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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I - PLATE GIRDER - SCI1309

UNIT I
PLATE GIRDER

Plate girders are typically used as long-span floor girders in buildings, as bridge girders, and as crane girders in industrial structures.

Commonly term girder refers to a flexural cross section made up of a number of elements. They are generally considerably deeper than the deepest rolled sections and usually have webs thinner than rolled sections. Plate girders are at their most impressive in modern bridge construction where main spans of well over 200m are feasible, with corresponding cross- section depths, haunched over the supports, in the range of 5-10m.

Need:

1. Large Spans (above 20m)
2. Heavy Loads
3. Road or Rail Bridges
4. When rolled I-Sections are not available (i.e., above 500mm depth)

Elements of Plate Girder: (Welded)

1. Web
2. Flanges
3. Stiffeners (to avoid Web Buckling & Web Crippling Failure)

a. Transverse Stiffener (Vertical)

i. End Bearing Stiffener

ii. Intermediate Stiffener

b. Longitudinal Stiffener (Horizontal)

i. 1st stiffener at 0.20d from top

ii. 2nd stiffener at 0.50 d from top

Selection of Stiffeners:

<table>
<tr><th>Case</th><th>$k = d/t_w$</th><th>End Bearing Stiffener</th><th>Intermediate Stiffener</th><th>Longitudinal Stiffer</th><th>b/t_f</th></tr>
<tr><td>I</td><td><67</td><td>NO</td><td>NO</td><td>NO</td><td>8.4</td></tr>
<tr><td>II</td><td>100-110</td><td>YES</td><td>NO</td><td>NO</td><td rowspan="3">13.6</td></tr>
<tr><td>III</td><td>200</td><td>YES</td><td>YES</td><td>NO</td></tr>
<tr><td>IV</td><td>>250</td><td>YES</td><td>YES</td><td>YES</td></tr>
</table>

Case I: Design is Simple (similar to beam). But, uneconomical
Case II: Economical.

Example: For k=67,

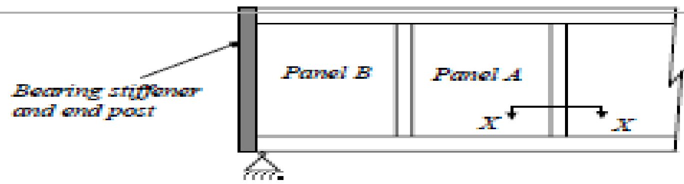
depth= 67 x tw and b= 8.4 x tf

Stiffeners are provided to transfer transverse concentrated compressive force on the flange into the web and are essential for desired performance of web panels. These are referred to as bearing stiffeners. **Intermediate web stiffeners** are provided to improve its shear capacity. Design of these stiffeners is discussed below.

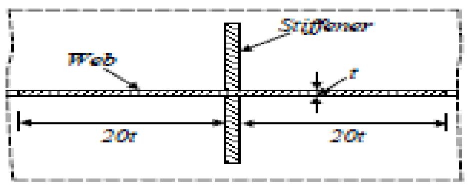
Load bearing stiffeners Whenever there is a risk of the buckling resistance of the web being exceeded, especially owing to concentrated loads, load-bearing stiffeners are provided. Normally a web width of 20 t on both sides as shown in Fig. is assumed to act along with the stiffener provided to resist the compression as an equivalent cruciform shaped strut of effective length 0.7 times its actual length between the top and bottom flanges. The bearing stress in the stiffener is checked using the area of that portion of the stiffener in contact with the flange through which compressive force is transmitted.

Intermediate stiffeners The intermediate stiffeners are provided to prevent out of plane buckling of web at the location of stiffeners. The buckling resistance Pq of the stiffener acting as a strut (with a cruciform section as described earlier) should be not less than (Vt -Vs) where Vt is the maximum shear force in the panel and Vs is the buckling resistance of web without considering tension field action. In its limit Vs will be equal to Vcv of the web without stiffeners. Sometimes the stiffeners are provided for more than one of the above purposes. In such cases stiffeners are considered for their satisfactory resistance under combined load effects. Such combined loads are common.

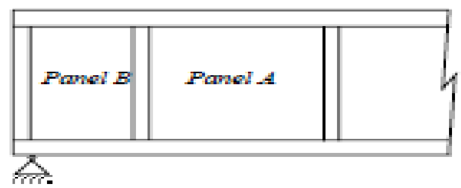
Longitudinal stiffeners Longitudinal stiffeners are hardly used in building plate girders, but sometimes they are used in highway bridge girders for aesthetic reasons. They are not as effective as transverse stiffeners. Nowadays, the use of longitudinal stiffeners is rare due to welding problems. For design of longitudinal stiffeners there are two requirements: • A moment of inertia to ensure adequate stiffness to create a nodal line along the stiffener • An area adequate to carry axial compression stress while acting integrally with the web.



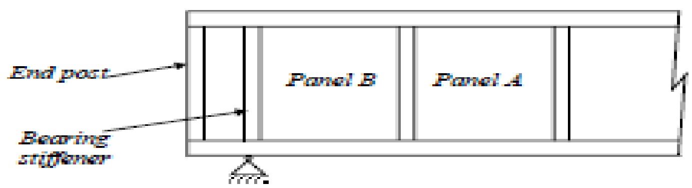
(a) End panel designed using tension field action and end post designed for both bearing and to resist tension field



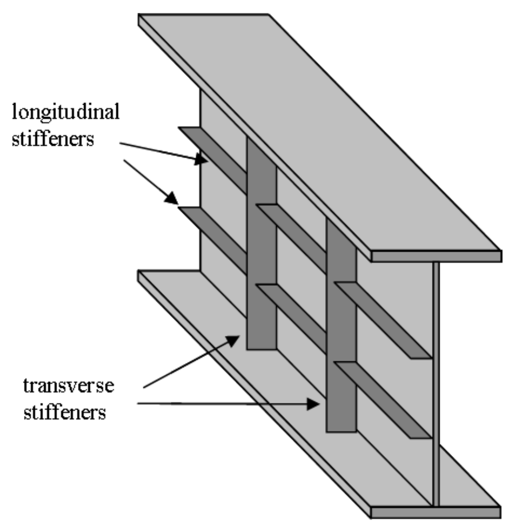
(b) Section at X-X



(c) End panel designed without using tension field action



(d) End panel designed using tension field strengthened by additional stiffener (Double stiffener)



