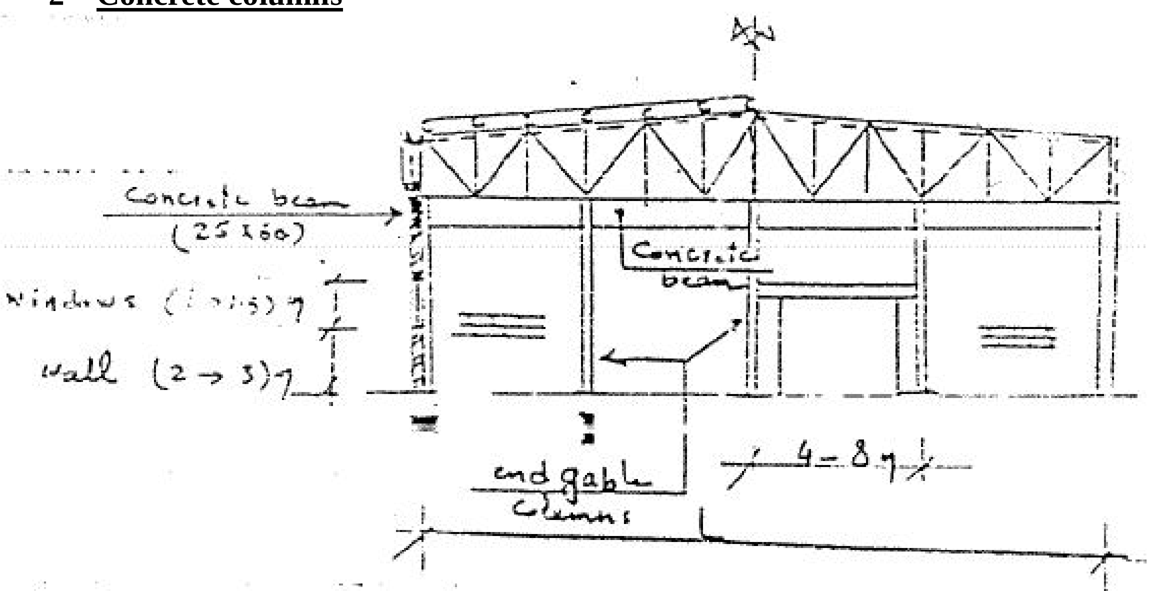
End gables:

1- Steel column

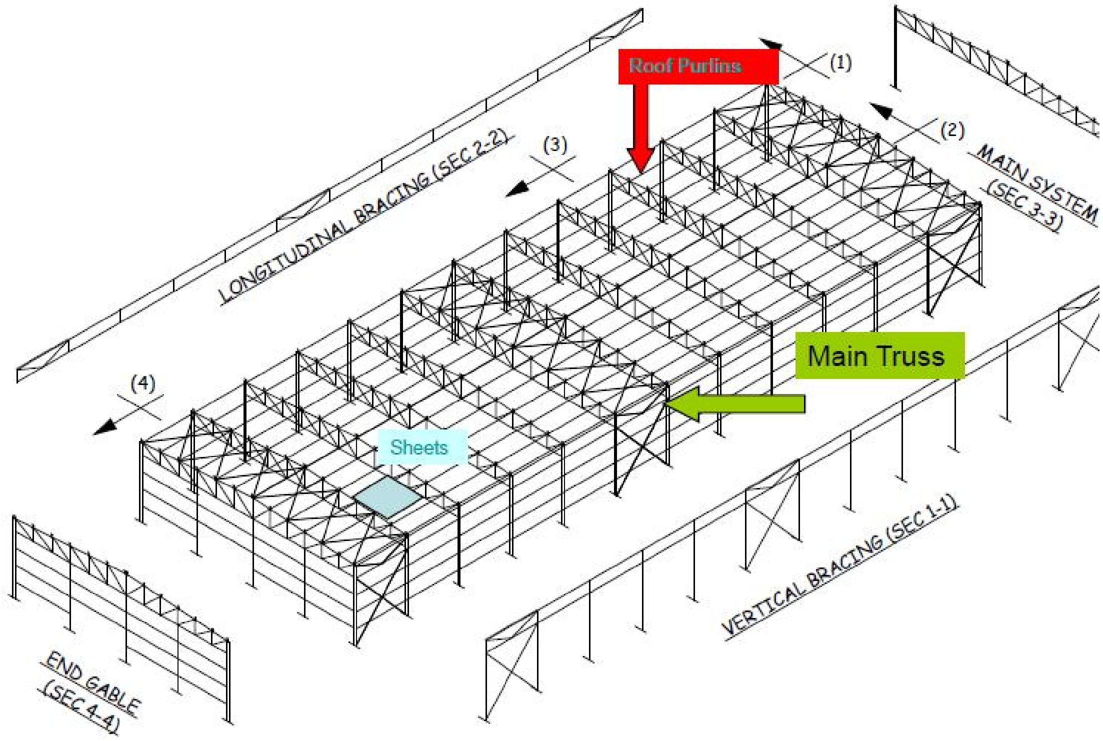


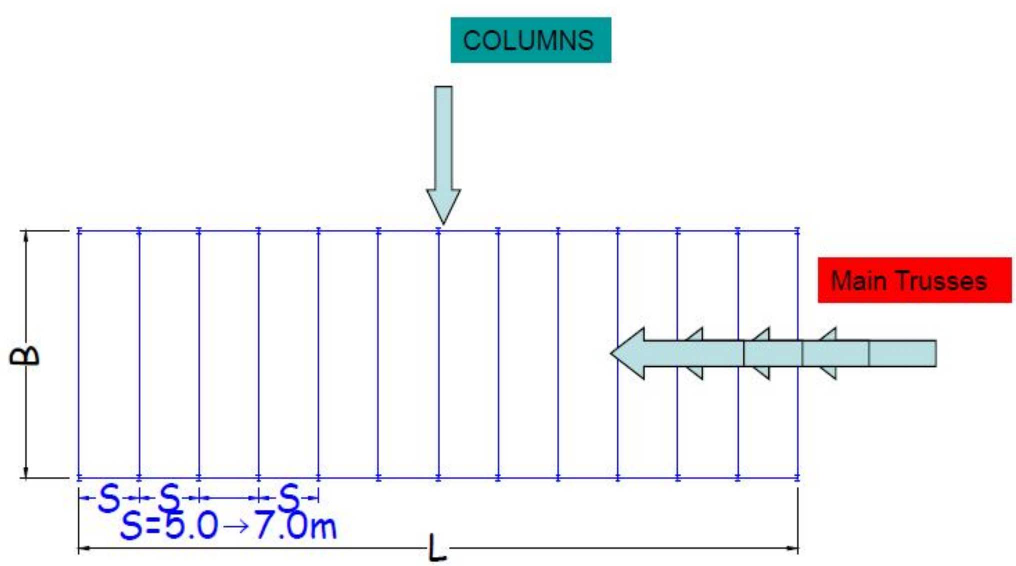


2- Concrete columns

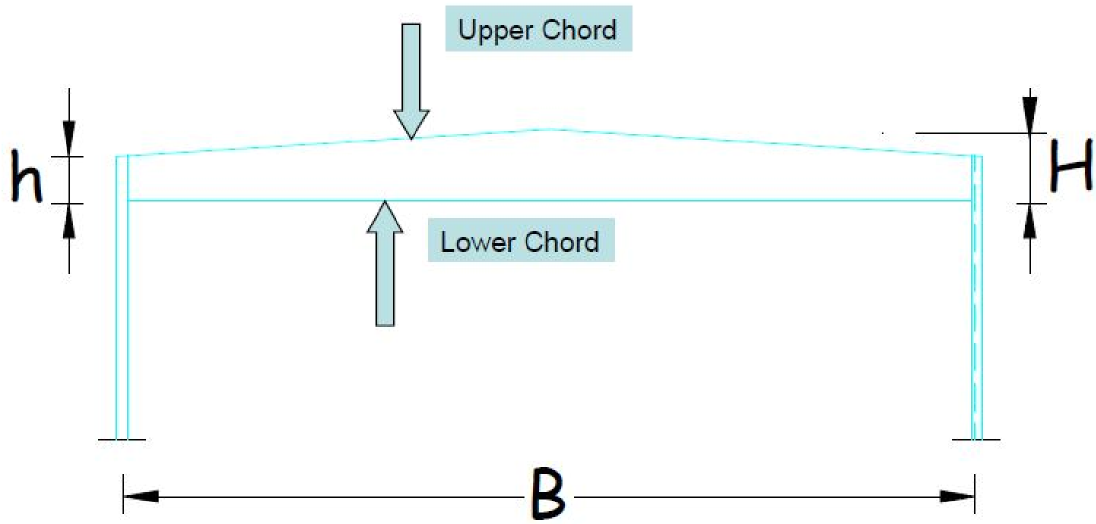


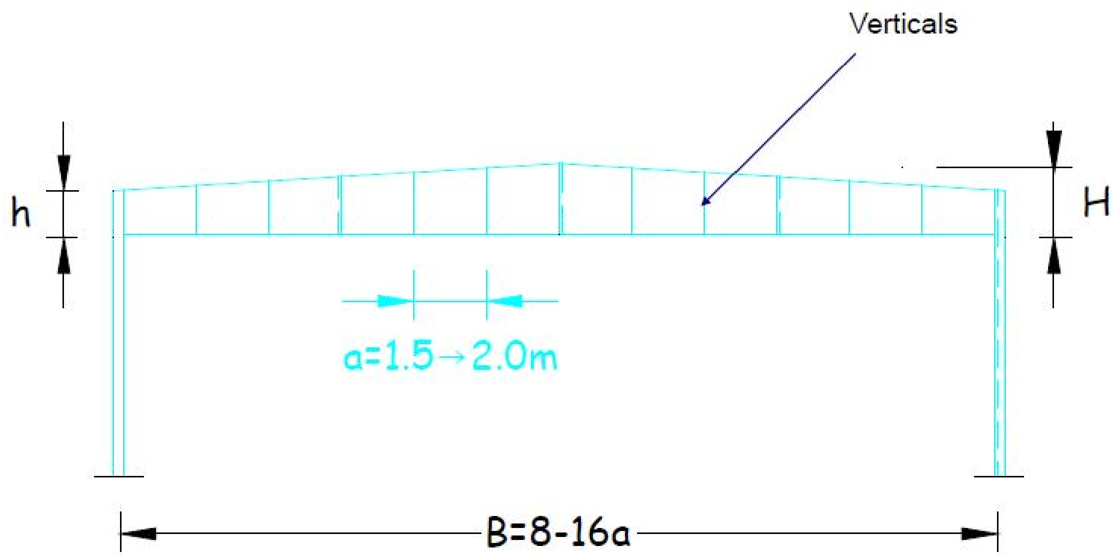
TRUSSES OVER STEEL COLUMNS



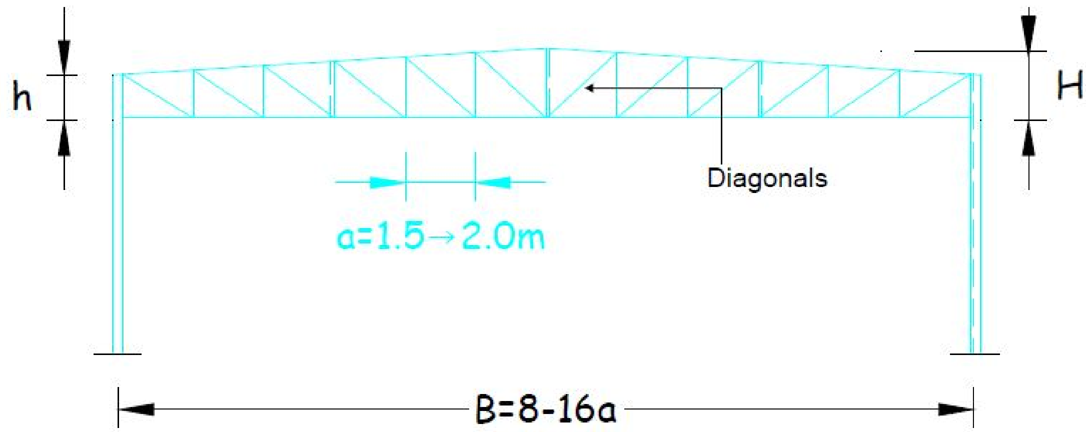


$$H = \frac{B}{12-16} \qquad h \geq 1.0m$$

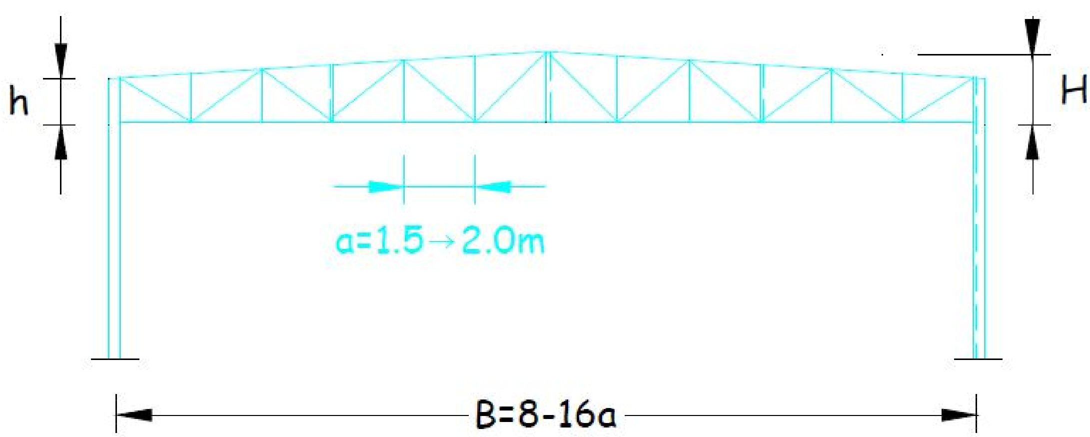




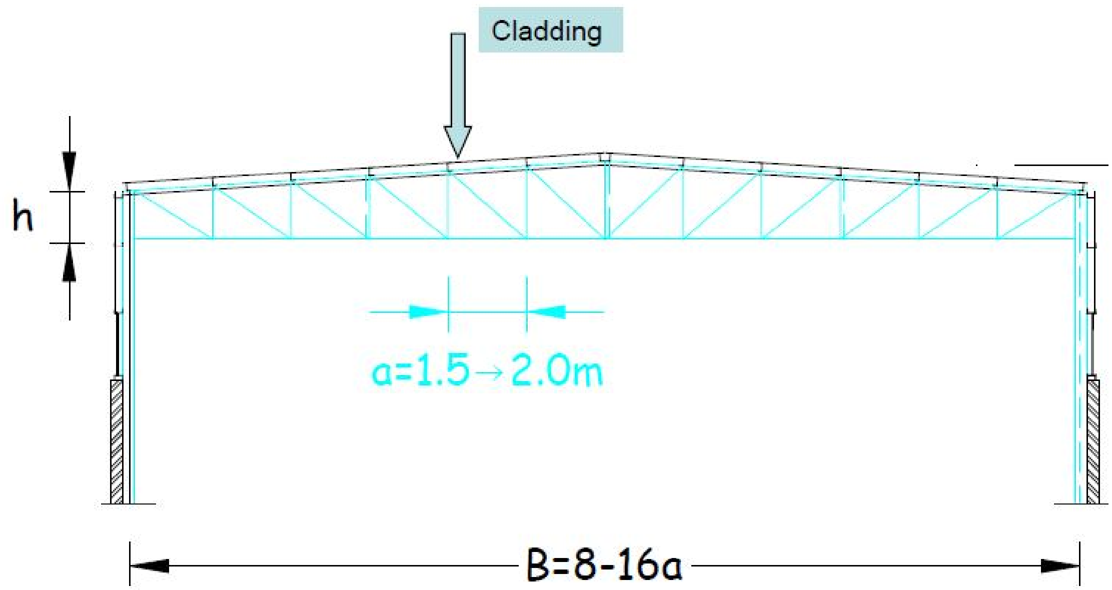
MAIN SYSTEM



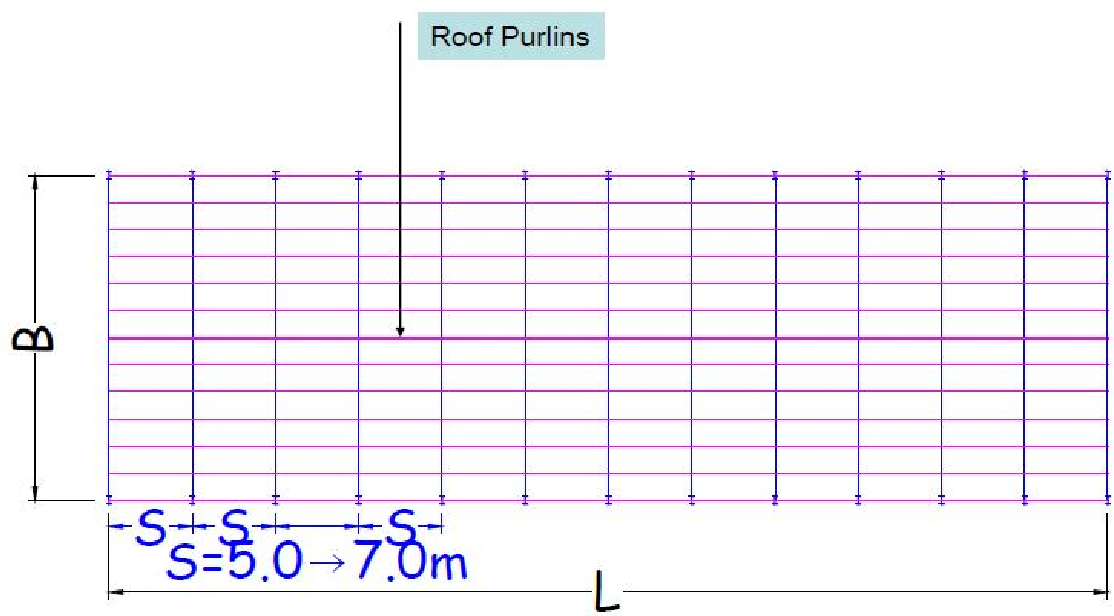
N-Truss

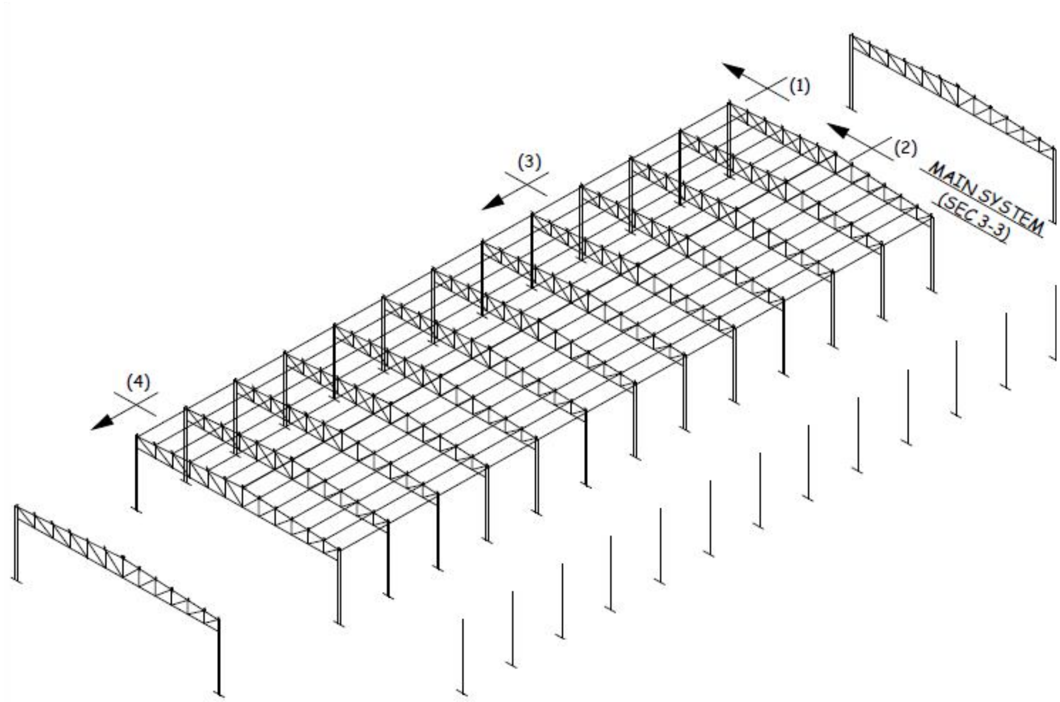


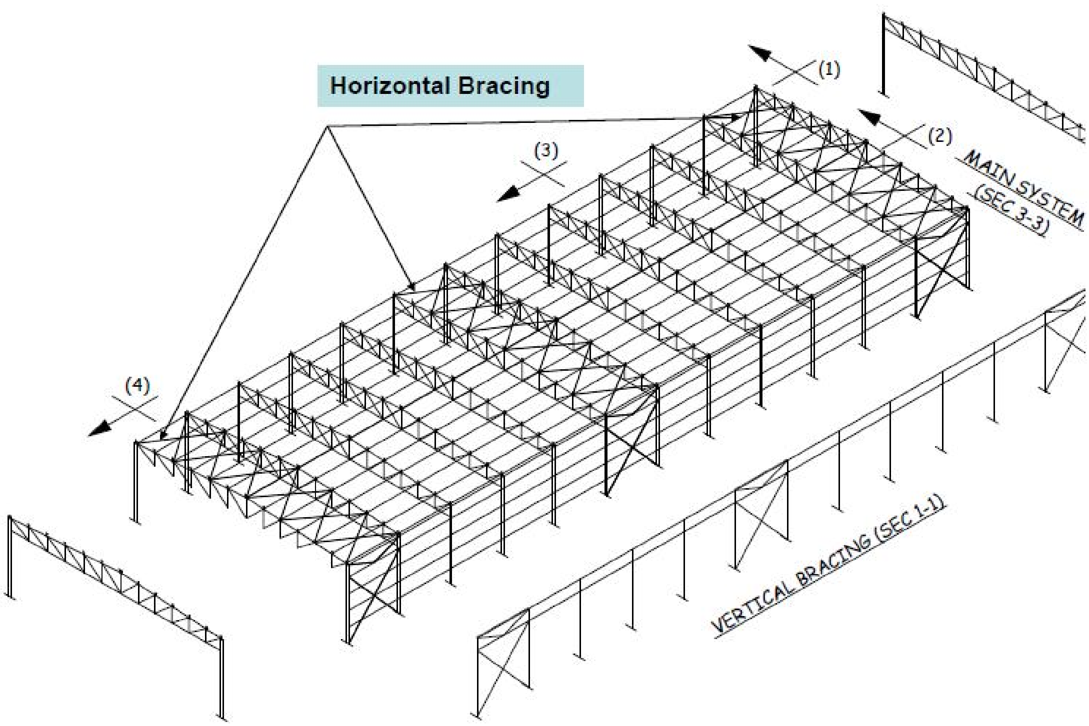
W-Truss

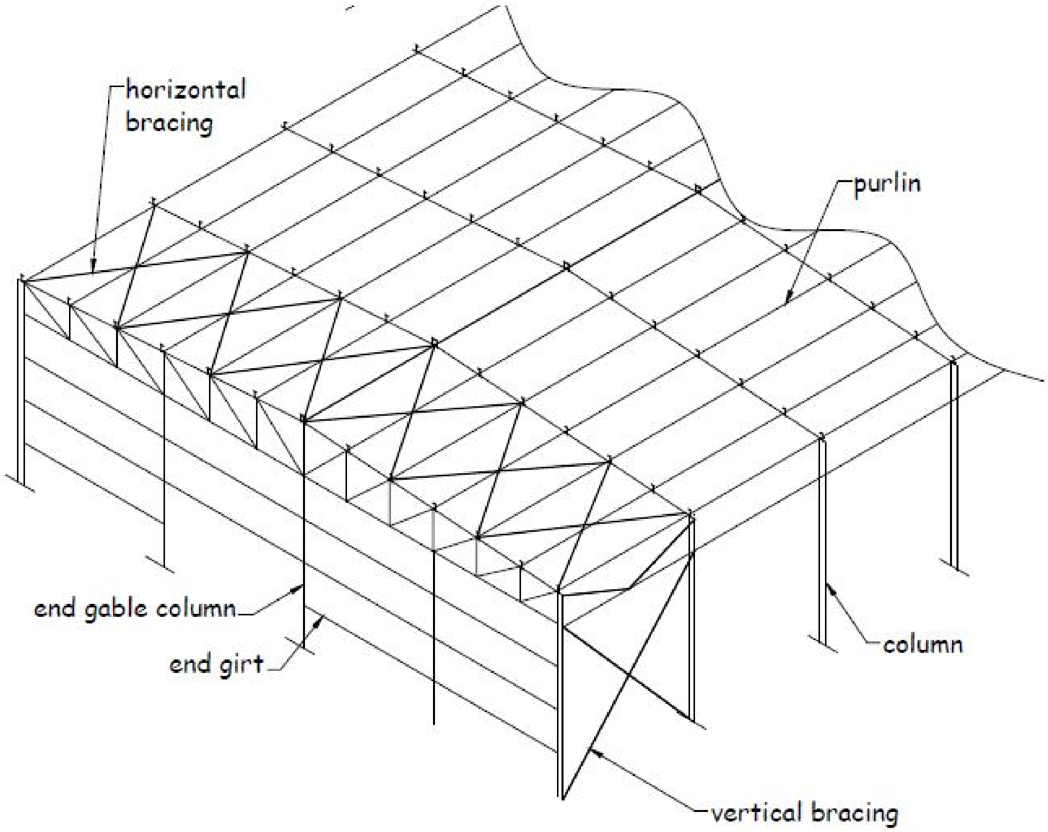


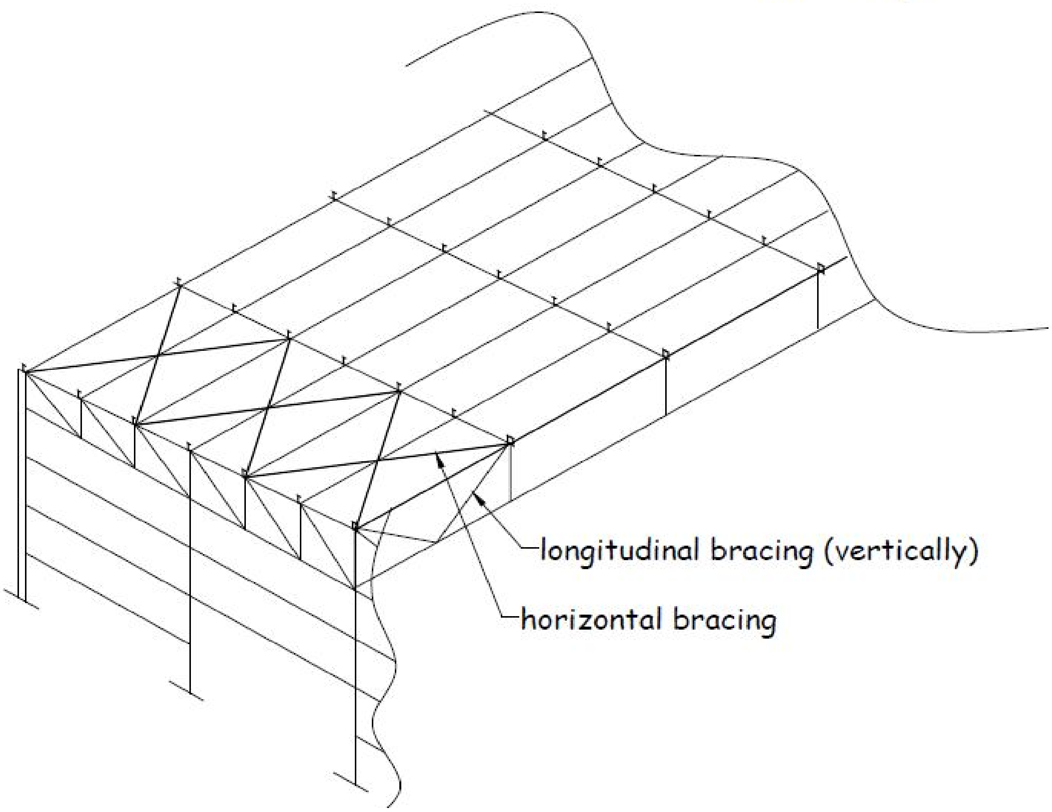
MAIN SYSTEM

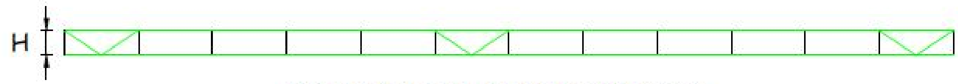



(1)
(2)
(3)
(4)
MAIN SYSTEM
(SEC 3-3)

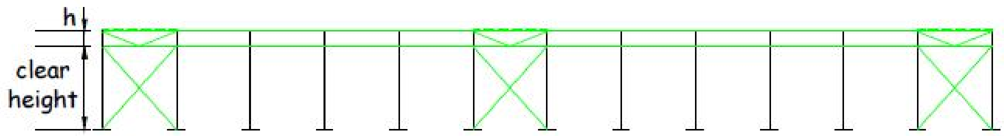

Horizontal Bracing
(1)
(2)
(3)
(4)
MAIN SYSTEM
(SEC 3-3)
VERTICAL BRACING (SEC 1-1)


horizontal bracing
purlin
end gable column
end girt
column
vertical bracing

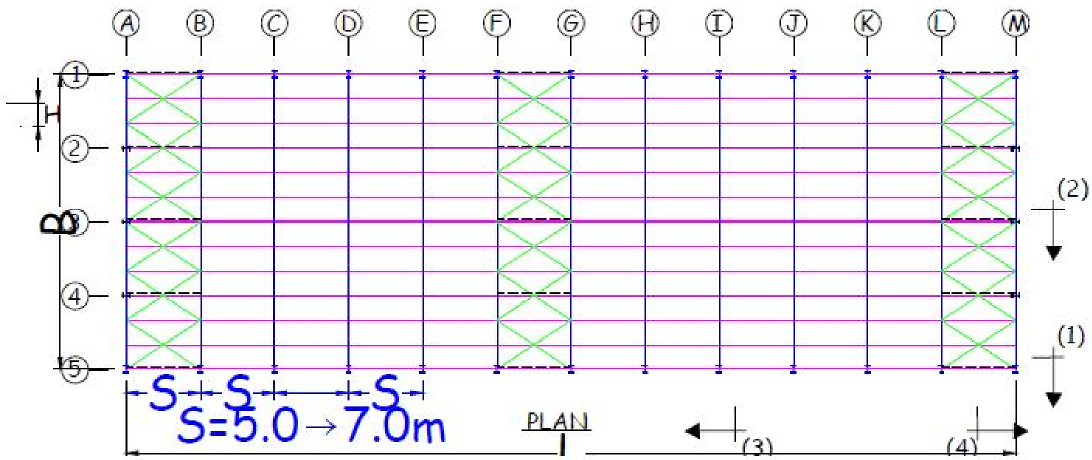

longitudinal bracing (vertically)
horizontal bracing



LONGITUDINAL BRACING (SEC 2 2)



VERTICAL BRACING (SEC 1-1)



PLAN

2. LOADS AND ALLOWABLE STRESSES

2.1 General

The structural safety shall be established by computing the actual stresses in the structural member and being sure that these stresses do not exceed the allowable (working) stresses specified below. The actual stresses are calculated used different load combinations mentioned in the “Egyptian Code of Practice for Loads and Forces for Structural Elements”. Deflections shall also be computed and shall in no case exceed the limits herein after specified.

2.2 primary and secondary stresses

The stresses shall be calculated according to the following two cases of loading:

2.1 Case I: Primary Stresses

This case of loading is the case of main loads affecting the structure. Any of the following loading types or the sum of them is considered a main “primary load”. Dead Loads, Live Loads or superimposed Loads, Dynamic Effects, Centrifugal Forces ...etc

2.2 Case II: Primary and Additional Stresses

In case of adding one or more of the loads which are classified as secondary loads to the primary loads to the previously mentioned “Case I”, the case of loading is then denoted by (case “II”). Examples of the secondary loads are (Wind Loads or Earthquake Loads, Braking Forces, Lateral Shock, Settlement of Supports in addition to the Shrinkage and Creep of concrete).

Important notes:

1. Stresses due to wind loads shall be considered as primary stresses (Case I) for such structures as transmission and microwave towers, transmission pole structures and wind bracing systems.

2. In the design of steel sections, they shall be designed according to “case I”, then they should be checked fir (case II) and the actual stresses shall in no case exceed the allowable stresses by more than 20%.

2.3 Loads

Loads are taken from the Egyptian code for claculating the loads and forces in buildings.

There are some special loads for steel structures. In the following sections, some of these loads are summarized.

2.3.1 DEAD LOAD

Dead loads are taken as the loads of the covering material in addition to the own weight of steel structure itself. In case of lack of information about the exact weight of the covering material, the following weights may be used in the structural analysis;

Single Layer Steel Sheets 5kg/m^2 to 8 kg/m^2

Double Layered Steel Sheets with Isolation 10kg/m^2 to 15 kg/m^2

2.3.2 LIVE LOAD

The minimum live loads are taken according the Egyptian code of prctice. In case of inclined roofs, live loads are considered according the angle of roof slope "α" according to the curves presented in Figure (2-1). In this figure, two curves are plooted for both accessible and inaccessible roofs. The value of the live load can be calculated using equations 2-1 and 2-2 instead of the curve.

L.L. = 60 – 66 2/3 tanα > 20 kg/m2 (for Inaccessible roofs) (2-1)

L.L. = 200 – 300 tanα > 20 kg/m2 (for Accessible roofs) (2-2)

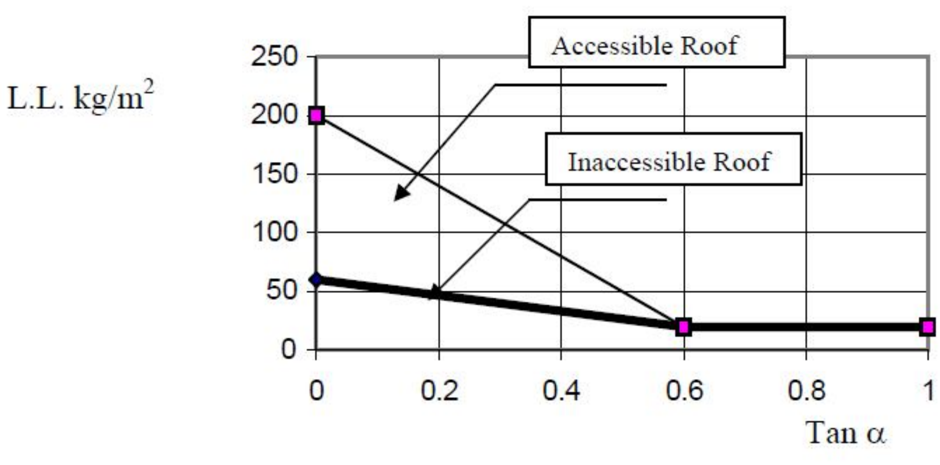


Figure (2-1) Live Loads on Inclined Roofs

2.3.3 WIND LOAD

Wind loads are the forces that affect the building in direction perpendicular to the surfaces of the buildings and structures. This force is considered positive if it is in surface direction (pressure) and negative if it is outside the surface, away from the surface direction (suction).

The external pressure or suction of wind force affecting the building surfaces is calculated using the following equation:

$$P_e = C_e \, k \, q \qquad (2\text{-}3)$$

Where;

P_e is the external design static wind pressure affecting the external unit area,

C_e is the coefficient of wind effect (pressure or suction) and it depends on the building geometric shape and is taken as shown in figures (2-2) to (2-5) for different types of frames.

k is a coefficient changes with the change of building height and taken according to table (2-1),

q is the basic wind pressure and it depends on the location of the building and is taken according to table (2-2) for areas inside Egypt.