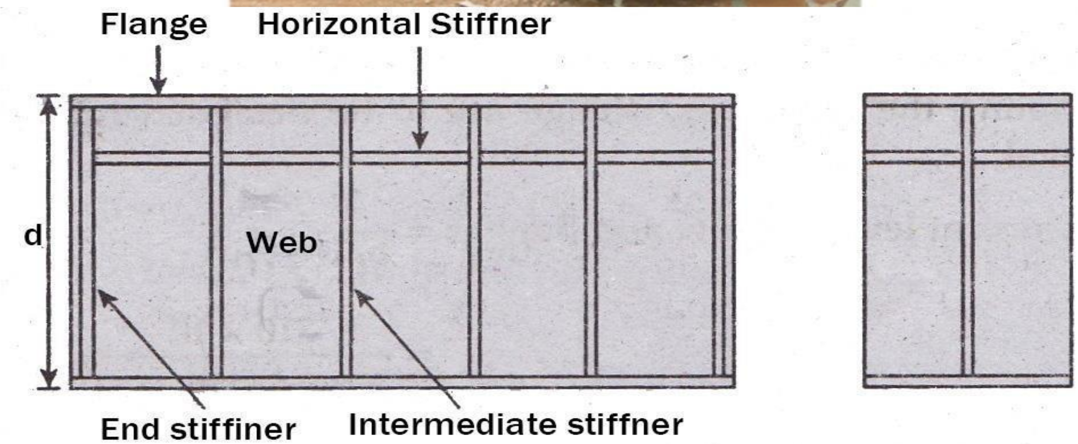
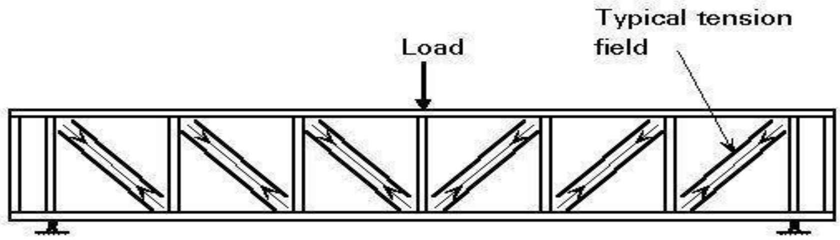


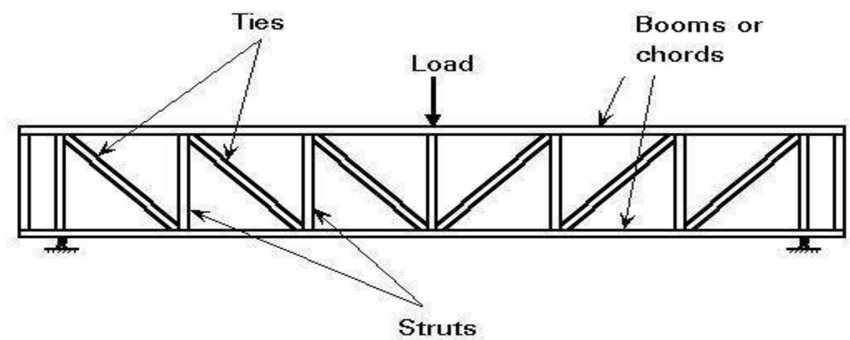
Fig : Plate Girder

Tension field method

Design of plate girders with intermediate stiffeners, as indicated in Fig., can be done by limiting their shear capacity to shear buckling strength. However, this approach is uneconomical, as it does not account for the mobilisation of the additional shear capacity as indicated earlier. The shear resistance is improved in the following ways: i. Increasing in buckling resistance due to reduced c/d ratio; ii. The web develops tension field action and this resists considerably larger stress than the elastic critical strength of web in shear Figure 6.13 shows the diagonal tension fields anchored between top and bottom flanges and against transverse stiffeners on either side of the panel with the stiffeners acting as struts and the tension field acting as ties. The plate girder behaves similar to an N-truss. The nominal shear strength for webs with intermediate stiffenerscan be calculated by this method according to the design provision given in code.



(a) Tension field action in individual sub panels of a girder with transverse stiffeners



(b) Typical N-truss for comparison

Design of Plate Girder:

Step 1: Assume Self Weight of Beam @ WL/2000

Where,

W = Superimposed Load on

beam L = Effective Span

Step 2: Calculate Bending Moment & Shear Force

Step 3: Find economical depth,

d d = $[M. k / fy]^{1/3}$

Where, k = d/tw = 67 or 100 or 200 or 250 (based on case 1, 2, 3, 4)

tw = k.d

Select suitable Web Plate

Step 4: Select suitable Flange

Equate M = C x Z or T x Z and find Af

Where,

C = Compressive Force = Area of Flange x Design Stress

= Af x (fy/mo)

Z = Lever Arm = C/c of two flanges

Step 5: Equate Af = b. tf and find tf and b

Where,

tf = thickness of flange plate

b = 8.4 tf for Case I

b = 13.6 tf for Case II, III, IV

Step 6: Check for moment carrying capacity of

beam Md > Mu

Md = Z. (fy/γ_{mo})

Where,

Z = Ixx / Ymax

Ixx = 2 $[b.tf^3/12 + (b.\ tf)\ Y^2]$ (Note: Ixx of web is

negligible) Ymax = Neutral Axis to Top of the flange

Step 7: Check for Shear Resistance (clause 8.4 of IS 800:2007)

Vd > Vu

Step 8: Check for web crippling (at

supports) Fw > Vu

Fw = $[(b1+n2)\ tw].[(fyw/\ \gamma_{mo})]$

Flange splices A joint in the flange element provided to increase the length of flange plate is known as flange splice. The flange splices should be avoided as far as possible. Generally, the flange plates can be obtained for full length of the plate girder. In spite of the availability of full length of flange plates, sometimes it becomes necessary to make flange splices. Flange joints should not be located at the points of maximum bending moment.

Connection between web and flange of Plate girder

If 'V' is the shear force acting on the section, then shear stress at the junction is,

$$q_w = \frac{V}{b\,I_z}(a\,\bar{y})$$

$\therefore$ Shear force per unit length $= \dfrac{V}{I_z}(a\,\bar{y})$

If weld of throat thickness 't' is provided on both side, then strength of slop weld per unit length

$$= 2t\,\frac{f_w}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating the force to strength we get

$$\frac{V}{I_z}(a\,\bar{y}) = 2t\,\frac{f_w}{\sqrt{3}} \times \frac{1}{1.25}$$

Hence throat thickness of weld 't' can be found, from which size of weld is obtained as $s = \dfrac{t}{0.7}$.

In finding shear stress, moment of inertia of flange alone may be considered i.e.

$$I_z = \frac{b_f\,d^3}{12}$$

If weld size comes out too small intermittant welding may be adopted.

End Panel Design

In the end panel, The nominal shear strength, *Vn,* of webs with or without intermediate stiffeners s governed by buckling may be evaluated using one of the following methods given under cl.8.4.2.2 of IS 800 : 2007,

a) Simple post-critical method — The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by: *Vn= Vcr*

b) Tension field method — The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and if c/d >= 1.0, where c,*d* are the spacing of transverse stiffeners and depth of the web, respectively.