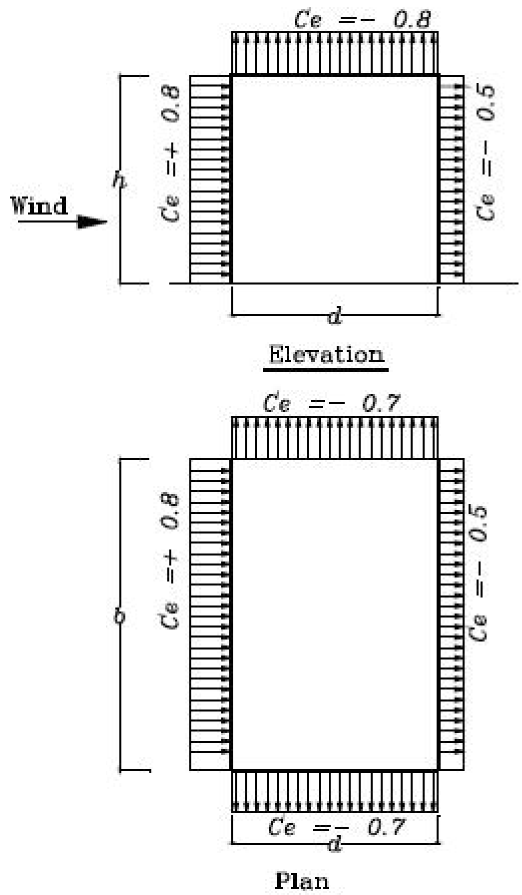


Figure (2-2) Coefficient of Wind Effect on Rectangular Frames

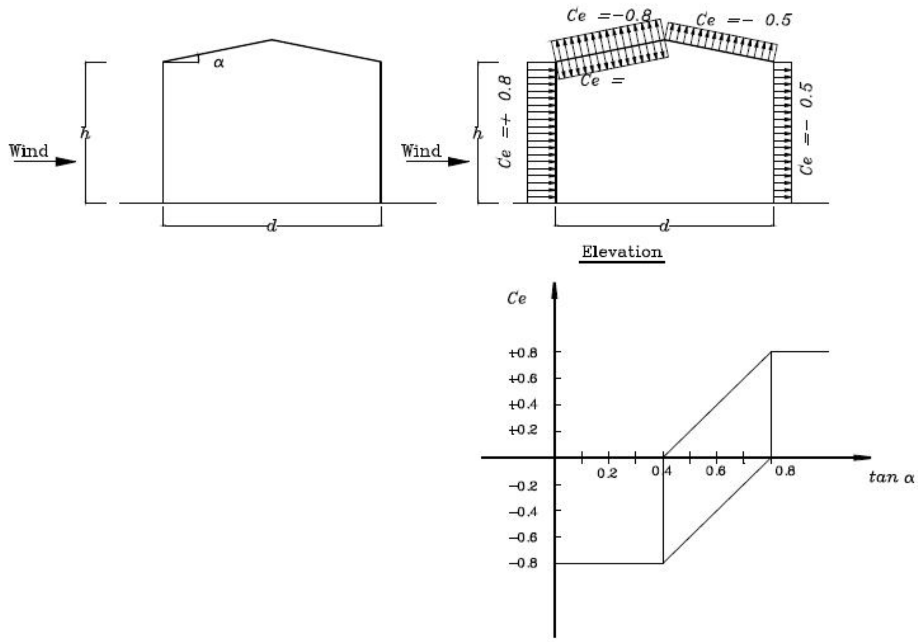


Figure (2-3) Coefficient of Wind Effect on Pitched Roof Frames

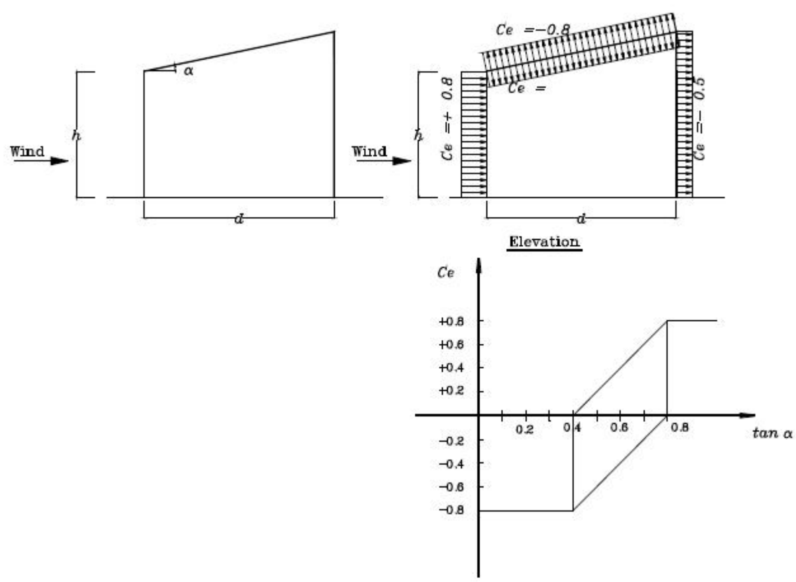


Figure (2-4) Coefficient of Wind Effect on Inclined Roof Frames

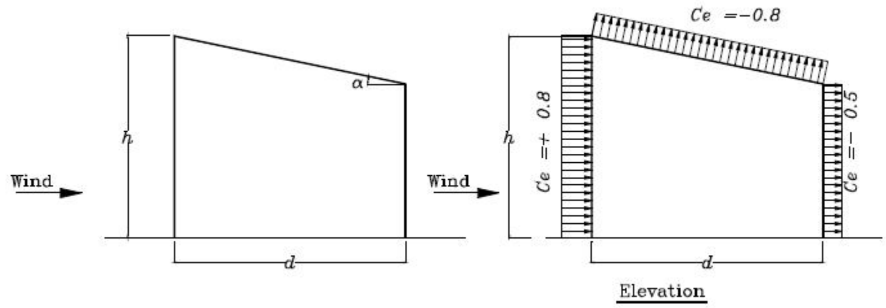


Figure (2-5) Coefficient of Wind Effect on Inclined Roof Frames

Table (2-1) Values of Coefficient *"k"*

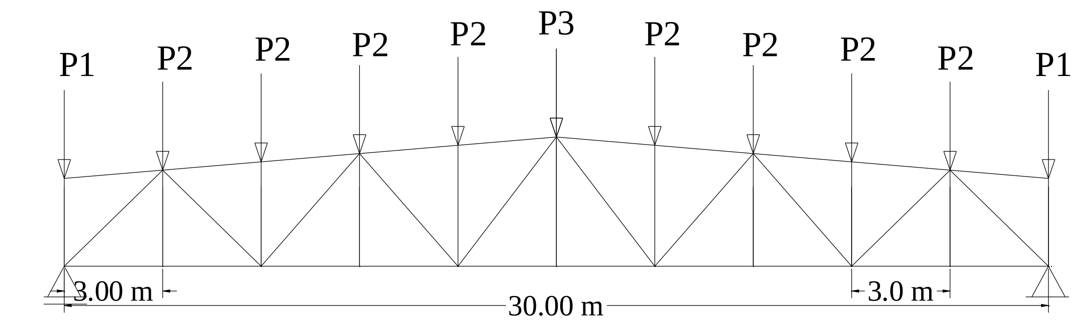
The height of building taken to calculate the coefficient *"k"* is the height of location the wind pressure is calculated at it.

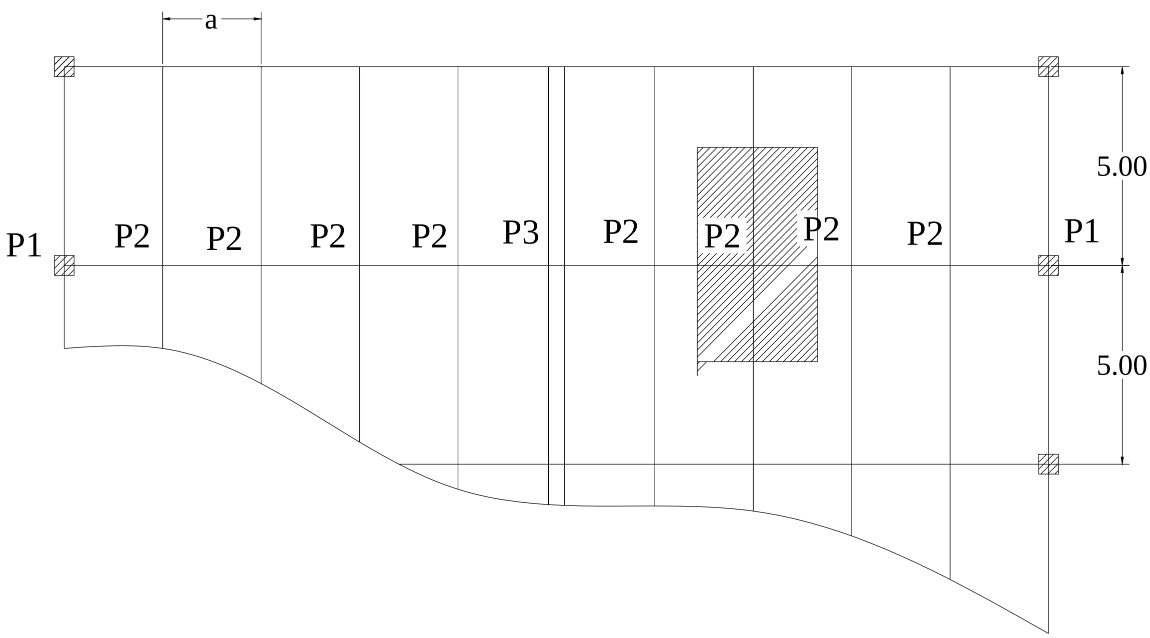
Height "m"	Coefficient *"k"*
0 to 10	1.0
10 to 20	1.10
20 to 30	1.30
30 to 50	1.50
50 to 80	1.70
80 to 120	1.90
120 to 160	2.10
More than 160	2.30

Table (2-2) Basic wind Pressure at different Locations in Egypt

Location	q "Kg/m^2"
Marsa Matrouh, El-Dabaa, Ras-Sedr	90
Alexandria, El-Saloum,Abu Sower, Hurghada and all coastal areas	80
Cairo, Asyot, Belbis	70
Fayoum, Menya, Aswan, Modereyat El-Tahrir	60
Tanta, Mansoura, Damanhour	50

Loads





1- Dead Load

P = o.w.t of corregated sheet x (panel width/cosα) x spacing between trusses + o.w.t of purlin x spacing between trusses + o.w.t of truss * panel width

P1= 5x1.5/cos α + 8 x 5 + 100 x 1.5= kg

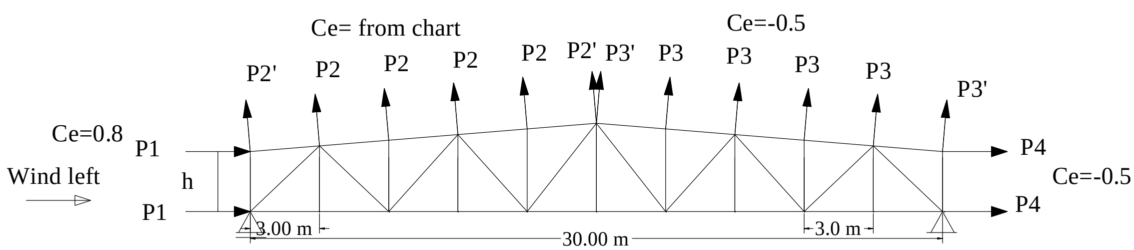
P2= 5x3/cos α + 8 x 5 + 100 x 3= kg

2- Live load

Live load intensity = (60-66.67x tan α) x spacing x panel width

P1= (60-66.67 x tan α) x 5 x 1.5= kg

P2= 2x P1 kg



3- Wind load

P= c_e k q x service area

P1= 0.8 x 1 x70 x h/2 x S kg

P2’= -0.8 x 1 x 70 x (a/2cos α) x S kg

P2= -0.8 x 1 x 70 x (a/cos α) x S kg

P3’= - 0.5 x 1 x 70 x (a/2cos α) x S kg

P3= - 0.5 x 1 x 70 x (a/2cos α) x S kg

P4= - 0.5 x 1 x 70 x (h/2) x S kg

3. DESIGN OF TENSION MEMBERS

Tension members are encounters in most steel structures. They occur as principal structural members in bridges and roof trusses, in truss structures such as transmission and microwave towers and wind bracing systems in multi-story buildings. These members can be defines as "Members which carry only axial tensile forces". Some examples of these members are shown in figure (3.1) and their cross-sections are shown in figure (3.2).

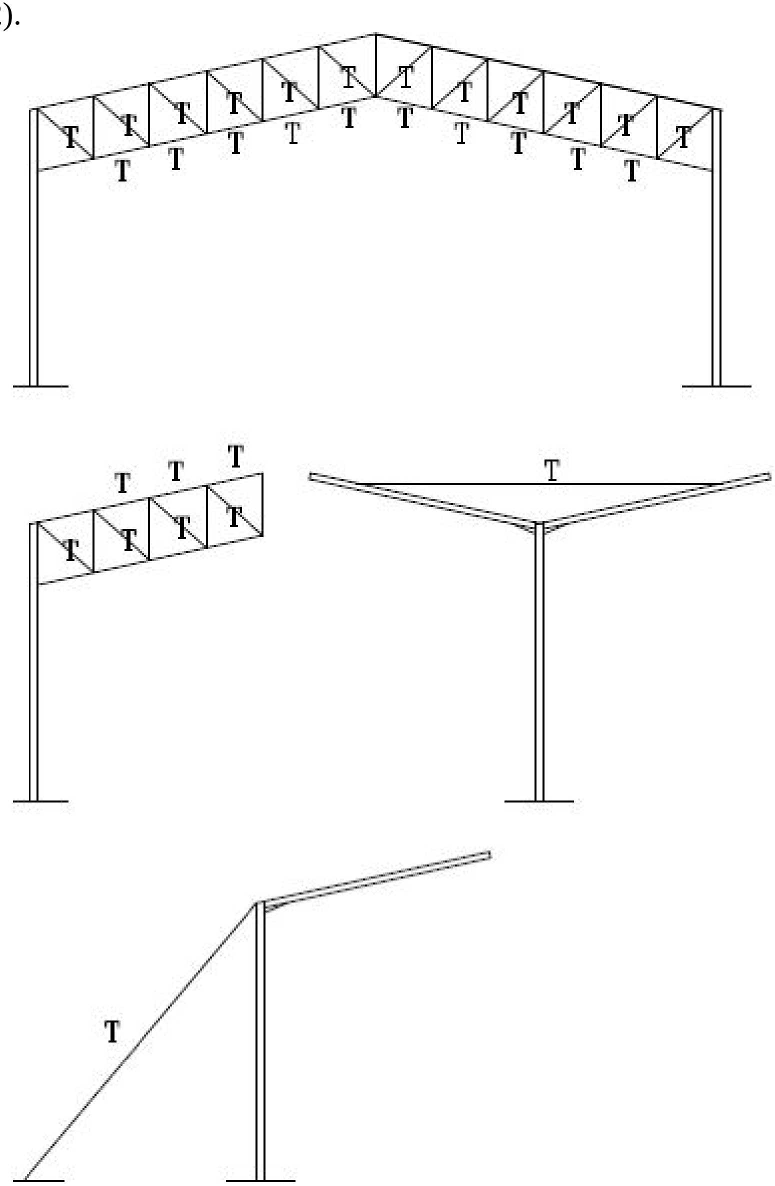


Figure (3.1) Samples of Steel Structures Containing Tension Members

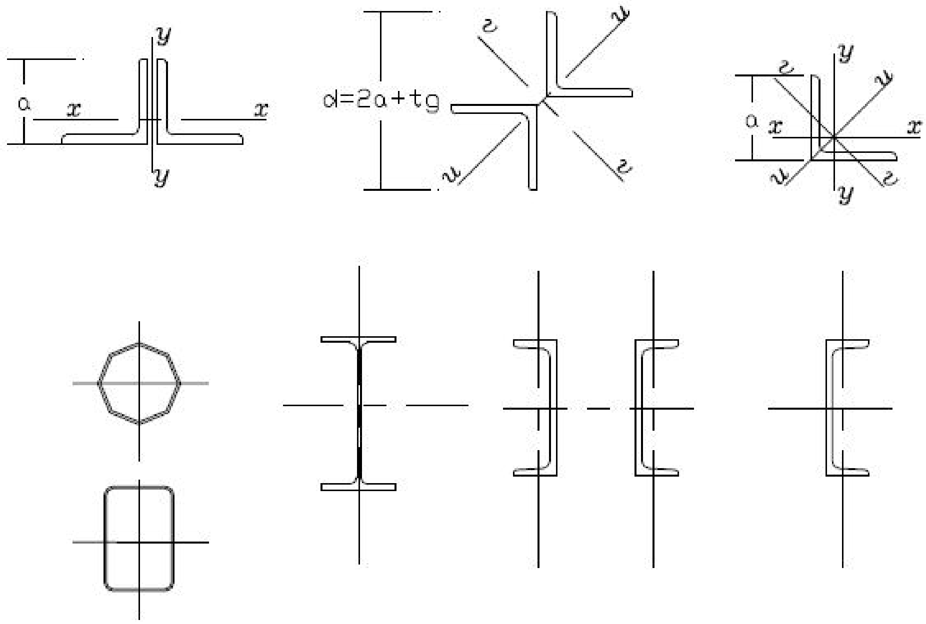


Figure (3.2) Samples of Cross-Sections of Tension Members

3.2 TENSION MEMBER RESISTANCE:

The maximum resistance of a tension member "T_r" is calculated using the following equation:

$$T_r = A_{net} \, . \, F_t \qquad (3\text{-}1)$$

where A_{net} is the effective net area of the cross section and F_t is the allowable stress of the steel in tension which is defined by the Egyptian Code of Practice, ECP 2001, as:

$$F_t = 0.58 \, F_y \qquad (3\text{-}2)$$

3.2.1 Allowable Tensile Stresses for Steel

The allowable tensile strength for steel is calculated using equation (3-2) in which, F_y is the nominal yield stress of steel. Table (3-1) presents the nominal yield stress and the nominal ultimate stress for different steel grades used in structural design.

Table (3-1) Nominal Yield Stresses and Ultimate Stresses for Different Steel Grades

Grade of Steel	Nominal Values of Yield Stress F_y and Ultimate Strength F_u Thickness "t"			
	$t \leq 40$ mm		40 mm $\leq t \leq 100$mm	
	Fy t/cm^2	Fu t/cm^2	Fy t/cm^2	Fu t/cm^2
St 37	2.40	3.60	2.15	3.40
St 44	2.80	4.40	2.55	4.10
St 52	3.60	5.20	3.35	4.90

According to table (3-1) and equation (3-2), F_t can be taken from table (3-2):

Table (3-2) Allowable Tensile Stresses for Different Steel Grades

Grade of Steel	F_t (t/cm^2)	
	$t \leq 40$ mm	40 mm $\leq t \leq 100$mm
St 37	1.40	1.30
St 44	1.60	1.50
St 52	2.10	2.0

The above values for the allowable stress, F_t, may be increased by 20% if secondary stresses are considered *(case of loading II)*.

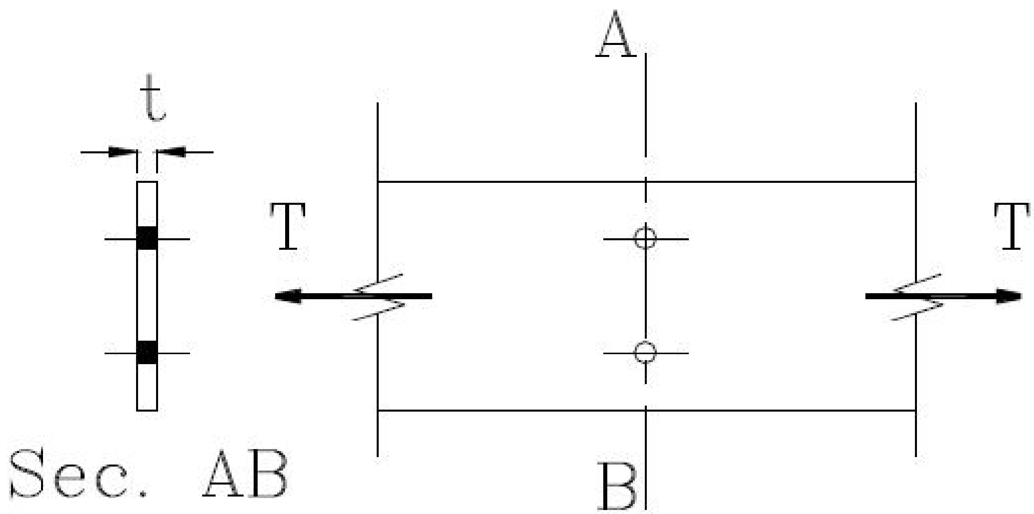
3.2.2 Effective Net Area "A_{net}"

Whenever tension member is to be fastened by means of bolts or rivets, holes must be provided at the connection. As a result, the member cross-sectional area at the connection is reduced and the strength of the member may also be reduced depending on the size and location of the holes. The effective area in resisting the tensile stress of the cross-section is called the effective net area A_{net}.

The tension member resistance must be calculated based on the effective net area of the cross-section A_{net} which can be calculated as follows:

i- For Lined up Holes:

Whenever there is only one hole or multiple holes lined up transverse to the loading direction, there is only one potential line may exist. The controlling failure line is that which passes through the bolt hole/ or holes (line AB in Figure).



$A_{net} = A_{gross} - n\,(d + 2\text{ mm})\,t$ (3-3)

Where n = number of bolts in the cross section perpendicular to the direction of loading.

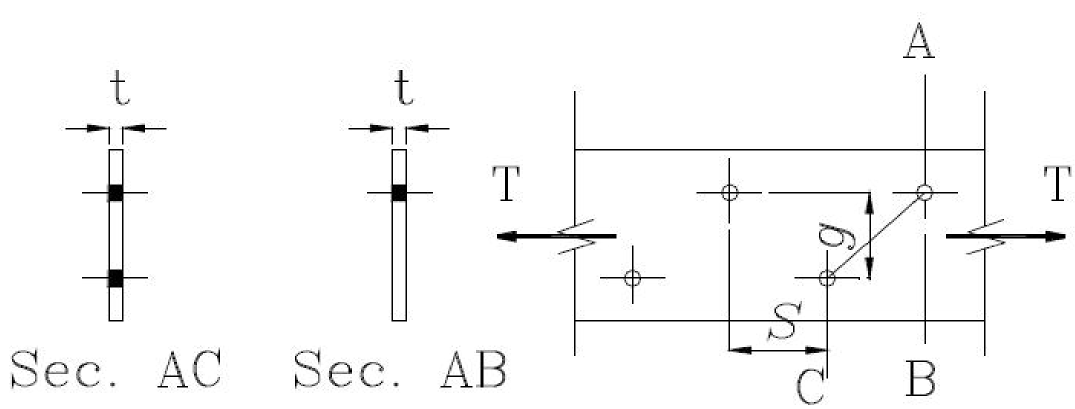
t = the thickness of plate element at which the hole lies.

d = bolt diameter

It is noted that the hole diameter exceeds the bolt diameter by 2 mm.

ii- For Staggered Holes:

Whenever there is more than one hole and the holes are not lined up transverse to the loading direction, more than one potential line may exist. The controlling failure line is that which gives the minimum area. In the opposite figure, there are two lines of staggered holes. The failure line may be through one hole (section A-B), or it might be along a diagonal path (section A-C).

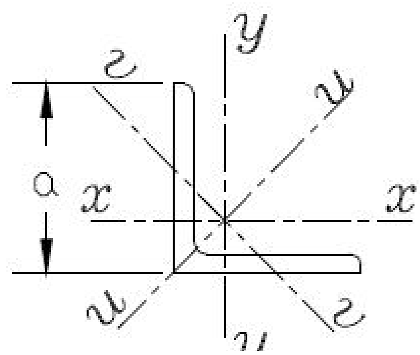


$$A_{net}\,(Path\ A\text{-}B) = A_{gross} - (d + 2mm)\ t \qquad (3\text{-}4)$$

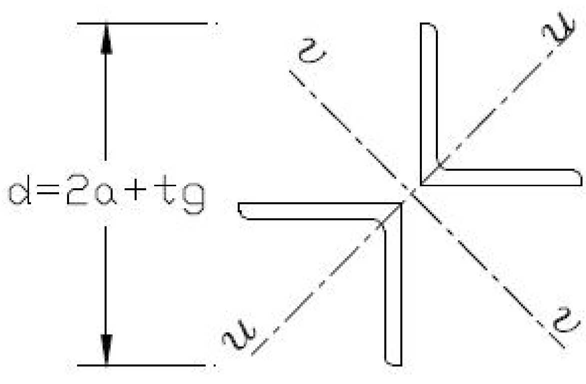
$$A_{net}\,(Path\ A\text{-}C) = A_{gross} - 2.\ (d + 2mm)\ t + (s^2/4g)\ t \qquad (3\text{-}5)$$

The net area A_{net} is the minimum of the two values.

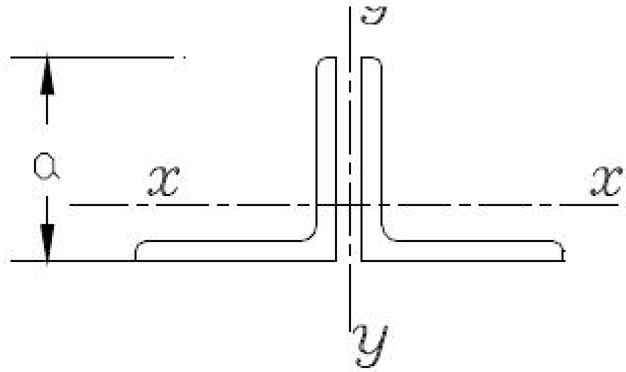
3.3 MEMBERS USED IN TRUSSES



Unsymmetric Sections (Single Angles)
Used in Vertical and diagonal members of truss
Minimum radius of gyration is $\boldsymbol{i_v = 0.2\ a}$



Symmetric Sections (Star-Shaped)
Used in Vertical members of truss at intersection with longitudinal bracing, and long tension members
Minimum radius of gyration is $\boldsymbol{i_u = 0.385\ a}$



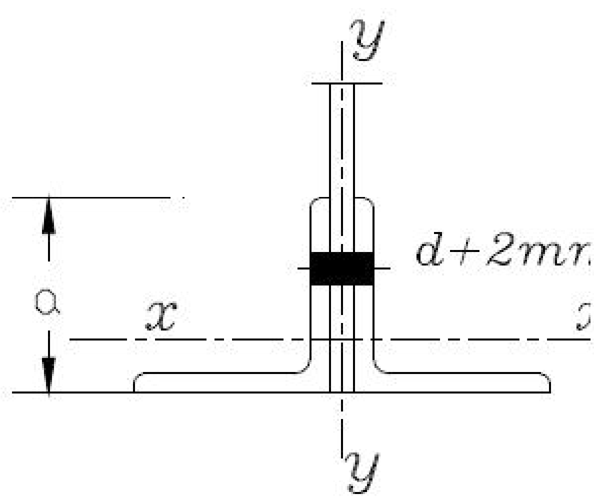
Symmetric Sections (two angles back-to-back)
Used in chord members of truss and web members of high values of force.
Minimum radius of gyration is $\boldsymbol{i_x = 0.3\, a}$
Maximum radius of gyration is $\boldsymbol{i_y = 0.45\, a}$

3.4 APPLICATION ON TRUSS MEMBERS WITH BOLTED CONNECTIONS

Truss members are considered symmetric sections when the center of gravity of the members coincides with the center of gravity of the connecting gusset plate.

3.4.1 Symmetric Sections:

Sections composed of two angles back-to-back and two angles star shaped (where there is no eccentricity at the location of the C.G. of the member and the C.G. of the gusset plate) are considered symmetric sections.



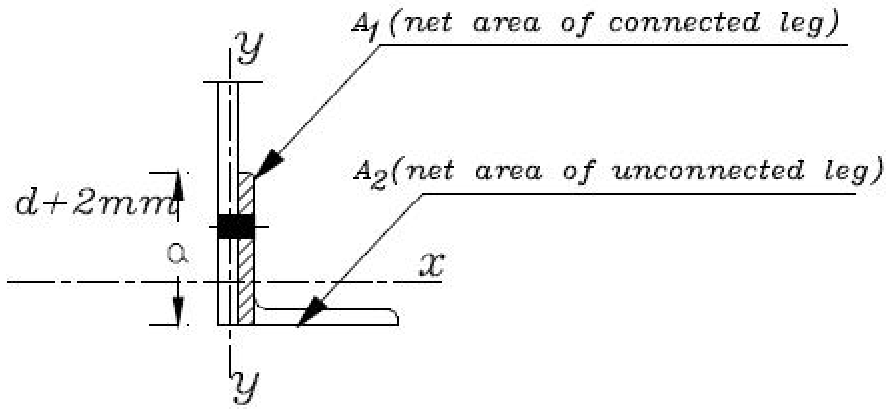
Net Area = Gross Area – Area of Bolt's holes

$A_{net} = A_{gross} - A_{Holes}$

Where the diameter of the bolt hole = diameter of the bolt + 2 mm for drilled bolts.

3.4.2 Unsymmetric Sections (single angle)

In single angle truss members where there is eccentricity between the C.G. of the member and the C.G. of the gusset plate at the location of the connection (the system shown in figure), the net area of the member is calculated as follows:

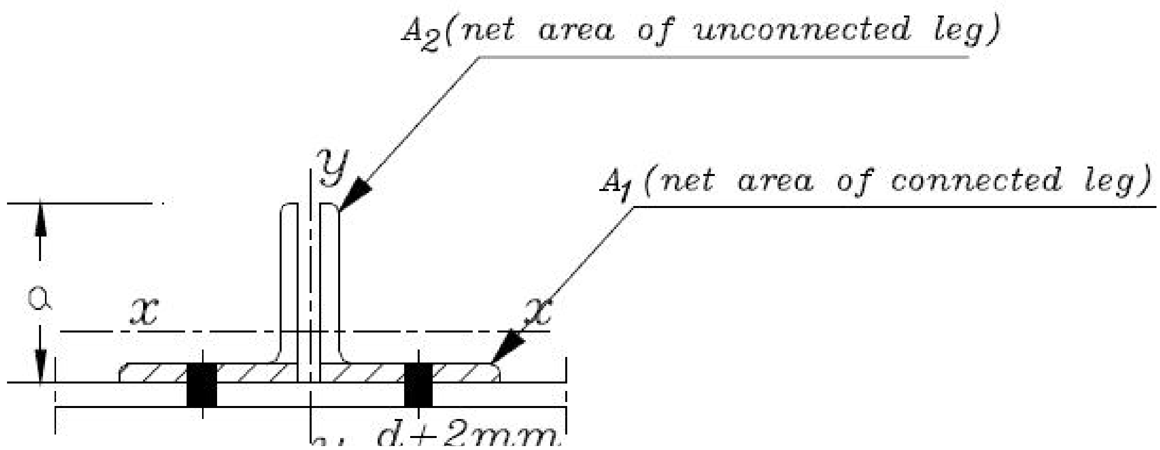


$$A_{net} = A_1 + A_2(3A_1/(3A_1+A_2)) \qquad (3\text{-}6)$$

Where A_1 is the net area of the connected leg and A_2 is the area of the unconnected leg.

3.4.3 Un-symmetric sections (double angles).

The two angles are considered un-symmetric when they lie on one side of the gusset plate as shown in figure. To allow for the eccentricity of the connection (the system shown in figure), the net area of the member is calculated as follows:

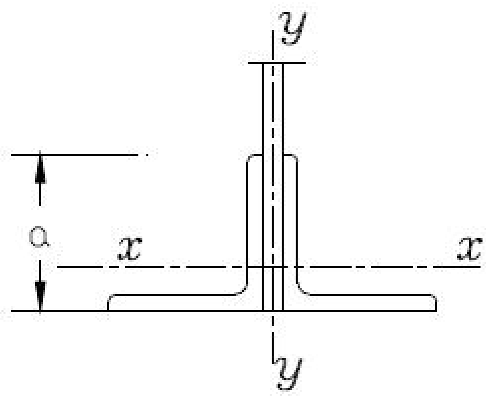


$$A_{net} = A_1 + A_2(5A_1/(5A_1+A_2)) \qquad (3\text{-}7)$$

Where A_1 is the net area of the connected leg and A_2 is the area of the unconnected leg.

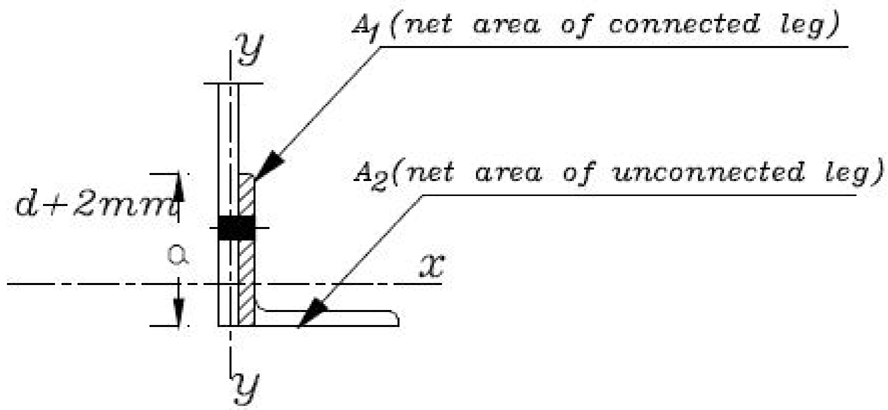
3.5 APPLICATION ON TRUSS MEMBERS WITH WELDED CONNECTIONS

3.5.1 Symmetric Sections:



$A_{net} = A_{gross}$

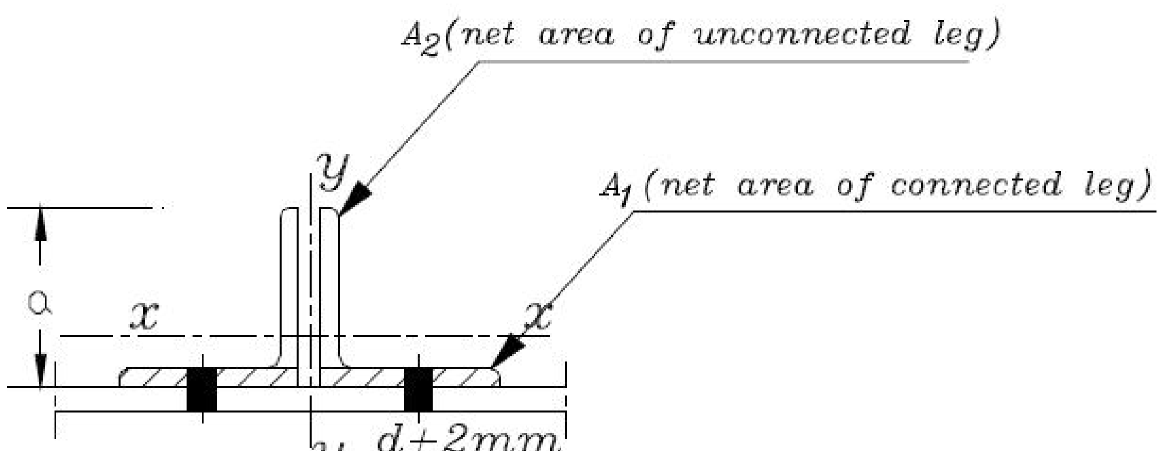
3.5.2 Unsymmetric sections (e.g. single angle):



$A_{net}= A_1+A_2\,(3A_1/(3A_1+A_2))$

Where A_1 is the net area of the connected leg and A_2 is the area of the unconnected leg (there are no bolt holes to be deducted from areas A_1 and A_2).

3.5.3Unsymmetric Sections (Double Angles):



$A_{net}= A_1+A_2\,(5A_1/(5A_1+A_2))$

Where A_1 is the net area of the connected leg and A_2 is the area of the unconnected leg (there are no bolt holes to be deducted from areas A_1 and A_2).