In the tension field method, the nominal shear resistance, *Vn,is* given by: *Vn = Vtf*

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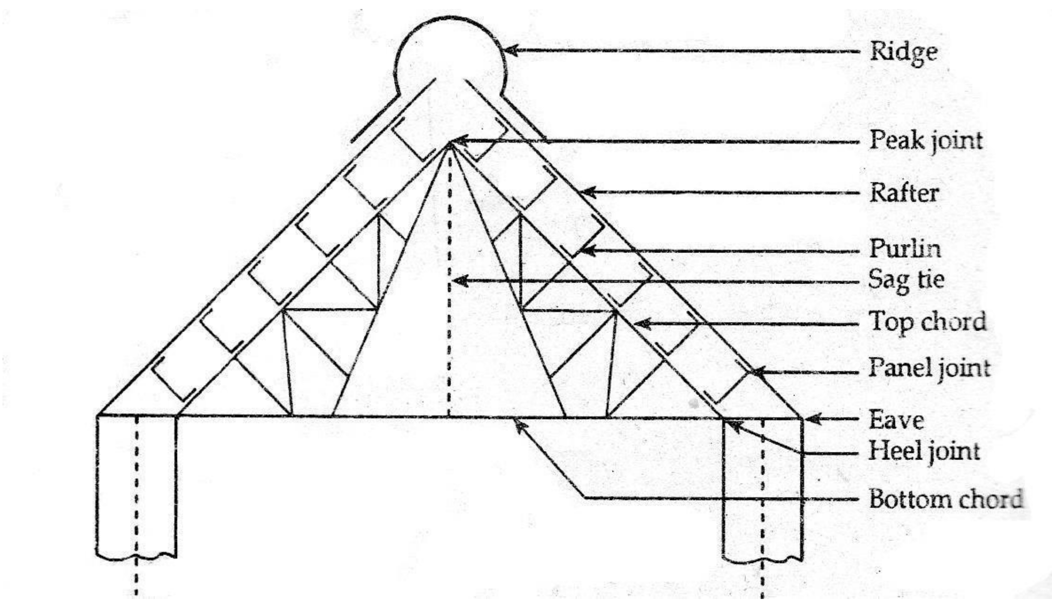
UNIT –I I - ROOF TRUSSES AND INDUSTRIAL STRUCTURES - SCI1309

UNIT II
ROOF TRUSSES AND INDUSTRIAL STRUCTURES

Roof Truss

When sloping roofs have to be provided, roof trusses become necessary. At places of heavy rainfall or heavy snowfall, sloping roofs are necessary which have to be supported by roof trusses. Workshops warehouses, industrial buildings etc. also need sloping roofs and hence roof trusses. For many single storey buildings sloping roofs on trusses are common. When a roof is to be provided for a building which does not have interior supports and the exterior walls are more than12 m apart, a roof truss will be a convenient arrangement to support the roof.

Components of a Steel Roof Truss



A roof truss consists essentially of the following components;

1. *Upper chord members*

2. *Bottom chord members*

3. *Web members*

The uppermost line of members which extend from one support to the other through the apex is called the upper chord, whereas the bottom chord consists of the lowermost line of members extending from one support to other. In trusses simply supported at the ends, the members in the top chord are subjected to compression and the members of the bottom chord are subjected to tension. But in cantilever trusses, the top chord members will be in tension and the bottom chord members will be in compression. Usually in simply supported trusses, for the normal loadings, the top and bottom chord members near the support carry greater forces.

The top and the bottom chord members are connected by vertical or diagonal members called web members. The joint at the support is called the heel joint while the joint at the ridge is called the peak joint.

Struts

The members which do not belong to top or bottom chord and subjected to compressive forces are called struts.

Ties

The members which do not belong to top/ bottom chord but are mainly subjected to the tension are called ties.

Span

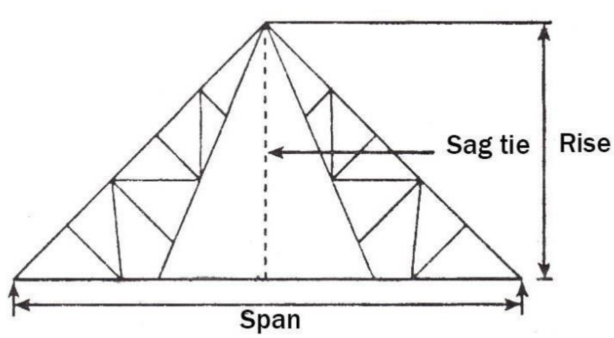
The distance between the supporting end joints of a truss is called its span. When supported on walls, the distance between the centres of bearings is considered as the span. When framed into columns the distance between the column faces is regarded as span.

Rise

The rise of a truss is vertical distance between apex and line joining the supports.

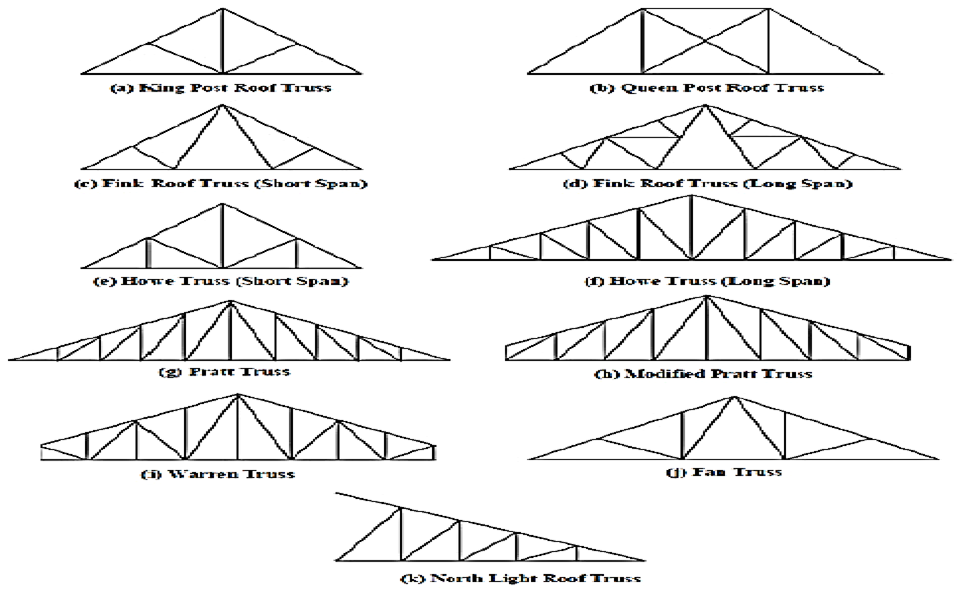
Pitch

The ratio of the rise to the span is called the pitch. A minimum pitch of 1/6 is to be maintained for G.I. sheet covering and expected and 1/12 is to be maintained for AC sheet covering.The preferable pitch is 1/4 if snow load is expected and 1/6 if snow load is not expected.

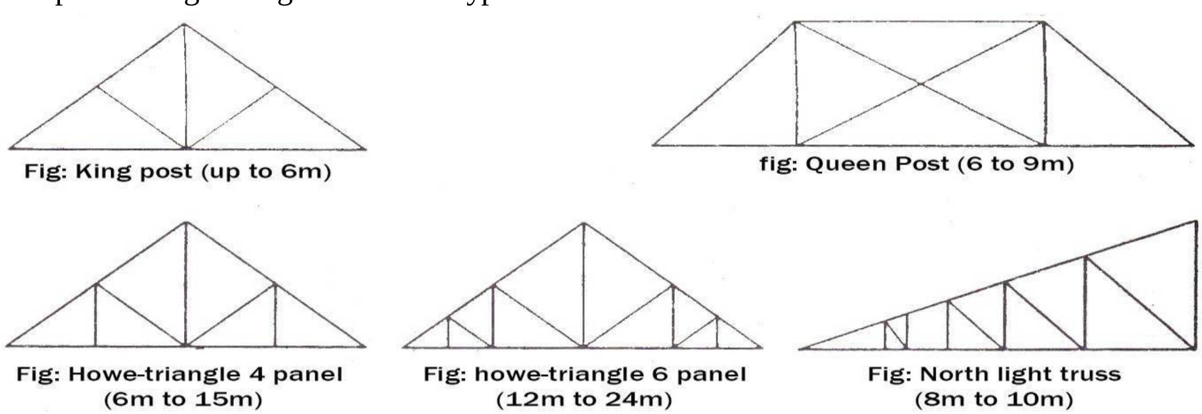


Pitch = Rise/Span

Types of Roof Trusses



The triangle is the primary pin joined frame which has a stable figure. Hence, all trusses should comprise triangular figures various types of trusses are used shown below.