3.6 STIFFNESS REQUIREMENTS FOR TENSION MEMBERS:

To avoid sag, vibration and slack of tension member the ECP, 2001defined:

3.6.1 The Maximum Slenderness Ratio

The maximum slenderness ratio of a tension member (excluding wires):

1. Tension member in	$\lambda = l_{eff} / r$
Buildings	**300**
Bridges:	
Roadway Bridges	180
Railway Bridges	160
Vertical hungers	300
Bracing system	200

3.6.2 The length/depth ratio (Recommended Values)

Stiffness requirements for tension members (excluding wires) are:

Tension member in	l / d
Buildings	60
Roadway Bridges	35
Railway Bridges	30

3.7 DESIGN PROCEDURES:

Step1: Estimate the cross section:

Bolted connections – Symmetric Sections $A_{required} = \dfrac{Force}{0.85 F_t}$

Bolted connections – Unsymmetric Sections $A_{required} = \dfrac{Force}{0.85 \text{ x } 0.85 \text{ x } F_t}$

Welded connections – Symmetric Sections $A_{required} = \dfrac{Force}{F_t}$

Welded connections – Unsymmetric Sections $A_{required} = \dfrac{Force}{0.85 \text{ x } F_t}$

Then choose a suitable cross-section

Step2. Check of the Chosen Member:

1-Check of the actual Stress f_{act} $$f_{act} = \frac{Force}{A_{net}} \le 0.58Fy$$

2-Check of the length top depth ratio $\frac{l}{d} \le 60$ (for diagonals and chord members only)

3-Check member stiffness $\lambda_{max} \le 300$

$\lambda_{\max}$ is the maximum of $\lambda_{in} = \frac{l_{b-in}}{r_x}$ and $\lambda_{out} = \frac{l_{b-out}}{r_y}$ in case of two angles back-to back;

and $\lambda_{\max} = \frac{l_{b-\max}}{r_{u-1L}}$ in case of two angles star-shaped

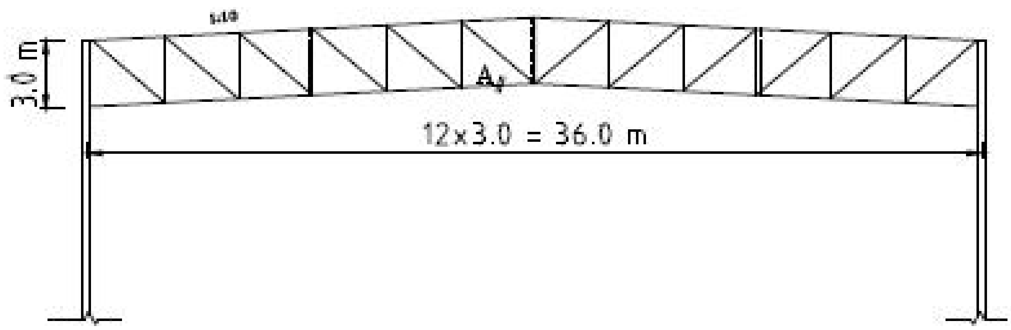
and $\lambda_{\max} = \frac{l_{b-\max}}{r_{v-1L}}$ in case of single angle sections.

4-minimum angle leg "a" shall be $\ge$ 3 d + t

Where ***d*** is the bolt diameter and ***t*** is the angle leg thickness.

Example 1:

Design the lower chord tension member "A" shown in the Figure. The force in the member is 30t (Case II) and the bolts used in the connections are 16 non-pretensioned bolts.
Data: Force = 30 t (Case II)   Length = 3.00 m Lx= 3.0 m and L_y = 9.0 m



Estimation of the cross section:

$$A_{req} = \frac{Force}{0.85F_t}$$

A_{req} = 30 / (0.85 x 1.4 x 1.20) = 21.0 cm^2
Choose 2 angles back-to-back for the lower chord. A of 1L = 21.0/2= **10.50 cm^2**
Knowing that for 2 angles back-to-back with equal legs:
r_x=0.3 *a and* r_y =   0.45 *a*
$\lambda_x = L_x / (r_x = 0.3a) = 300/0.3\,a = 300 \quad a\ req = 3.33\ cm$
$\lambda_y = L_y / (r_y = 0.45\,a) = 900/(0.45x300) = 6.67\ cm$

For construction : *a-t* > 3d = 4.8 cm.

From the tables choose 2Ls **80x80x8**

Check:

1. Strength:

$A_{net} = A_{gross} - A_{Holes}$

$A_{net} = 2\ (12.3 - (1.6+0.2) * 0.8) = 21.72\ cm^2$.

$F_{act} = 30 / 21.72 = 1.38\ t/cm^2 < 1.4x1.2$ (case II) $=1.68\ t/cm^2$

Safe

2. Stiffness:

$l/d = 300 / 8 = 37.5 \qquad < 60$ OK

$L_x/r_x = 300 / 2.42 = 124 \qquad < 300$ OK

$I_y = 2\ (I_{y\ (1L)} + A_{(1L)} * (e_y + 0.5 * t_{g\ pl})^2) = 2\ (72.3 + 12.3\ (2.26+0.5x1.0)^2) = 332\ cm^4$

$A = 2 \times 12.3 = 24.6\ cm^2$

$$r_y = \sqrt{\frac{I_y}{A_{2L}}} = \sqrt{\frac{332}{24.6}} = 3.67$$

$L_y/r_y = 900 / 3.67 = 245 \qquad < 300$ OK

3. Construction requirement:

$a = 8.0\ cm > 3\ d + t = 3x1.6+0.9 = 5.7\ cm$ OK

Example 2:

Design the same member of Example 1 with welded connections.

Estimation of the cross section:

$A_{req} = 30 / (1.4 \times 1.2) = 17.86\ cm^2$

Choose 2 angles back-to-back for the lower chord. A of 1L = 17.86/2 = **8.92 cm²**

For 2 angles back-to-back with equal legs:

r_x=0.3 a and ry=0.45 a

$r_x = L_x / (rx=0.3\ a) = 300/(0.3x300) = 3.33$ cm

$r_y = L_y / (r_y=0.45\ a) = 900/(0.45x300) =$ **6.67 cm**

From the tables choose 2Ls **70x70x7**

Check:

1. Strength:

$A_{net} = A_{gross} = 2 \times 9.4 = 18.8\ cm^2$.

f_{ca} = 30 / 18.8 = 1.595 t/cm^2< 1.4x1.2 (case II) =1.68 t/cm^2. Safe

2. Stiffness:
l/d = 300/7 = 42.85 < 60 O.K. Safe

$$\frac{l_x}{r_x} = \frac{300}{2.1} = 142.8 < 300 \quad \text{O.K.}$$

$$I_y = 2[I_{y(1L)} + A_{1L}(e_y + \frac{t_{gusset}}{2})^2] = 2[42.4 + 9.4(1.97 + \frac{1}{2})^2] = 199.5 \text{ cm}^4$$

A = 2x9.4 = 18.8 cm^2

$$r_y\sqrt{\frac{I_y}{A}} = \sqrt{\frac{199.5}{18.8}} = 3.26$$

$$\frac{l_y}{r_y} = \frac{900}{3.26} = 276 < 300 \quad \text{O.K.}$$

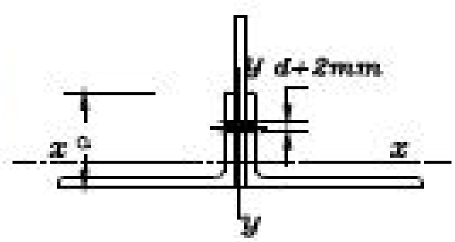
Example 3:

Design the same tension member of Example 1 using unequal angles. Use 16 non-pretensioned bolts in all the truss connections and 10 mm gusset plates.

Data: Force = + 30 t (Case II), Member length = 3.0 m, L_{in} = 3.0 m, L_{out}=9.0 m

Estimation of the cross section:
A_{req} = 30 / (0.85 x 1.4 x 1.2) = 21.0 cm^2.
Choose 2 angles back-to-back for the lower chord. Area of 1L = 21.0/2= **10.50 cm^2**

Choose 2 Ls back-to-back 65x100x7
(choose unequal angle and use the longer leg to add stiffness for the member around the Y-Y axis as shown in the Figure.



Check:
1. Strength:
A_{net} = 2 (11.2 - 1.8 x 0.7) = 19.88 cm^2.

- F_{act} = 30 / 19.88 = 1.51 t/cm^2 < 1.4x1.20 t/cm^2. Safe

2. Stiffness:

- $l/d = 300 / 6.5 = 46.2$ < 60 OK
- $l_v/r_v = 300 / 1.84 = 163$ < 300 OK

$I_y = 2\,[\,113 + 11.2\,\{\,(1.0/2 + 3.23\,\}^2] = 537.7\ cm^4$

$$r_y = \sqrt{\frac{537.7}{2x11.2}} = 4.899$$

$l_y/r_y = 900 / 4.899 = 184$ < 300 OK

3. Construction requirement:

a = 6.5 cm > 3d+t = 3x1.6+0.7 = 5.5 cm OK

Example 4

Design a diagonal member in a truss if the tensile force in the member is 6 t (Case I) and the member length is 3.6 m. The bolts used in all the truss connections are M16 non-pretensioned bolts grade 4.6, with 10 mm thick gusset plates.

Data: Force = + 6t (Case I) Length = 3.6 m.

Estimation of the member cross section:

Assume the section is single angle (unsymmetric section)

$A_{req} = 6.0 / (\,0.85 \times 0.85 \times 1.4\,) =$ **5.93 cm²**

For 1 L: $rv = 0.2$ a

$L / (rv=0.2\ a) = 300$ $a = 360/(0.2x300) =$ **6.0 cm**

for construction : a > 3d+ t $a\text{-}t > 4.8\ cm$.

To satisfy the above 3 requirements: choose **1L 60x60x6**

Check:

1. Strength:

A_1 = 6.0x0.6 - (1.6+0.2)x0.6 = 2.52 cm^2

A_2 = (6.0-0.6)x0.6 = 3.24 cm^2

$A_{net} = A_1 + A_2\{3A_1 / (3A_1 + A_2)\}$ = 2.52 + 3.24 {3 x 2.52 / (3 x 2.52 + 3.24)}
= 4.79 cm^2

- F_{act} = 6 / 4.79 = 1.25 t/cm^2 < 0.58 f_y = 1.40 t/cm^2 Safe

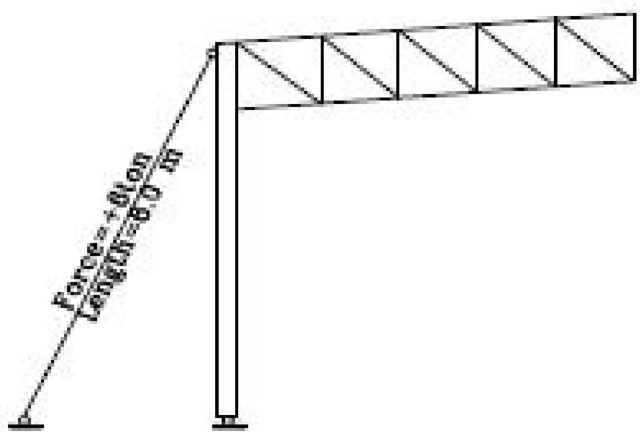
<u>2. Stiffness</u>

- l/d = 360 / 6.0 = 60 OK
- l/r_v = 360 / 1.17 = 308 > 300 unsafe

Use L **65x65x7** and recheck stiffness only:

l/r_v = 360 / 1.26 = 286 < 300 OK

Example 5
Design the shown tension member if the length of the member is 6.0 m and it carries a force of 6 t. The bolts used in the connections are M16 non-pretensioned bolts with 10 mm gusset plates.



Estimation of the member cross section:

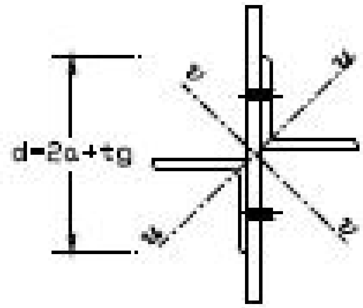
A_{req} = 6/ (0.85 x 1.4) = 5.04 cm^2
(Assuming that the cross section will be symmetric about the guest plate: Star-shaped cross-section)

A of one angle = 5.04/2 = **2.5 cm^2**
For star-shaped angle: $r_u = 0.385a$
$a = L / 0.385\,a < 300$
$a >$ 600/0.385x300 = 5.2 cm

$L/d < 60$, d> 600/60 = **10 cm** $(d = 2.a + t_g)$

Construction requirements:
a-t > 4.8 cm.

From table choose **2 Ls star-shaped 55x55x5.**



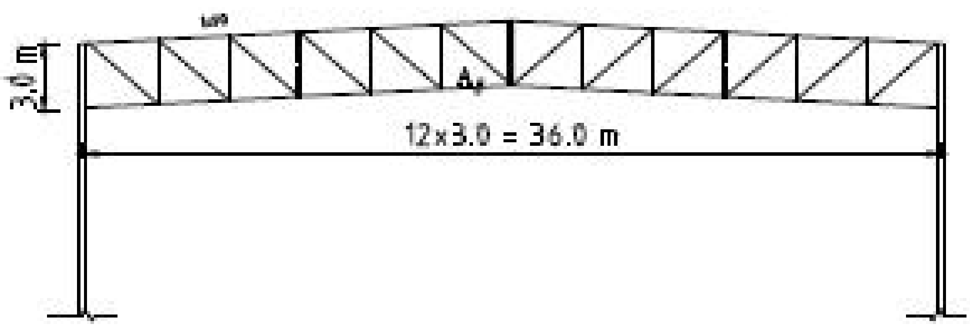
Check:

<u>1.Strength:</u>
Anet = 2 (5.32 - (1.6+0.2)x0.5) = 8.84 cm^2
- F_{act} = 6/8.84 = 0.67 t/cm^2 < 0.58 f_y t/cm^2 safe.

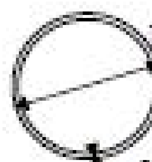
<u>2. Stiffness:</u>
l/d = 600 / (2x5.5+1.0) = 50 > 60 OK
l/r_u = 600 /2.09 = 287 < 300 OK

Example 6:

Design the lower chord tension member "A" shown in the Figure using circular hollow section (tube). The force in the member is 30t (Case II) and the member is connected to the truss members by using weld.
Data: Force = 30 t (Case II) Length = 3.00 m Lx= 3.0 m and L_y = 9.0 m



Estimation of the cross section:



$$A_{req} = \frac{Force}{F_t}$$

A_{req} = 30 / (1.4 x 1.20) = 17.86 cm^2
Choose CHS 133x5
A = 20.1 cm^2
$r_x = r_y$ = 4.53 cm

Check:

2. Strength:
A_{gross}= 20.1 cm^2
f_{act} = 30 / 20.1 = 1.49 t/cm^2 < 1.4x1.2 (case II) =1.68 t/cm^2
Safe

2. Stiffness:
l/d = 300 / 13.3 = 22.55 < 60 OK
l_x/r_x = 300 / 4.53 = 66.23 < 300 OK

4. DESIGN OF COMPRESSION MEMBERS

4.1 CLASSIFICATION OF CROSS-SECTION

In the Egyptian code of practice, the sections are classified according to local buckling of component plate elements into three section classes; compact sections, non-compact sections and slender sections as follows:

4.1.1 Class 1 (Compact Sections)

Compact sections are the sections that can achieve plastic moment capacity without the occurrence of local buckling of any of its component plate elements subjected to compressive stress.

4.1.2 Class 2 (Non-Compact Sections)

Non-compact sections are the sections that can achieve the yield moment capacity without the occurrence of local buckling of any of its component plate elements subjected to compressive stress.

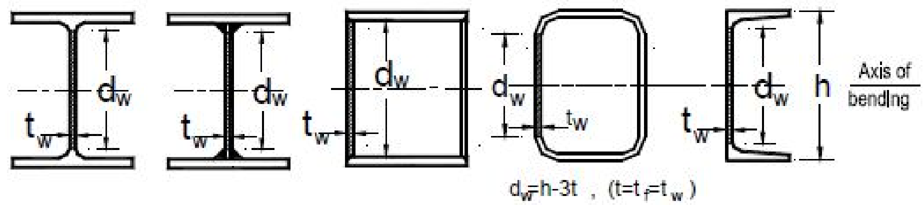
4.1.3 Class 3 (Slender Sections)

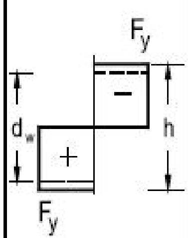
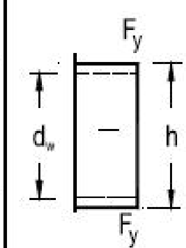
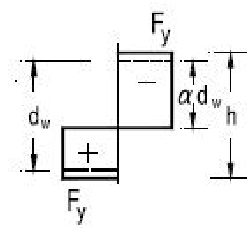
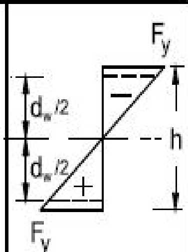
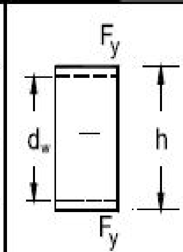
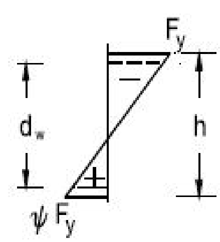
Slender sections are the sections in which local buckling of component plate elements subjected to compressive stress takes place before failure takes place. When any of the compression component elements of a cross-section is classified as class 3, the whole section shall be designed as class 3 cross-section.

The limiting plate width-to-thickness ratio for class 1 and class 2 are given in the following tables:

Table (4.1a) Maximum Width to Thickness Ratios for Stiffened Compression Elements

(a) Webs: (Internal elements perpendicular to axis of bending)

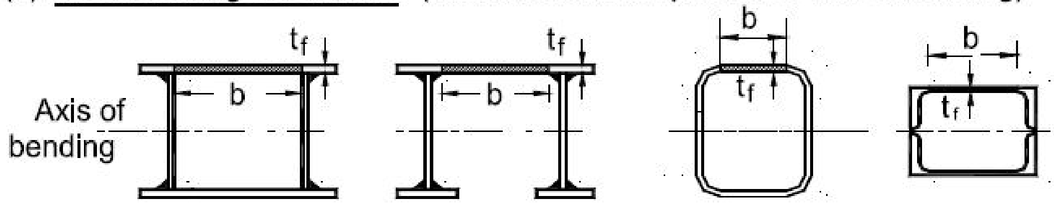


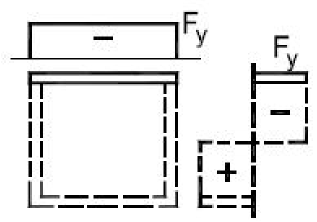
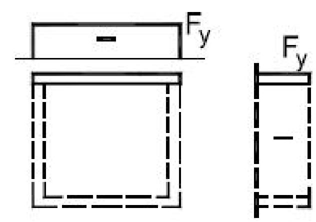
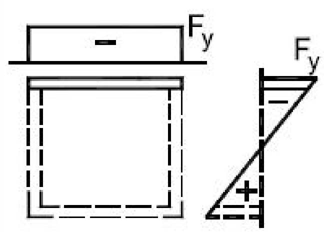
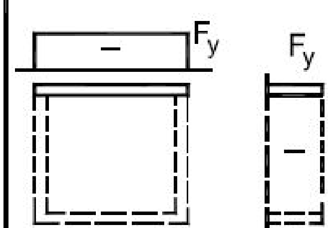
Class / Type	Web Subject to Bending	Web Subject to Compression	Web Subject to Bending and Compression	
1. Compact Stress distribution In element.				
Not for single channel	α = 0.5	α = 1.0	$\alpha > 0.5$	$\alpha \leq 0.5$
	$\frac{d_w}{t_w} \leq \frac{127}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leq \frac{58}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leq \frac{699/\sqrt{F_y}}{13\alpha - 1}$	$\frac{d_w}{t_w} \leq \frac{63.6/\alpha}{\sqrt{F_y}}$
2.Non-Compact Stress distribution In element.				
	ψ = -1	ψ = 1	$\psi > -1$	$\psi \leq -1$
	$\frac{d_w}{t_w} \leq \frac{190}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leq \frac{64}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leq \frac{190/\sqrt{F_y}}{2+\psi}$	$\frac{d_w}{t_w} \leq \frac{95(1-\psi)\sqrt{-\psi}}{\sqrt{F_y}}$

F_y in t/cm^2

Table (4.1b) Maximum Width to Thickness Ratios for Stiffened Compression Elements

(b) Internal Flange Elements: (Internal elements parallel to axis of bending)

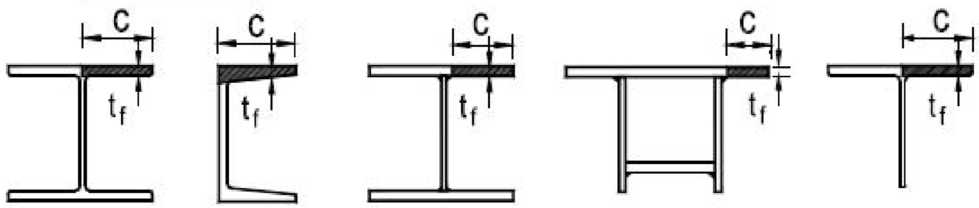


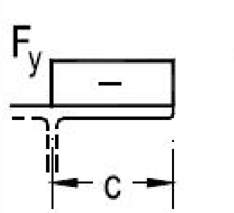
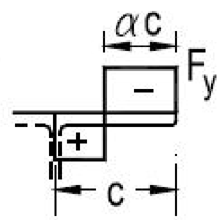
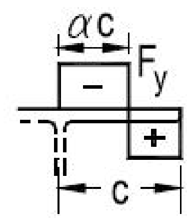
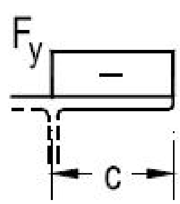
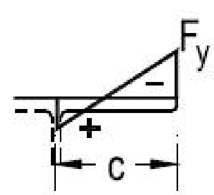
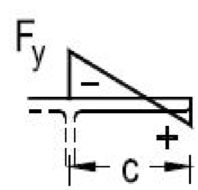
Class / Type	Section in Bending	Section in Compression
1. Compact Stess distribution in element and across section		
	$\frac{b}{t_f} \leqslant \frac{58}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$
2. Non-Compact Stess distribution In element and across section		
	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$

F_y in t/cm^2

Table (4.1c) Maximum Width to Thickness Ratios for Unstiffened Compression Elements

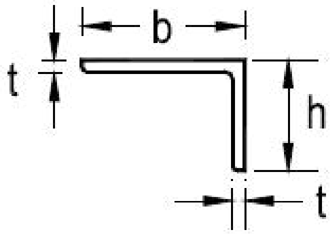
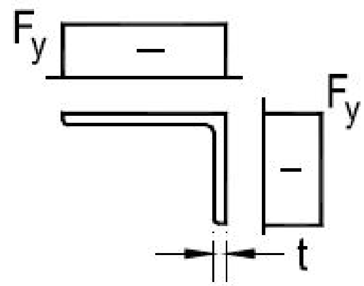
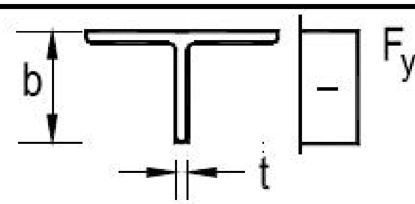
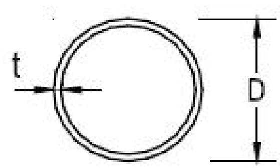
(c) Outstand Flanges



Class / Type	Flange Subject to Compression Due to M_x	Flange Subject to Compression and Bending: Tip in Comprssion	Flange Subject to Compression and Bending: Tip in Tension
1. Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{\alpha F_y}$
Welded	$\frac{c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{\alpha F_y}$
2. Non-Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 23/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 35\sqrt{K_\sigma/F_y}$	
Welded	$\frac{c}{t_f} \leqslant 21/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 32\sqrt{K_\sigma/F_y}$	

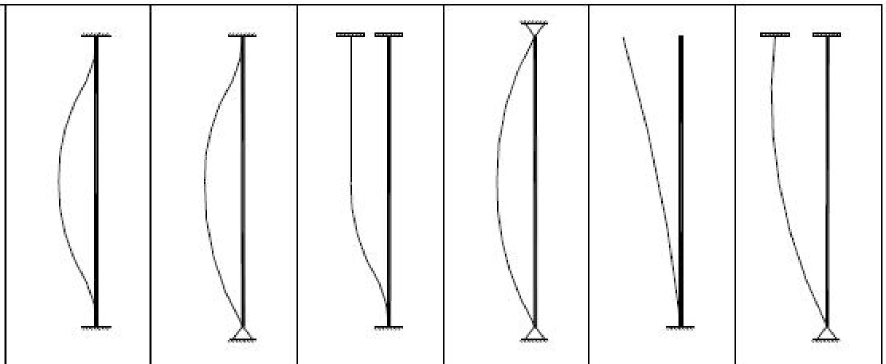
F_y in t/cm^2 For K_σ see Tables 2.3 & 2.4

Table (4.1d) Maximum Width to Thickness Ratios for Compression Elements

(d) Angles: Refer also to (Table 2.1c) "Outstand flanges"	 (Does not apply to angles in continuous contact with other components)
Class	Section In Compression
Stress distribution across section	
Non-compact	$b/t \le 23/\sqrt{F_y}$; $(b+h)/2t \le 17/\sqrt{F_y}$ (*)
Class	Section In Compression
(e) T-section:	
Non-compact	$b/t \le 30/\sqrt{F_y}$
(f) Tubular section:	
Class	Section In Bending and/or Compression
1. Compact	$D/t \le 165/F_y$
2. Non-Compact	$D/t \le 211/F_y$

F_y in t/cm^2 (*) For unequal angles

Table (4-2) Buckling Length Factor "K" for Members with Well Defined End-Conditions.

BUCKJLING MODE						
K	0.65	0.80	1.20	1.00	2.10	2.00
END CONDITIONS	ROTATION PREVENTED & TRANSLATION PREVENTED					
	ROTATION PERMITTED & TRANSLATION PREVENTED					
	ROTATION PREVENTED & TRANSLATION PERMITTED					
	ROTATION PERMITTED & TRANSLATION PERMITTED					

Tables (4-3) and (4-4) show the buckling lengths in most of the familiar compression members in trusses of buildings and bridges.

Table 4-3: Buckling Length of Compression Members in Buildings and Bridge Bracing Systems

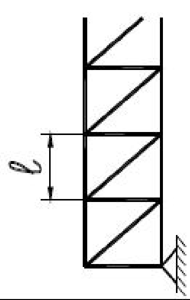
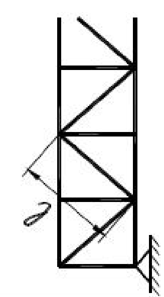
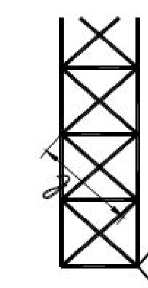
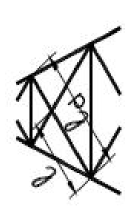
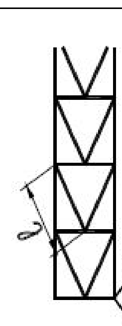
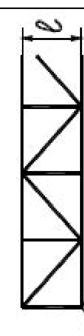
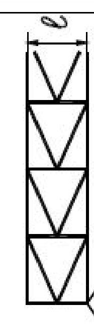
Member		In-Plane	Out-of-Plane: Compression Chord Effectively Braced	Out-of-Plane: Compression Chord Unbraced
Chords		ℓ	ℓ	0.75 span (Clause 4.3.2.2)
Diagonals -Single Triangulated web system		ℓ	ℓ	$1.2\,\ell$
-Multiple Intersected web rectangular system adequately connected		$0.5\,\ell$	$0.75\,\ell$	ℓ

Table 4-4: Buckling Length of Compression Members in Buildings and Bridge Bracing Systems (Cont.)

Member		In-Plane	Out-of-Plane: Compression Chord Effectively Braced	Out-of-Plane: Compression Chord Unbraced
Diagonals —Multiple Intersected web trapezoidal system adequately connected		ℓ	$0.8\,\ell_d$	—
– K-system		ℓ	$1.2\,\ell$	$1.5\,\ell$
Vertical members —Single triangulated web system		ℓ	ℓ	$1.2\,\ell$
—K-intersected web system		$0.5\,\ell$	$(0.75+0.25\frac{N_S}{N_L})\,\ell$	$(0.90+0.30\frac{N_S}{N_L})\,\ell$

N_S = Smaller value of compression force
N_L = Larger value of compression force

4.3 Maximum stiffness Limits

4.3.1 MAXIMUM SLENDERNESS RATIOS FOR COMPRESSION MEMBERS

The slenderness ratio of a compression member shall not exceed *max* λ given in the next table:

Maximum Allowable Slenderness Ratios of Compression Members

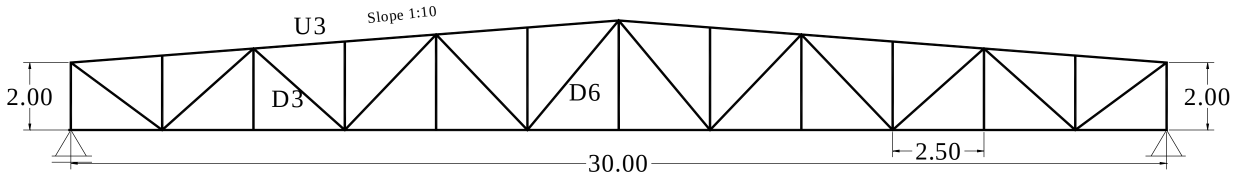
Member	λ_{max}
Buildings:	
Compression members	180
Bracing systems and secondary members	200

Design procedures:-

1- Get design compression force
2- Assume allowable stress F_{all} =(from $0.25F_y$ - to $0.4F_y$)
Or=$0.58F_y$ and factor of safety (from 1.3 to 2)
$Areq=C/F_{all}$.
3- From Tables choose sections (angles, I-shape, ….)
4- Get allowable stress according to slenderness ratios (λ_x, λ_y)
$\lambda_x = kl_x/r_x$
$\lambda_y = kl_y/r_y$
λ_{cr}= max. from (λ_x & λ_y)
if $\lambda_{cr} \leq 100$
$F_{all} = 1.4-0.000065\ \lambda_{cr}^2$ St.37
$= 1.6-0.000085\ \lambda_{cr}^2$ St.44
$= 2.1-0.000135\ \lambda_{cr}^2$ St.52
If $\lambda_{cr} > 100$
$F_{all} = 7500/\ \lambda_{cr}^2$
5- Check strength
$F_{act} = c/Area \leq F_{all}$
6- Check of required tie plate (for double angles only)
$L'/r_{min\ 1L} = L/r_{min\ 2L}$
$L'/L = (r_{min\ 1L}/r_{min\ 2L})$

Example 1:-

Design upper member chord U3 as shown in Figure1, where St. 37, 2L back to back and compression force=10 t and L=Lx=Ly=250cm.



Solution:-

1- A_{req}= 10/(0.25x2.4)=10/0.6=16.67 cm^2
   Choose 2L 70x70x7 A_{2L}= 2x9.4=18.8 cm^2
2- $\lambda x=kL_x/r_x$=250/(0.31x7)=115.21 <180
   $\lambda_y=kL_y/r_y$=250/(0.45x7)=79.37 <180
   λ_{cr}=115.21>100
   F_{all}=7500/(115.21)2=0.565 t/cm2
3- F_{act}= C/Area= 10/18.8=0.532 t/cm^2 <0.565 t/cm^2 O.k
4- Check of required tie plate
   L'= 1.36 / (0.31x7) x 250=156.68 cm
   156.68/250= 0.63
   Use one tie plate at mid

Example 2:-

Design Diagonal member D_3 as a compression member with 8t and 5t where L_{D3}=3.53 m.

Solution:-

For member D3
Areq=8/(0.25xx2.4)=8/0.6=13.33 cm2
Choose L 100x100x10 A= 19.2 cm2

3- $\lambda x=kL_x/r_x$=353/3.04= 116.12 <180
   $\lambda_y=kL_y/r_y$=1.2x 353/3.04= 139. 34 <180
   $\lambda_{cr}=kL/r_v$=1.2x353/1.95= 217 >180 unsafe
   choose 120x120x10
   $\lambda_{cr}=kL/r_v$=1.2x353/2.36= 179.49 <180

   $F_{all}=7500/(179.49)^2$=0.233 t/cm^2 Fall= 0.6x0.233=0.14 t/cm^2
   It is illogic to increase section but use 2L80x80x8 star shape for economic design
   $\lambda_{cr}=kL/r_v$=1.2x353/(0.385x8)=137.53 <180
   $F_{all}=7500/(137.53)^2$=0.396 t/cm^2
4- F_{act}= C/Area= 8/(2x12.3)=0.325 t/cm^2 <0.396 t/cm^2 O.k
5- F_{act}/F_{all}=0.325/0.396= 0.82 waste use smaller section

Tension Member (AISC-LRFD)

CHAPTER B

DESIGN REQUIREMENTS

This chapter contains provisions which are common to the Specification as a whole.

B1. GROSS AREA

The gross area A_g of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member. For angles, the gross width is the sum of the widths of the legs less the thickness.

B2. NET AREA

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

In computing net area for tension and shear, the width of a bolt hole shall be taken as $\frac{1}{16}$-in. (2 mm) greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$

where

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

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

In determining the net area across plug or slot welds, the filler metal shall not be considered as adding to the net area.

B3. EFFECTIVE AREA OF TENSION MEMBERS

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

(1) When tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective area A_e is equal to the net area A_n.

(2) When the tension load is transmitted by fasteners or welds through some but not all of the cross-sectional elements of the member, the effective area A_e shall be computed as follows:

(a) When the tension load is transmitted only by fasteners

$$A_e = A_n U \tag{B3-1}$$

where

U = reduction coefficient
$= 1-(\bar{x}/l) \le 0.9$
$\bar{x}$ = connection eccentricity, in. (mm)
l = length of the connection in the direction of loading, in. (mm)

(b) When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds

$$A_e = A_g U \tag{B3-2}$$

where

$U = 1-(\bar{x}/l) \le 0.9$
A_g = gross area of member, in.^{2} (mm^2)

(c) When the tension load is transmitted only by transverse welds

$$A_e = AU \tag{B3-3}$$

where

A = area of directly connected elements, in.^{2} (mm^2)
$U = 1.0$

(d) When the tension load is transmitted to a plate only by longitudinal welds along both edges at the end of the plate

$$A_e = A_g U \tag{B3-4}$$

where

For $l \ge 2w$ . . . . $U = 1.00$
For $2w > l \ge 1.5w$ . . . . $U = 0.87$
For $1.5w > l \ge w$ . . . . $U = 0.75$

where

l = length of weld, in. (mm)
w = plate width (distance between welds), in. (mm)

Larger values of U are permitted to be used when justified by tests or other rational criteria.

For effective area of connecting elements, see Section J5.2.

CHAPTER D

TENSION MEMBERS

This chapter applies to prismatic members subject to axial tension caused by static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Section H1.1. For threaded rods, see Section J3. For block shear rupture strength at end connections of tension members, see Section J4.3. For the design tensile strength of connecting elements, see Section J5.2. For members subject to fatigue, see Section K3.

D1. DESIGN TENSILE STRENGTH

The design strength of tension members, $\phi_t P_n$, shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

(a) For yielding in the gross section:

$$\phi_t = 0.90$$

$$P_n = F_y A_g \qquad \text{(D1-1)}$$

(b) For fracture in the net section:

$$\phi_t = 0.75$$

$$P_n = F_u A_e \qquad \text{(D1-2)}$$

where

A_e = effective net area, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
F_u = specified minimum tensile strength, ksi (MPa)

When members without holes are fully connected by welds, the effective net section used in Equation D1-2 shall be as defined in Section B3. When holes are present in a member with welded-end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Equation D1-2.

D2. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5.

The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

EXAMPLE D-1

Given: Determine the design strength of a W8×24 as a tension member in 50 ksi steel. How much dead load can it support?

Solution: If there are no holes in the member, $A_e = A_g$ and Equation 2-3 governs

$$\phi_t P_n = 45.0 \text{ ksi} \times A_g = 45.0 \text{ ksi} \times 7.08 \text{ in.}^2 = 319 \text{ kips}$$

Assuming that dead load is the only load, the governing load combination from Section A is 1.4D. Then, the required tensile strength

$$P_u = 1.4P_D \leq \phi_t P_n = 319 \text{ kips}$$

$P_D \leq$ 319 kips/1.4 = 228 kips maximum dead load that can be supported by the member.