The king post truss is mainly adopted for short spans (up to 6 m). It is usually built of wood completely or of wood combined with steel. Steel rods are used as tension members.

The queen post truss is found suitable for spans 6 m to 9 in. For ordinary buildings, the link type truss is found to be very satisfactory. These trusses are convenient for spans 12 m to 18 m.

For small spans, flat roofs may be supported on beams. But for larger spans, flat trusses are to be used. In this case, the upper chord will be inclined sufficiently to provide just the required slope for proper drainage.

In factory buildings where considerably more light is desirable, the saw tooth truss is used. In this type, the sleep sides of the trusses will be glazed. The glazed panels are usually faced towards North to avoid the direct glare of the sun and are hence called North light roof trusses. For long spans and where more head room has required the crescent, the truss is adopted. For such conditions the scissors truss, the curb truss, the shed truss, the three hinged arched truss, the Hammer beam truss are also used.

Purlins

As far as possible, purlins should be located on panel points off top chord members. However, it depends on the type of roofing materials also. Generally, the spacing of purlins varies from 1.35 m to 2.0 m.

Spacing of roof truss	Type of Steel Section
Small (3 m to 4 m)	Angles
Medium (4 m to 5.0 m	Channels
Large (> 5.0 m)	I-Section

If angles are used, outstanding legs are at the top and lug angles are used to connect the purlins to rafters.

Sheetings

Commonly used sheeting is G.I. and A.C.

G.I. Sheets

Corrugated iron sheets are galvanized for protection against corrosion and are used as roof coverings and side coverings

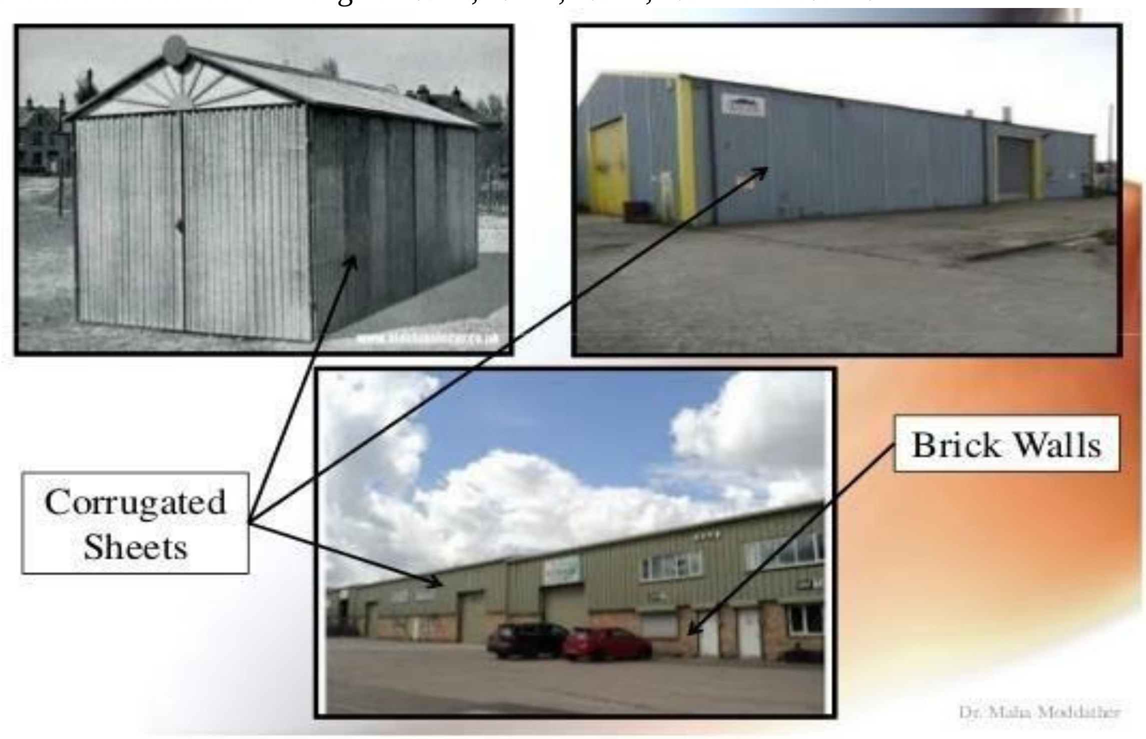
The common sizes of G.I. sheets are;

1. 8 corrugations, 75 mm wide and 19 mm deep which have an overall width of 660 mm.
2. 10 corrugations, 75 mm wide and 19 mm deep, which have an overall width of 810 mm.

The sheets are available in the gauges 16, 18, 20, 22 and 25

Note: thickness = 25/ gauge mm

The sheets area available in lengths 1.8 m, 2.2 m, 2.5 m, 2.8 m and 3.0 m.



The sheets should be used with the following overlaps.

Side laps: 1, 3/2 or 2 corrugations.

End laps: 100 mm, if a slope is more and 150 m, if a slope is less than 20°.

For lesser overlaps, suitable sealings should be made. The sheets should be fastened to purlins or sheeting rails by 8 mm diameter hook bolts at a maximum pitch of 350 mm.

The spacing of purlins depends on the applied loading, thickness of sheets and length of sheets. For common loading, the thickness of sheetings is so fixed that, with required overlaps the sheetings can be used fully.

A.C. Sheets

Asbestos cement sheets are better insulators for Sun's heat compared to GI sheet. They are used commonly in the factories and godowns. They are available in two common shapes i.e, corrugated and Trafford. They are available in the lengths of 1.75, 2.0, 2.5 and 3.0 m. They are available in thicknesses of 6 mm and 7 mm. The maximum permissible spacing is 1.4 m for 6 m sheets and 1.6 m for 7 mm sheets. They are to be used with a longitudinal overlap of 150 mm and a side overlap of one corrugation spacing of purlins is to be adjusted such that as far as possible the cutting of sheets is avoided

Loads on Roof Truss

The main loads on roof trusses are;

i) Dead Loads

ii) Imposed Loads

iii) Wind Loads

iv) Other Loads

Dead Loads

It includes the weight of sheetings, purlins, bracings, self-weight and other loads suspended from trusses.

The unit weight of various materials are given in is 875-part 1. The following values may be noted;

GI Sheets: 85 N / m^2

AC Sheets: 130 N / m^2

In general, the roof covering weight including laps, connector etc. may be taken as,

i) 100-150 N / m^2 for G.I. sheeting

ii) 170-200 N / m^2 for A.C. sheeting

The weight of purlins works out 100-120 N / $m^2$of the plan area.

On lower panel points additional occasional load may be considered. Such load is due to electrical fixtures, fans etc. This may be assumed to be 5 to 10 KN, distributed over lower panel points. If the false ceiling is to be suspended that load should be estimated separately.

There are various formulae suggested in the literature to assume the self-weight of truss. However, since it is a small percentage of total load anyone of them may be used. The following are the two formulae commonly used for a truss of span 'l'.

a) $w = 20 + 6.6\,L\ \mathrm{N/m^2}$ for a live load of $2\ \mathrm{kN/m^2}$

If a live load is more, the above value is to be increased by LL / 2.0.

b) $w = 10\left(\frac{L}{3} + 5\right)\frac{S}{4}\ \mathrm{N/m^2}$

where, S is the spacing of

trusses.

Imposed Load (Live load)

Normally, no access is provided for sloping roofs with a sheet. In such cases IS 875 part II makes the following provisions for live loads for the design of sheets and purlins.

Up to 10° slope = 0.75 KN / m^2

For more than 10° slope = 0.75 - 0.02 (θ-10) , where, θ is slope of sheeting. However, a minimum of 0.4 kN / m^2 live load should be considered in any case.

For the design of trusses, the above live load may be reduced to 2/3rd of live load.

The purlins and sheets should be checked to support a concentrated load of 0.9 KN at the worst position.

Wind Load

Basic Wind Pressure, pz = 0.60
Vz 2
Where Vz = K1. K2. K3. Vb
K1 = Risk Coefficent (Table 4 of IS 875)
K2 = Coefficent depends on Terrain, Height and Structure size factor (Table 5 of IS 875)
K3 = Topographic Factor = 1
Vb = Basic Wind Speed (IS 875 part 3) based on 6 zones of india (between 33 m/s to 55 m/s)

Load combinations

1. 1.5(DL+LL)
2. 1.2(DL+LL+WL)
3. 0.9DL + 1.5 WL

Design of Elements of Roof truss

Any element is designed as both tension and compression member.

For tension members, the steps are

i) Determine the gross section area of an element based on tensile force
ii) Select a suitable section from steel table
iii) Check whether the tensile strength of the section – minimum of strength governed by yielding, rupture and block shear is > tensile force
iv) No. of bolts at each end for connection = Factored tensile force / strength of a bolt.

For Compression members

i) Effective length of a member = 0.85 L
ii) Calculate the Slenderness ratio = KL/r for the same section selected for tension member
iii) Identify the buckling class
iv) Determine the design compressive stress, fcd
v) Check Compressive strength = fcd x c/s area of section > factored compressive force.

Design of Purlins

i) Critical load based on load combinations
ii) Vertical and horizontal components of Total load acting on purlin, Wy and Wz
iii) Bending moment about Y and Z axis, My and Mz
iv) Plastic section modulus required = Mz / fy
v) Selection of a channel / I section from steel tables
vi) Plastic section moduli Zpz and Zpy
vii) Plastic moment capacity about both axes, Mdz and Mdy
viii) Check for moment capacity
$$Mz / Mdz + My / Mdy < 1$$
ix) Check for deflection
$$\delta = 5\ Wz\ L^4 / (384\ E\ Iz) < Span / 180$$

End Bearings

When the span of truss is > 15 m, there will be considerable amount of contraction or expansion due to temperature changes.

If both the ends are fixed in position, the expansion/ contraction of truss will induce compressive / tensile temperature stresses in the members. This leads to buckling failure. Hence allowance is made for the free horizontal movement of atleast one end of the truss to avoid such temperature stresses and buckling.

Moreover the loads from the truss have to be transferred to the supporting members such as walls, columns etc through its ends and distributed evenly on a sufficiently larger area.

The trusses are to be anchored at their ends to avoid any uplift due to wind suction. Therefore the bearings are provided at the ends of truss to

i) Distribute the loads evenly on a larger area of support
ii) Allow horizontal movement and rotation at ends
iii) Prevent upward lift of truss

Different types of end bearings are

i) Bed plates

It is used in trusses of span less than 10 m, where horizontal movement will not take place. The gusset plates of end joints at eaves are fixed to the bed plates using gusset angles and sufficient number of countersunk rivets as shown in fig. The bed plates are fixed to the bed blocks or masonry by anchor bolts. The leg or flange of bottom tie member is also directly connected to bed plate using countersunk rivets.

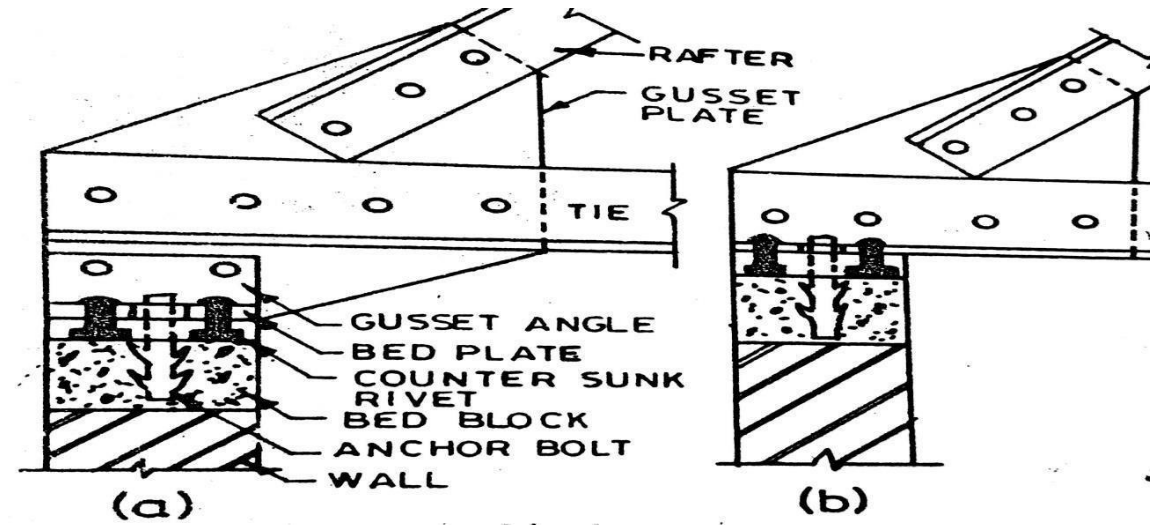
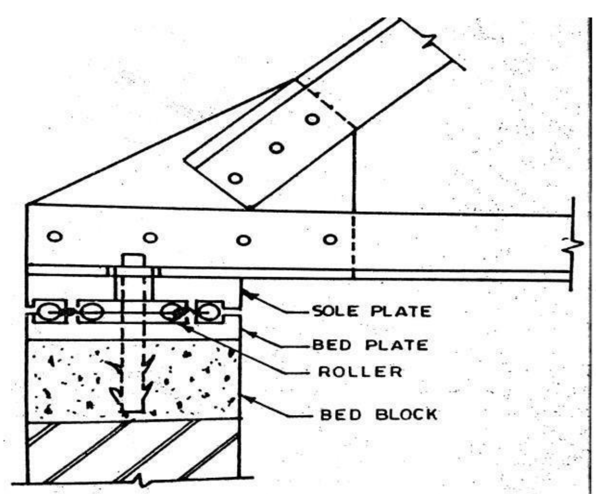