EXAMPLE D-2

Given: Repeat Example D-1 for a W8×24 in 50 ksi steel with four 1-in. diameter holes, two per flange, along the member (i.e., not at its ends) for miscellaneous attachments. See Figure D-1(a).

Solution: a. For yielding in the gross section

$\phi_t P_n$ = 319 kips, as in Example D-1.

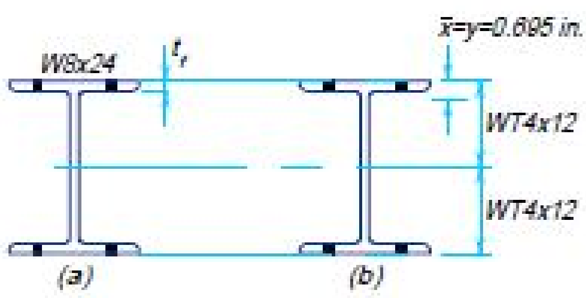
b. For fracture in the net section

$$\begin{aligned}
A_e &= A_n = A_g - 4 \times (d_{hole} + \tfrac{1}{16}\text{-in.}) \times t_f \\
&= 7.08 \text{ in.}^2 - 4 \times (1 + \tfrac{1}{16}\text{-in.}) \times 0.400 \text{ in.} \\
&= 5.38 \text{ in.}^2 \\
\phi_t P_n &= 48.8 \text{ ksi} \times A_e \\
&= 48.8 \text{ ksi} \times 5.38 \text{ in.}^2 = 263 \text{ kips} < 319 \text{ kips}
\end{aligned}$$

Fracture in the net section governs.

P_u = 1.4 $P_D \leq \phi_t P_n$ = 263 kips

$P_D \leq$ 263 kips / 1.4 = 188 kips



Note: If the holes had been at the end connection of the tension member, the U reduction coefficient would apply in the calculation of an effective net area.

EXAMPLE D-3

Given:

Repeat Example D-2 for holes at a bolted end-connection. There are a total of eight 1-in. diameter holes, as shown in Figure D-1(a), on two planes, 4 in. center-to-center.

Solution:

a. For yielding in the gross section $\phi_t P_n = 319$ kips, as in Example D-1.

b. For fracture in the net section, according to Equation B3-1 in Section B above, the effective net area

$$A_e = AU = A_n U$$

where

$$A_n = 5.38 \text{ in.}^2 \text{ as in Example D-2}$$

$$U = 1 - \frac{\bar{x}}{L},\ L = 4 \text{ in.*}$$

According to Commentary Figure C-B3.1(a), $\bar{x}$ for a W8×24 in this case is taken as that for a WT4×12. From the properties of a WT4×12 given in Part 1 of this Manual, $\bar{x} = y = 0.695$ in. See Figure D-1(b).

$$U = 1 - \frac{0.695 \text{ in.}}{4 \text{ in.}} = 0.826$$

Thus

$$A_e = 5.38 \text{ in.}^2 \times 0.826 = 4.45 \text{ in.}^2$$

$$\phi_t P_n = 48.8 \text{ ksi} \times A_e$$

$$= 48.8 \text{ ksi} \times 4.45 \text{ in.}^2 = 217 \text{ kips} < 319 \text{ kips}$$

Fracture in the net section governs. Again, assuming that dead load is the only load,

$$P_u = 1.4P_D \le \phi_t P_n = 217 \text{ kips}$$

$P_D \le 217$ kips / 1.4 = 155 kips maximum dead load that can be supported by the member.

Built-Up Members, Eyebars, and Pin-Connected Members

See Section D2 and D3 in the LRFD Specification.

Compression Member (AISC-LRFD)

CHAPTER E

COLUMNS AND OTHER COMPRESSION MEMBERS

This chapter applies to compact and non-compact prismatic members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, see Section H1.2. For members with slender compression elements, see Appendix B5.3. For tapered members, see Appendix F3.

E1. EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

1. Effective Length

The effective length factor K shall be determined in accordance with Section C2.

2. Design by Plastic Analysis

Design by plastic analysis, as limited in Section A5.1, is permitted if the column slenderness parameter λ_c does not exceed $1.5K$.

E2. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

The design strength for flexural buckling of compression members whose elements have width-thickness ratios less than λ_r from Section B5.1 is $\phi_c P_n$:

$$\phi_c = 0.85$$

$$P_n = A_g F_{cr} \tag{E2-1}$$

(a) For $\lambda_c \leq 1.5$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \tag{E2-2}$$

(b) For $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \tag{E2-3}$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \tag{E2-4}$$

A_g = gross area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
E = modulus of elasticity, ksi (MPa)
K = effective length factor
l = laterally unbraced length of member, in. (mm)
r = governing radius of gyration about the axis of buckling, in. (mm)

Table E-1.
Design Stress for Compression Members of 50 ksi Specified Minimum Yield Stress Steel, $\phi_c = 0.85$*

$\frac{Kl}{r}$	F_{cr} (ksi)	$\frac{Kl}{r}$	F_{cr} (ksi)	$\frac{Kl}{r}$	F_{cr} (ksi)	$\frac{Kl}{r}$	F_{cr} (ksi)	$\frac{Kl}{r}$	F_{cr} (ksi)
1	42.50	41	37.59	81	26.31	121	14.57	161	8.23
2	42.49	42	37.36	82	26.00	122	14.33	162	8.13
3	42.47	43	37.13	83	25.68	123	14.10	163	8.03
4	42.45	44	36.89	84	25.37	124	13.88	164	7.93
5	42.42	45	36.65	85	25.06	125	13.66	165	7.84
6	42.39	46	36.41	86	24.75	126	13.44	166	7.74
7	42.35	47	36.16	87	24.44	127	13.23	167	7.65
8	42.30	48	35.91	88	24.13	128	13.02	168	7.56
9	42.25	49	35.66	89	23.82	129	12.82	169	7.47
10	42.19	50	35.40	90	23.51	130	12.62	170	7.38
11	42.13	51	35.14	91	23.20	131	12.43	171	7.30
12	42.05	52	34.88	92	22.89	132	12.25	172	7.21
13	41.98	53	34.61	93	22.58	133	12.06	173	7.13
14	41.90	54	34.34	94	22.28	134	11.88	174	7.05
15	41.81	55	34.07	95	21.97	135	11.71	175	6.97
16	41.71	56	33.79	96	21.67	136	11.54	176	6.89
17	41.61	57	33.51	97	21.36	137	11.37	177	6.81
18	41.51	58	33.23	98	21.06	138	11.20	178	6.73
19	41.39	59	32.95	99	20.76	139	11.04	179	6.66
20	41.28	60	32.67	100	20.46	140	10.89	180	6.59
21	41.15	61	32.38	101	20.16	141	10.73	181	6.51
22	41.02	62	32.09	102	19.86	142	10.58	182	6.44
23	40.89	63	31.80	103	19.57	143	10.43	183	6.37
24	40.75	64	31.50	104	19.28	144	10.29	184	6.30
25	40.60	65	31.21	105	18.98	145	10.15	185	6.23
26	40.45	66	30.91	106	18.69	146	10.01	186	6.17
27	40.29	67	30.61	107	18.40	147	9.87	187	6.10
28	40.13	68	30.31	108	18.12	148	9.74	188	6.04
29	39.97	69	30.01	109	17.83	149	9.61	189	5.97
30	39.79	70	29.70	110	17.55	150	9.48	190	5.91
31	39.62	71	29.40	111	17.27	151	9.36	191	5.85
32	39.43	72	20.09	112	16.99	152	9.23	192	5.79
33	39.25	73	28.79	113	16.71	153	9.11	193	5.73
34	39.06	74	28.48	114	16.42	154	9.00	194	5.67
35	38.86	75	28.17	115	16.13	155	8.88	195	5.61
36	38.66	76	27.86	116	15.86	156	8.77	196	5.55
37	38.45	77	27.55	117	15.59	157	8.66	197	5.50
38	38.24	78	27.24	118	15.32	158	8.55	198	5.44
39	38.03	79	26.93	119	15.07	159	8.44	199	5.39
40	37.81	80	26.62	120	14.82	160	8.33	200	5.33

* When element width-to-thickness ratio exceeds λ_r, see Appendix B5.3 of LRFD Specification

EXAMPLE E-1

Given: Design a 25-ft high, free standing A618 (F_y = 50 ksi) steel pipe column to support a water tank with a weight of 75 kips at full capacity. See Figure E-1.

Solution: For a live load of 75 kips, the required column strength (from Section A) is $P_u = 1.6P_L = 1.6 \times 75$ kips = 120 kips.

Continuous

From Table C-2, case e, recommended K = 2.1. $KL = 2.1 \times 25.0$ ft = 52.5 ft.

Try a standard 12-in. diameter pipe (A = 14.6 in.^{2}, I = 279 in.^{4}):

$$r = \sqrt{I/A} = \sqrt{279 \text{ in.} / 14.6 \text{ in.}^2} = 4.37 \text{ in.}$$

$$\frac{Kl}{r} = \frac{52.5 \text{ ft} \times 12 \text{ in./ft}}{4.37 \text{ in}} = 144.2$$

From Table E-1, $\phi_c F_{cr}$ = 10.3 ksi

The design compressive strength

$$\phi_c P_n = (\phi_c F_{cr})A_g = 10.3 \text{ ksi} \times 14.6 \text{ in.}^2$$
$$= 150 \text{ kips} > 120 \text{ kips required} \quad \textbf{o.k.}$$

To complete the design, bending due to lateral loads (i.e., wind and earthquake) should also be considered. See Sections F and H.

EXAMPLE E-2

Determine the adequacy of a W14×120 building column.

Given: 50 ksi steel; K = 1.0; story height = 12.0 ft; required strength based on the maximum total factored load is 1,300 kips.

Solution:

$$K_xL_x = K_yL_y = 1.0 \times 12.0 \text{ ft} = 12.0 \text{ ft}$$

Because $r_y < r_x$,

$$\left(\frac{Kl}{r}\right)\text{maximum} = \frac{K_yL_y}{r_y} = \frac{12.0 \text{ ft} \times 12 \text{ in./ft}}{3.74 \text{ in.}} = 38.5$$

From Table E-1, $\phi_c F_{cr}$ = 38.14 ksi

Design compressive strength

$$\phi_c P_n = (\phi_c F_{cr})A_g = 38.14 \text{ ksi} \times 35.3 \text{ in.}^2$$
$$= 1{,}346 \text{ kips} > 1{,}300 \text{ kips required} \quad \textbf{o.k.}$$

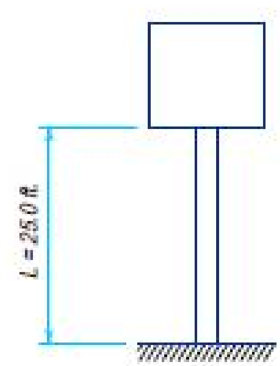


Fig. E-1

EXAMPLE E-3 Select the most economical W14 column for the case shown in Figures E-2 and E-3.

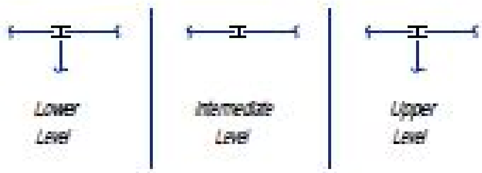


Fig. E-2. Plan views.

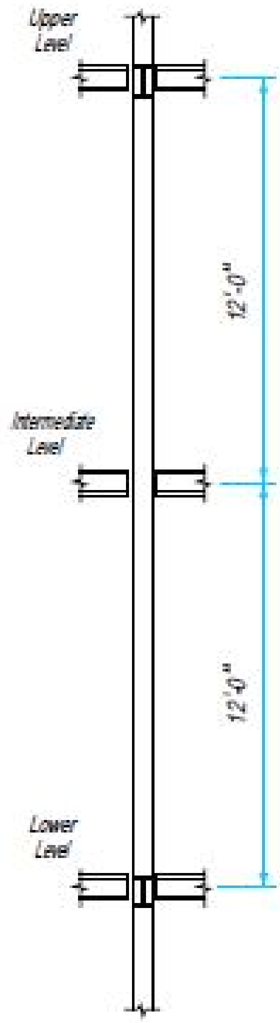


Fig. E-3. Elevation.

Given: 50 ksi steel; $K = 1.0$; required strength based on the maximum total factored load is 1,300 kips. The column is braced in both directions at the upper and lower levels, and in the weak direction at the intermediate level.

Solution: Try a W14×120 (as in Example E-2):

$$\frac{K_x l_x}{r_x} = \frac{1.0 \times 24.0 \text{ ft} \times 12 \text{ in./ft}}{6.24 \text{ in.}} = 46.2$$

$$\frac{K_y l_y}{r_y} = \frac{1.0 \times 12.0 \text{ ft} \times 12 \text{ in./ft}}{3.74 \text{ in.}} = 38.5$$

$$\frac{Kl}{r}\max = \frac{K_x l_x}{r_x} = 46.2$$

From Table E-1, $\phi_c F_{cr} = 36.35$ ksi

$$\text{Required } A_g = \frac{1{,}300 \text{ kips}}{36.35 \text{ ksi}} = 35.8 \text{ in.}^2 > 35.3 \text{ in.}^2 \text{ provided}$$

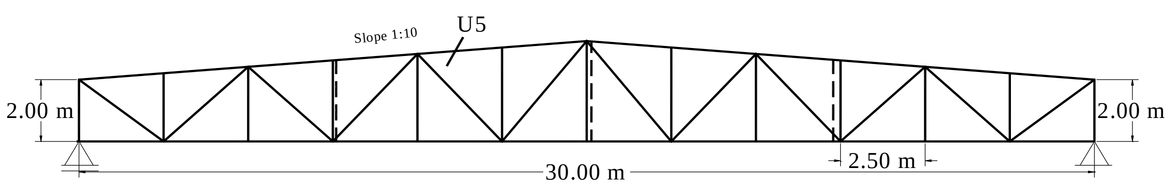
W14×120 **n.g.**

By inspection W14×132 is **o.k.**

Use W14×132

Example E-4

Design of lower chord member U_5 as shown in Figure, where F_y=50ksi, F_{U5}=10 t.



Solution

$L_x=L_y= 2.5/\cos 5.7\sim2.512 \text{ m}/0.3048=8.24 \text{ ft}$

$F_{u5}=10 \text{ t} /0.4536=22.05$ kip

Try 2L 3x 3x 1/2

$\lambda_x=kL_x/r_x=8.24\text{x}12/(0.3\text{x}3) = 109.87$ <200

$\lambda_y=kL_y/r_y=8.24\text{x}12/(0.45\text{x}3) =73.24$ <200 O.k

$\lambda_c=kL(F_y/E)^{0.5}/(r\pi)=8.24\text{x}12\text{x}(50/29000)^{0.5}/(0.3\text{x}3\text{x}3.14)$

$= 1.453 < 1.5$

$F_{cr}=[(0.685)^\wedge\lambda_c^2]F_y=[0.685^\wedge1.453^2]\text{x}50=22.5$ ksi

$\Phi_c F_{cr}=0.85\text{x}22.5=19.13$ ksi

$\Phi_c P_n = \Phi_c F_{cr} A_g = 19.13 \times 2.75 = 52.6$ ksi < $F_{us} = 22.05$ kip O.k waste

Try 2L 2.5x2.5x1/2

Metric Conversion Factors for Common Steel Design Units Used in the LRFD Specification

Unit	Multiply	by	to obtain
length	inch (in.)	25.4	millimeters (mm)
length	foot (ft)	0.3048	meters (m)
mass	pound-mass (lbm)	0.4536	kilogram (kg)
stress	ksi	6.895	megapascals (MPa), N/mm²
moment	kip-in	113,000	N-mm
energy	ft-lbf	1.356	joule (J)
force	kip (1,000 lbf)	4,448	newton (N)
force	psf	47.88	pascal (Pa), N/m²
force	plf	14.59	N/m
temperature	To convert °F to °C: $t_c° = (t_f° - 32)/1.8$		

force in lbf or N = mass × *g*
where *g*, acceleration due to gravity = 32.2 ft/sec² = 9.81 m/sec²

Connections
Welded connections

5.6 DESIGN, STRENGTH AND LIMITATIONS OF FILLET WELDED CONNECTIONS

5.6.1 Nomenclature of The Common Terms

Fig. 5.5 shows the nomenclature of the common terms for fillet welds.

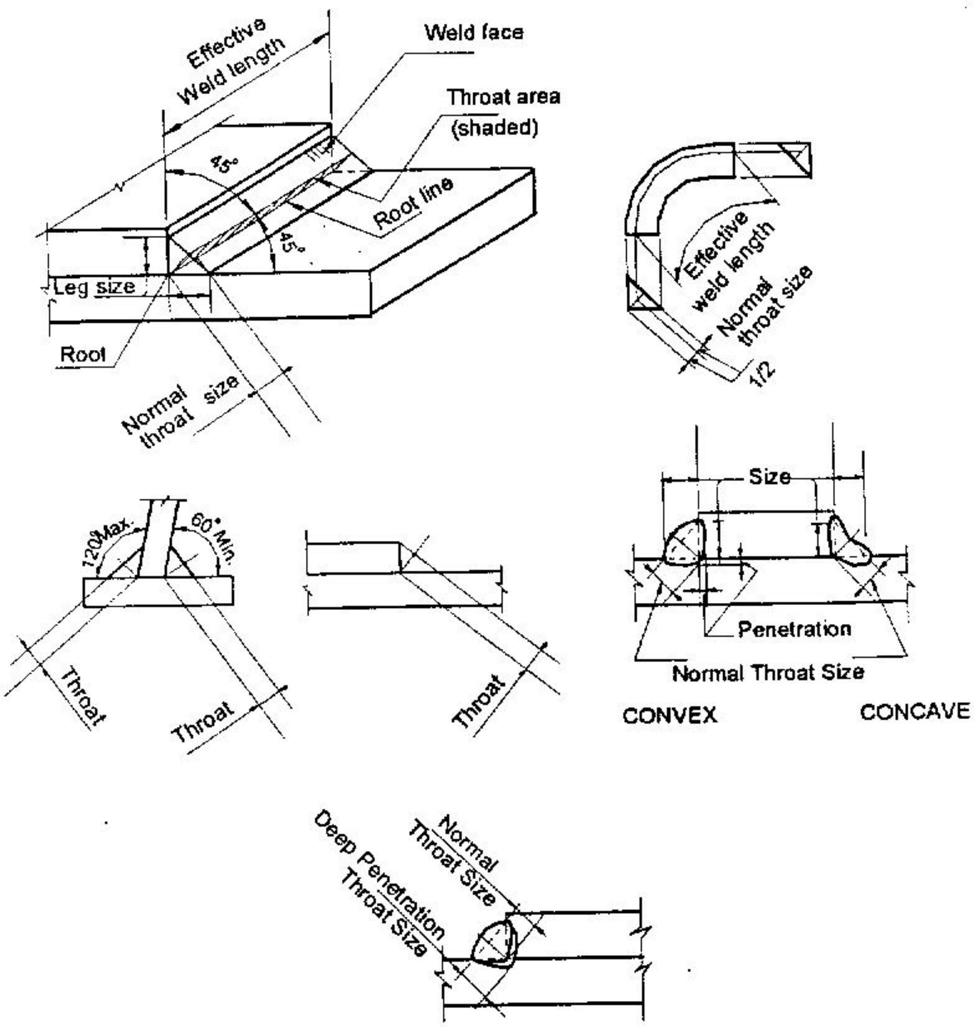


Figure (5.5) Fillet Weld Nomenclature

5.6.2 Different Types of Fillet Welded Connections

Fillet welds are made between plates surfaces which are usually at right angles, but the angle between the plates may vary from 60° to 120°. Tee joints, corner welds and cruciform joints are all combinations of fillet welds and are as shown in Fig. 5.6.