Fig. Bed plates



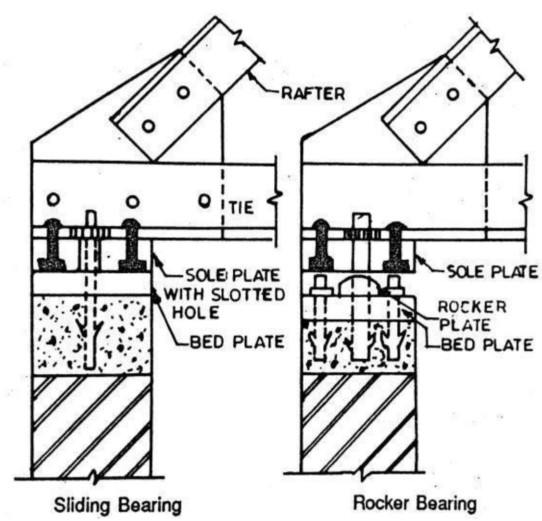


Fig. Roller bearing

Fig. Sliding and Rocker bearings

ii) Sliding bearing

It is used in trusses of span 10 m – 25 m, where horizontal movement will take place. One end of truss is provided with bed plate and other with sliding bearing. It consists of 2 plates – a sole plate at top and a bed plate at bottom. The contact surfaces of these two plates are machined smooth and greased for easy sliding over one another.

iii) Rocker bearing

When the span exceeds 25 m, the ends rotate along with horizontal movement. To allow the rotation, a rocker bearing is provided at one end of the truss. But it will not allow horizontal movement. A rocker plate of cylindrically curved top surface is placed in between the sole plate and bed plate and welded to the bed plate as shown in fig.

iv) Roller bearings

It is provided at one the ends of truss having span > 25 m to allow both rotation and horizontal movement. A sufficient number of cylindrical or segmental rollers of required size are placed in between the sole plate and bed plate as shown in fig. Suitable arrangements are made to keep the rollers in position at the required spacing and for their rolling outwards and inwards within the permissible limit. The sole plate is connected by anchor bolts through slotted holed to the bed plate and bed block.

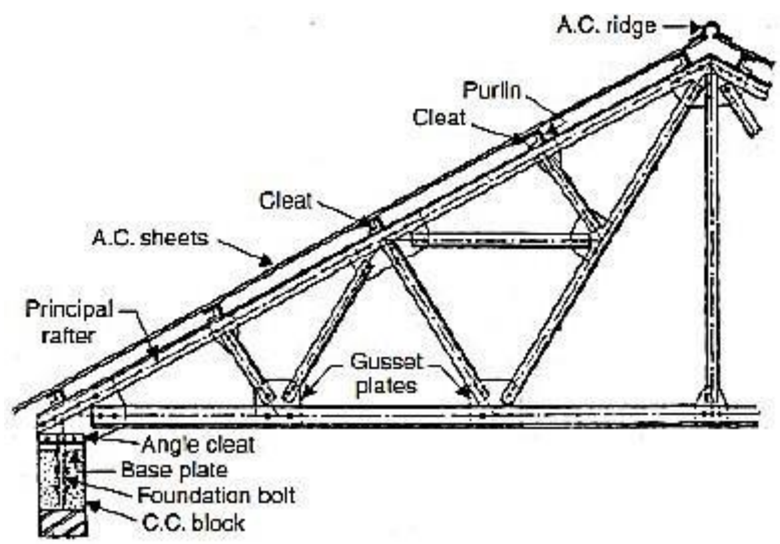
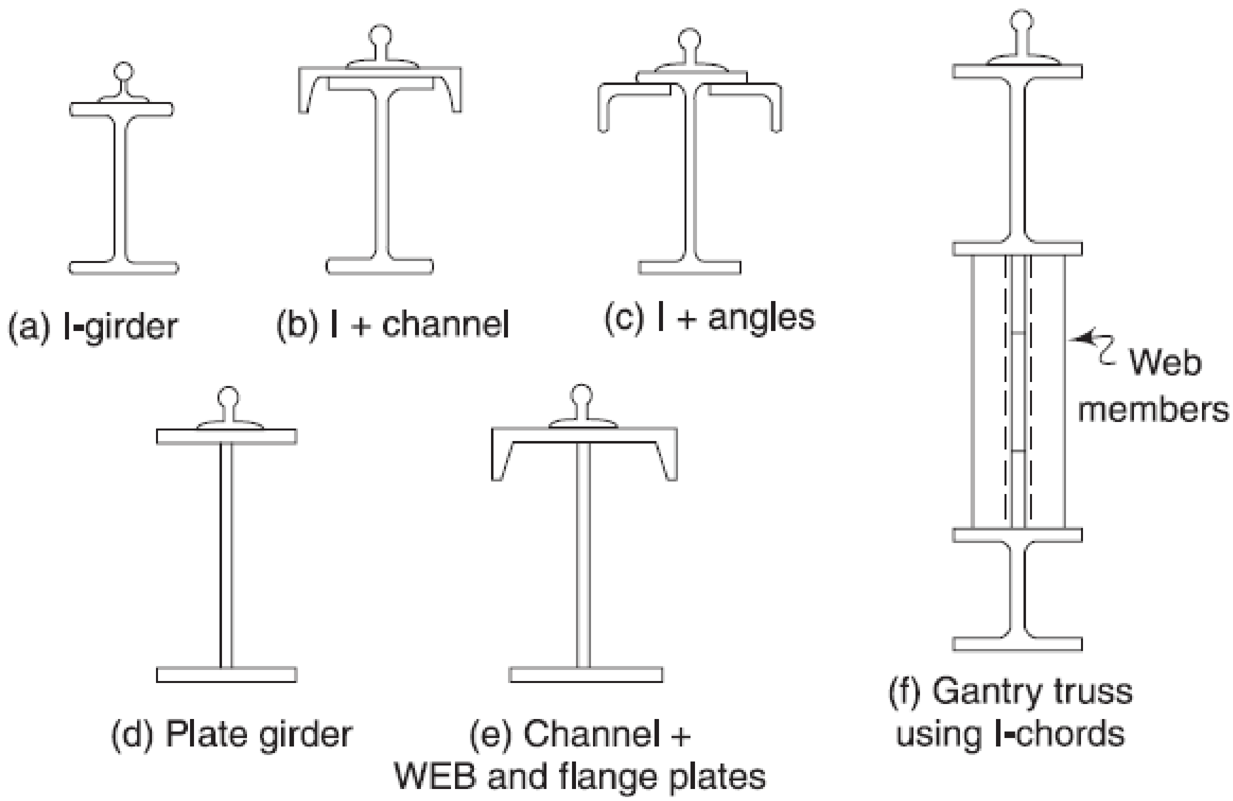