The ideal fillet is normally of the mitre shape which is an isosceles triangle as shown in Fig. 5.7. (h). The mitre and convex welds are stronger than a concave fillet weld of the same leg length when the weld is subject to static loadings, but the concave is stronger when subject to dynamic loadings.

Vertical welds made upwards in one run, are generally convex. Usually low currents produce the convex welds.

The penetration of the weld should reach the root where the contour of penetration is usually as shown in (g) of Fig. 5.7.

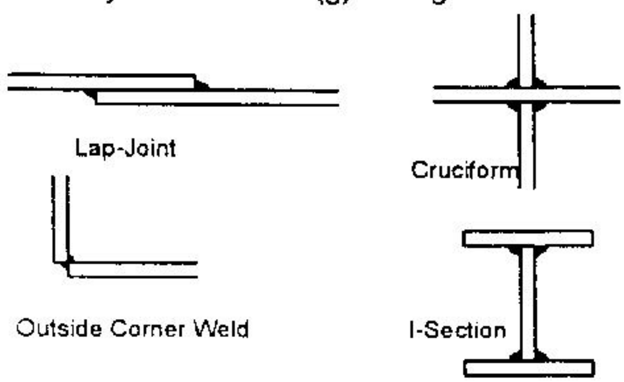


Figure (5.6) Combinations of Fillet Welded Connections

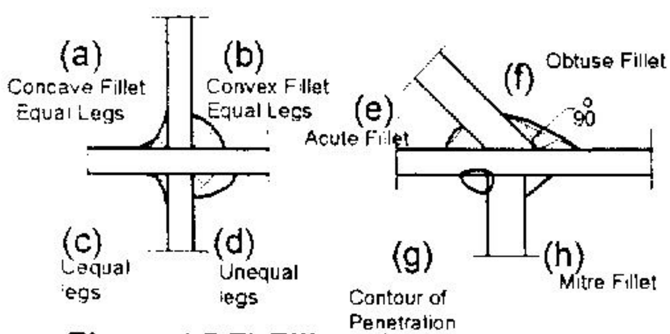


Figure (5.7) Fillet Weld Configurations

5.6.3 Strength of Fillet Welds

5.6.3.1 Effective Area of Fillet Welds

The effective weld section is equal to the largest triangle which can be inscribed between the fusion surfaces and the weld surface, provided there is as a minimum root penetration, this penetration is not taken into account. The effective throat (t) is then the distance from the root to the surface of the isosceles triangular weld along the line bisecting the root angle as shown in Fig. 5.8.

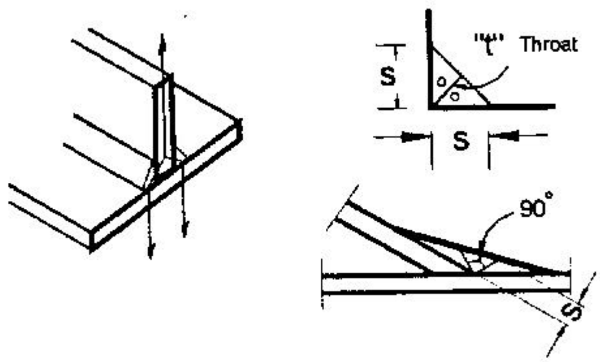


Figure (5.8) Dimensions of Size and Throat of Fillet Weld

Fillet welds are stressed across the throat (t) of the weld, while their size is specified by the leg length (s) where:

$$t = K.s \qquad \text{5.1}$$

The value of "K" depends on the angle between the fusion faces and it may be taken as follows:

Degree	60° – 90°	91° – 100°	101° – 106°	107°– 113°	114°– 120°
K	0.7	0.65	0.6	0.55	0.5

5.6.3.2 Strength and Permissible Stresses

The stress in a fillet weld loaded in an arbitrary direction can be resolved into the following components :

$f_{\perp}$ = the normal stress perpendicular to the axis of the weld.
$q_{//}$ = the shear stress along the axis of the weld.
$q_{\perp}$ = the shear stress perpendicular to the axis of the weld.

These stresses shall be related to the size (s) of the legs of the isosceles triangle inscribed in the weld seam if the angle between the two surfaces to be welded is between 60^{O} and 90^{O}. When this angle is greater than 90^{O} the size of the leg of the inscribed rectangular isosceles triangle shall be taken.

The permissible stresses F_{pw} for all kinds of stress for fillet welds must not exceed the following:

$$\text{All kind of stresses } F_{pw} \leq 0.2\, F_u \quad \text{.........................} \qquad \mathbf{5.2}$$

Where F_u is the ultimate strength of the base metal (see table 5.1).

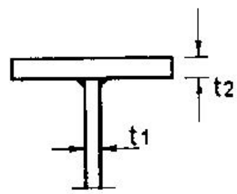
In case where welds are simultaneously subject to normal and shear stresses, they shall be checked for the corresponding principal stresses. For this combination of stresses, an effective stress value **f_{eff}** may be utilized and the corresponding permissible weld stress is to be increased by 10 % as follows:

$$\mathbf{f_{eff}} = \sqrt{f_{\perp}^{2} + 3(q_{\perp}^{2} + q_{//}^{2})} \leq 1.1\, F_{pw} \quad \text{...............} \qquad \mathbf{5.3}$$

The effective length of a fillet weld is usually taken as the overall length of the weld minus twice the weld size (s) as deduction for end craters.

iii- It is recommended that the following limitations in sizes of fillet welds as related to the thickness of the thicker part to be joined should be observed as shown in Fig. 5.10.

t (max. of t_1 or t_2) (mm)	Size s (mm)
≤ 10	≥ 4
10-20	≥ 5
20-30	≥ 6
30-50	≥ 8
50-100	≥ 10



Figure(5.10) Thickness of Plates to be Welded

iii- The minimum size of fillet welds is 4 mm for buildings and 6 mm for bridges.

c- Fillet Weld Length

i- The effective length for load transmission should not be less than 4 times the weld size (s) or 5 cm whichever is largest.

ii- The maximum effective length of fillet welds should not exceed 70 times the size. Generally in lap joints longer than 70 s a reduction factor β allowing for the effects of non- uniform distribution of stress along its length is to be utilized where:

$$\beta = 1.2 - 0.2\, L/(70\, s) \qquad 5.4$$

Where
L = overall length of the fillet weld.
$\beta \leq 1$

BOLTED CONNECTIONS

6.1 MATERIAL PROPERTIES

6.1.1 Non- Pretensioned Carbon and Alloy Steel Bolts

For non – pretensioned bolts, where the forces acting transverse to the shank of the bolt are transmitted either by shear or bearing, the nominal values of the yield stress F_{yb} and the ultimate tensile strength F_{ub} are as given in Table 6.1 :-

Table (6.1) Nominal Values of Yield Stress F_{yb} and Ultimate Tensile Strength F_{ub} for Bolts

Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
F_{yb} (t/cm^2)	2.4	3.2	3.0	4.0	4.8	6.4	9.0
F_{ub} (t/cm^2)	4.0	4.0	5.0	5.0	6.0	8.0	10.0

6.4.1 Shear Strength R_{sh}

i- The allowable shear stress q_b for bolt grades 4.6, 5.6 and 8.8 shall be taken as follows:

$$q_b = 0.25\ F_{ub} \quad \text{..} \quad \mathbf{6.1}$$

ii- For bolt grades 4.8, 5.8 , 6.8, and 10.9, the allowable shear stress q_b is reduced to the following:-

$$q_b = 0.2\, F_{ub} \quad \ldots\ldots\ldots\ldots \quad 6.2$$

iii- For the determination of the design shear strength per bolt (R_{sh}) , where the shear plane passes through the threaded portion of the bolt:-

$$R_{sh} = q_b \,.\, A_s \,.\, n \quad \ldots\ldots\ldots\ldots \quad 6.3$$

Where :

A_s = The tensile stress area of bolt.

n = Number of shear planes.

6.4.2 Bearing Strength R_b

i- The bearing strength of a single bolt shall be the effective bearing area of bolt times the allowable bearing stress at bolt holes:-

$$R_b = F_b \,.d.\, \min \sum t \quad \ldots\ldots\ldots\ldots \quad 6.4$$

Where :

F_b = Allowable bearing stress.

d = Shank diameter of bolt.

$\min \sum t$ = Smallest sum of plate thicknesses in the same direction of the bearing pressure.

ii- For distance center- to center of bolts not less than 3d, and for end distance in the line of force greater than or equal to 1.5 d, the allowable bearing stress F_b (t/cm^2):

$$F_b = \alpha F_u \quad \text{...} \quad \mathbf{6.5}$$

Where :

F_u = The ultimate tensile strength of the connected plates.

As the limitation of deformation is the relevant criteria the α-values of Equation 6.5 are given in Table 6.2.

Table (6.2) Values of α for Different Values of End Distance

	End distance in direction of force			
	≥ 3d	≥ 2.5d	≥ 2.0d	≥ 1.5d
α	1.2	1.0	0.8	0.6

6.5.2 Design Principles of High Strength Pretensioned Bolts

6.5.2.1 The Pretension Force

The axial pretension force T produced in the bolt shank by tightening the nut or the bolt head is given by:-

$$T = (0.7)\, F_{yb}\, A_S \quad \text{...} \quad \mathbf{6.9}$$

Where :

F_{yb} = Yield (proof) stress of the bolt material, (Table 6.1).

A_S = The bolt stress area.

6.5.2.2 The Friction Coefficient or The Slip Factor " μ"

i- The friction coefficient between surfaces in contact is that dimensionless value by which the pretension force in the bolt shank is to be multiplied in order to obtain the frictional resistance P_S in the direction of the applied force.

ii- The design value of the friction coefficient depends on the condition and the preparation of the surfaces to be in contact. Surface treatments are classified into three classes, where the coefficient of friction **μ** should be taken as follows:-

μ = 0.5 for class A surfaces.
μ = 0.4 for class B surfaces.
μ = 0.3 for class C surfaces.

iii- The friction coefficient μ of the different classes is based on the following treatments:

In class A:

- Surfaces are blasted with shot or grit with any loose rust removed, no painting.
- Surfaces are blasted with shot or grit and spray metallized with Aluminium.
- Surfaces are blasted with shot or grit and spray metallized with a Zinc based coating.

In class B:

- Surfaces are blasted with shot or grit and painted with an alkali-zinc silicate painting to produce a coating thickness of 50-80 μm.

In class C:

- Surfaces are cleaned by wire brushing, or flame cleaning, with any loose rust removed.

6.5.2.3. The Safe Frictional Load (P_s)

The design frictional strength for a single bolt of either grade 8.8 or 10.9 with a single friction plane is derived by multiplying the bolt shank pretension **T** by the friction coefficient **μ** using an appropriate safety factor γ as follows:-

$$P_s = \mu T / \gamma \qquad \textbf{6.10}$$

Where :

T	=	Axial pretensioning force in the bolt.
μ	=	Friction coefficient.
γ	=	Safety factor with regard to slip .
	=	**1.25** and **1.05** for cases of loading **I** and **II** respectively for ordinary steel work.
	=	**1.6** and **1.35** for case of loading **I** and **II** respectively for parts of bridges, cranes and crane girders which are subjected mainly to dynamic loads.

Table 6.3 gives the pretension force (T) and the permissible frictional load (P_s) per one friction surface for bolts of grade 10.9.

Table (6.3) Properties and Strength of High Strength Bolts (Grade 10.9*)

Bolt Diameter (d) mm	Bolt Area (A) cm^2	Stress Area (A_s) cm^2	Pretension Force (T) tons	Required Torque (M_a) kg.m	Permissible Friction Load of One Bolt Per One Friction Surface (P_s) tons							
					Ordinary Steel Work				Bridges and Cranes			
					St. 37&42-44 (μ=0.4)		St. 50-55 (μ=0.5)		St. 37&42-44 (μ=0.4)		St. 50-55 (μ=0.5)	
					Cases of Loading				Cases of Loading			
					I	II	I	II	I	II	I	II
M12	1.13	0.84	5.29	12	1.69	2.01	2.11	2.52	1.32	1.56	1.65	1.95
M16	2.01	1.57	9.89	31	3.16	3.37	3.95	4.71	2.47	2.92	3.09	3.66
M20	3.14	2.45	15.43	62	4.93	5.90	6.17	7.36	3.85	4.56	4.82	5.71
M22	3.80	3.03	19.08	84	6.10	7.27	7.63	9.10	4.77	5.65	5.96	7.06
M24	4.52	3.53	22.23	107	7.11	8.45	8.89	10.60	5.55	6.58	6.94	8.22
M27	5.73	4.59	28.91	157	9.25	11.03	11.56	13.78	7.22	8.55	9.03	10.70
M30	7.06	5.61	35.34	213	11.30	13.48	14.13	16.86	8.83	10.46	11.04	13.07
M36	10.18	8.17	51.47	372	16.47	19.64	20.58	24.55	12.86	15.24	16.08	19.05

$T = (0.7)\ F_{yb}.A_s$ $\quad M_a = 0.2\ d.T.$ $\quad P_s = \mu\ T / \gamma$

* For HSB grade 8.8 , the above values shall be reduced by 30%

Higher Technological Institute — ***CT 122 Metallic Structure (I)***
Department of Civil Engineering — ***Final Examination***

***Time 90 min*.**

***Specify any missing information and state your assumptions clearly*.**

Problem I

A shed frame-truss with spacing equals 4.50 m as shown in figure (1) covers an area 7.50 mx45m. ***It is required to***

1- Draw a general layout. (10 marks)

2- Design of member **A, G** and check the safety of member **B,** if its cross-section is **2L 70x70x7. (15 marks)**

3- Design connection of Joint 1 if members **C**, **D** are **2L 60x60x6** as a welded connection and members **E**, **F** are **L80x80x8** as a bolted connection. (15 marks)

Given: (ECP), Steel 37, F_y=2.4 t/cm^2, bolts M12 quality (8.8).

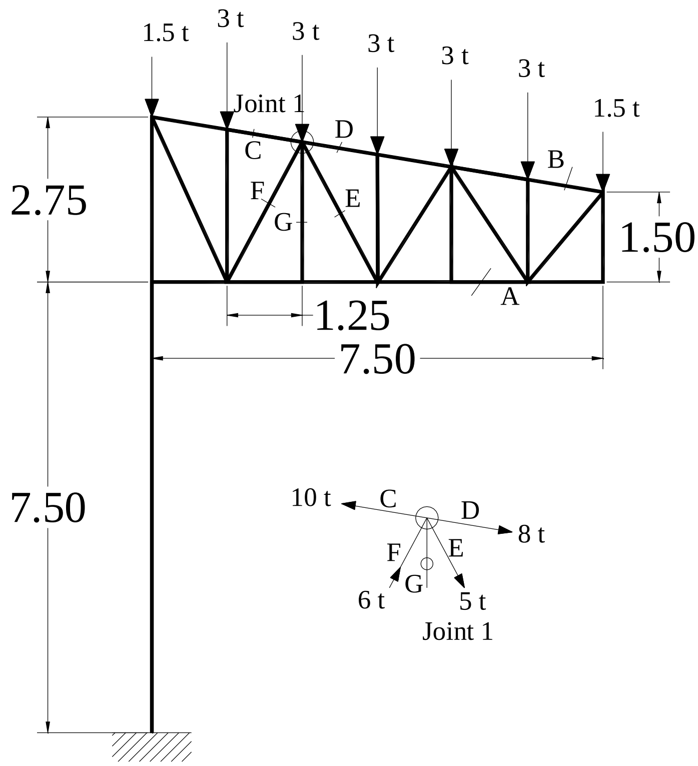


Figure (1) Shed Frame-Truss

"With my best wishes"
Dr. Essam Amoush

Design of Joint 1

For member C

R_{sh}=0.25x8x0.84x2=3.36 t

R_b= 0.6x3.6x1.2x1.0=2.592 t

$R_{bearing}$=min form {Rsh & Rb} =2.592 t

N. o. bolts = 10/2.592=3.86 bolts

Take 4 bolt / line

For Member D

N. o. bolts=8/2.592=3.09 bolts

Take 4 bolts / line

For member E & F

F_{pw}=0.2x3.6=0.72 t/cm2

Assume S_{min}=4mm

S_{max}=8mm

Take S=6 mm

F_1= 6x5.74/8=4.305 t

L_1xS= 4.305/0.72=5.98 cm2

L_1=5.98/0.6=9.97 cm

$L_{1\ eff.}$=9.97+2S=9.97+2x0.6=11.17 cm ~112 mm

F_2= 6x2.26/8=1.7 t

L_2xS=1.7/0.72=2.36 cm2

L_2=2.36/0.6=3.93 cm

$L_{2\ eff.}$= 3.93+2x0.6=5.13 cm ~52 mm