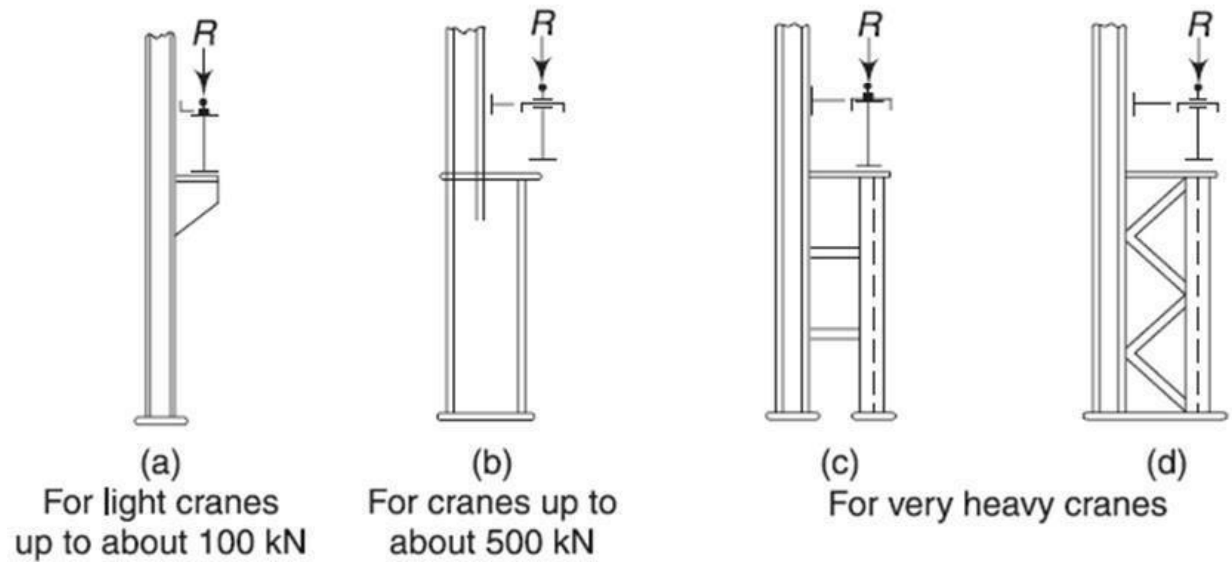
Fig Elevation of a fink roof truss

GANTRY GIRDER

Introduction

In manufacturing plant it is essential to provide overhead travelling crane to transport heavy components of machines from one place to another. The movement of the load is of three dimensional nature. The crane is required to lift heavy mass vertically and horizontally, also the crane with load is required to move along the length of the shed. The cranes are either hand –or electrically operated. The crane moves on rails which are at its ends. The rails are provided on a girder known as a gantry girder. The gantry girder spans over gantry columns. If capacity of crane is moderate, the gantry girders rest on brackets connected to roof column of industrial shed.





Characteristics

♣ Design of gantry girder is a classic example of laterally unsupported beam

♣ It is subjected to in addition to vertical loads and horizontal loads along and perpendicular to its axis

♣ Loads are of dynamic nature and produce vibration

♣ Compression flange requires critical attention

Codal Provisions

♣ Partial safety factor for both dead load and crane load is 1.5 (Table 4, p.29)

♣Partial safety factor for serviceability for both dead load and crane load is 1 (Table 4, p.29)

Deflection Limits (Table 6, p.31)

Category	Maximum Deflection
Vertical deflection	Manually Operated – Span/500
	Electric operated - Span/750 upto 50t capacity
	Electric operated -Span/1000 over 50t capacity
Lateral deflection	Relative displacement between rails supporting 10 mm or crane - span/400

Other Considerations

♣ Diaphragm must be provided to connect compression flange to roof column of industrial building to ensure restraint against lateral torsional buckling at ends.

♣ Span is considered to be simply supported to avoid bumping effect.

Design Steps

The design of the gantry girder subjected to lateral loads is a trial and error procedure. It is assumed that the lateral load is resisted entirely by the compression top flange of the beam and any reinforcing plates, channels, etc. and that the vertical load is resisted by the combined beam. Various steps involved in the design are as follows

1. Maximum wheel load is to be calculated. The wheel load is maximum when the trolley is closest to the gantry girder. This load is to be correspondingly increased for the impact.

2. Maximum bending moment in the gantry girder due to vertical loads is to be computed. This consists of the bending moment due to maximum wheel loads (including impact) and the bending moment due to dead load of the gantry and rails. The bending moment due to dead loads is maximum at the centre of the girder, whereas the bending moment due to wheel load is maximum below one of the wheels. For simplicity, the maximum bending moment due to dead load is directly added to the maximum wheel load moment.

3. Maximum shear force is to be calculated. This consists of the shear force due to wheel loads and dead loads from the gantry girder and rails.

- Generally an I- section with a channel section is chosen, though an I-section with a plate at the top flange may be used for light cranes.
- When the gantry is not laterally supported, the equation to be used to select a trial section is as follows:

$Zp = Mu/fy$ ....(1)

$Zp(trial) = kZp$, ($k= 1.4\text{-}1.5$) ....(2)

Generally, the economic depth of a gantry girder is about (1/12)th of the span. The width of the flange is chosen to be between (1/40) and (1/30)th of the span to prevent the excessive lateral deflection.

4. The plastic section modulus of the assumed combined section is found out by considering a neutral axis which divides the area in two equal parts, at distance y to the area centroid from the neutral axis.Thus,

$Mp = 2fy.A/2y = A.y.fy$, where Ay = plastic modulus Zp......................... (3)

5. When lateral support is provided at the compression (top) flange, the chosen section should be checked for the moment capacity of the whole section (clause 8.2.1.2 of IS800):

$Mdz = BbZpfy/\gamma mo \leq 1.2\ Zefy/\gamma mo$...(4)

Above value should be greater than applied bending moment. The top flange should be checked for bending in both the axes using the following interaction equation:

$(My/Mndy) + (Mz/Mndz) \leq 1.$ ....(5)

6. If the top (compression) flange is not supported, the buckling resistance is to be checked in the same way as in step 4 but replacing fy with the design bending compressive stress fbd (calculated using Section 8.2.2 of the code).

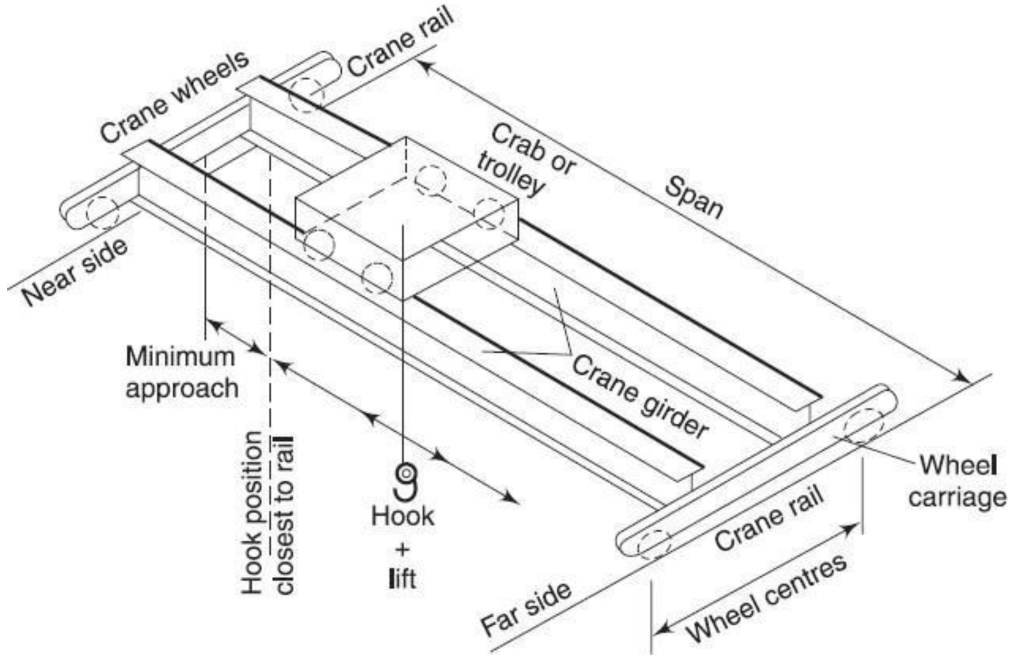
7. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local buckling and, if necessary, load carrying stiffeners must be introduced to prevent local buckling of the web.

8. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local crushing. If necessary, bearing stiffeners should be introduced to prevent local crushing of the web.

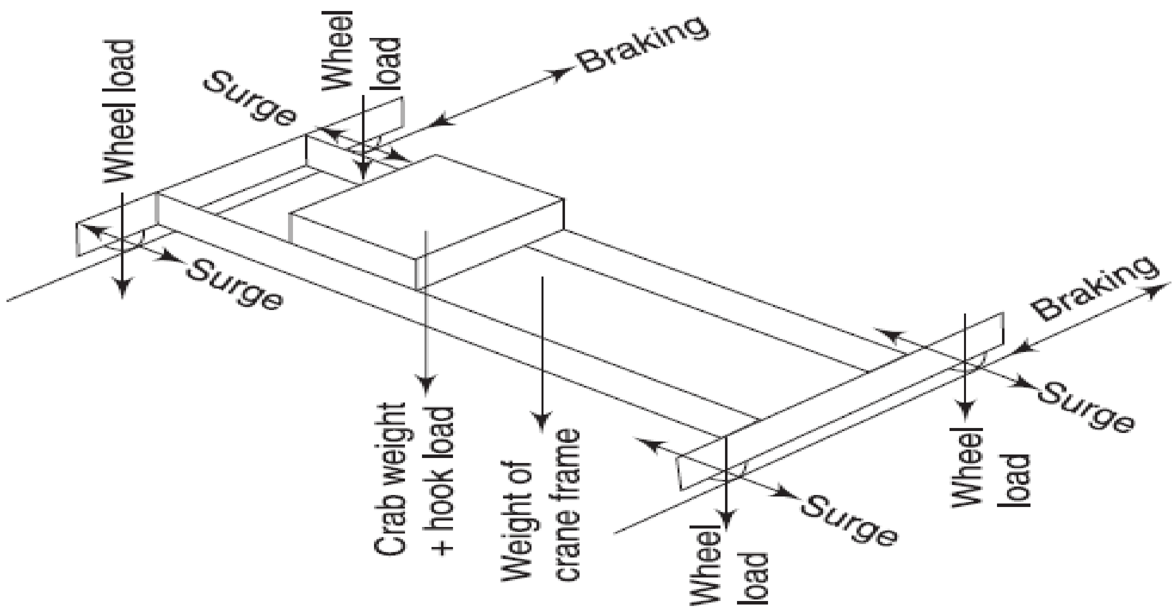
9. The maximum deflection under working loads has to be checked.

10. The gantry girder is subjected to fatigue effects due to moving loads. Normally, light -and Medium -duty cranes are not checked for fatigue effects if the number of cycles of load is less than 5×10^6. For heavy-duty cranes, the gantry girders are to be checked for fatigue loads (see IS 1024 and IS 807). Refer section 13 of the code for design provisions for fatigue effects. The fatigue strength is to be checked at working loads.

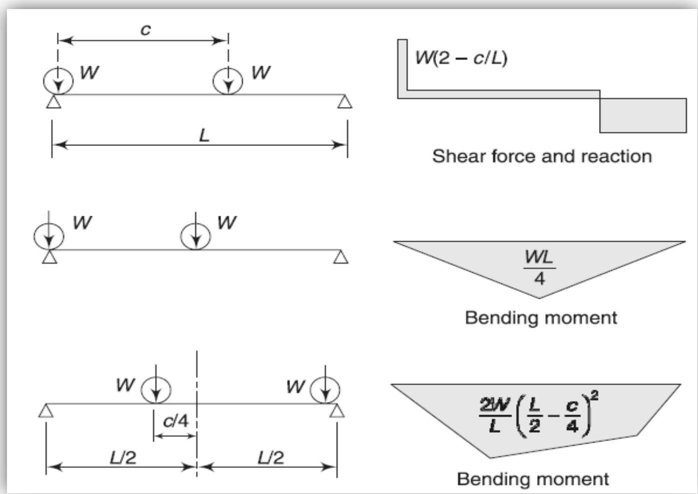
Fig . Various loads acting on gantry girder

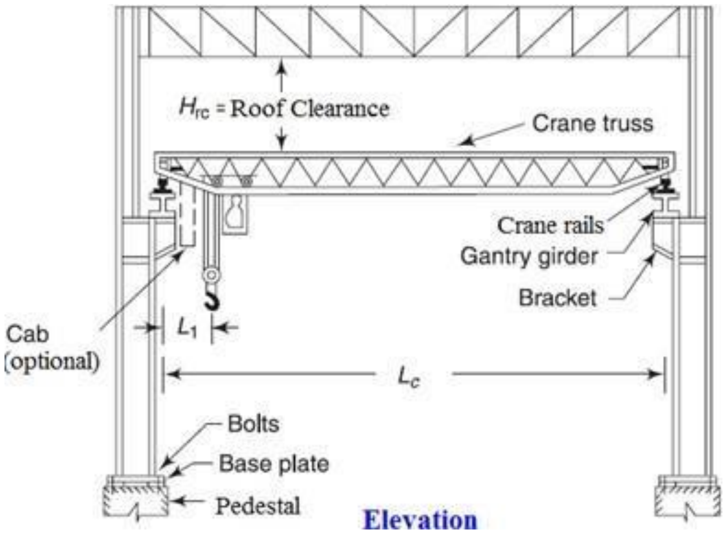


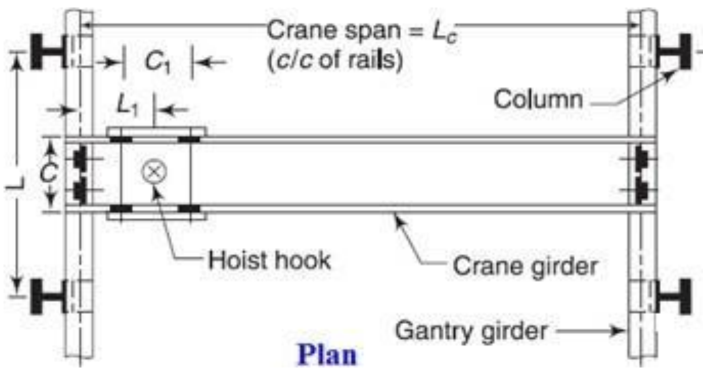
Crane wheels
Crane rail
Crab or trolley
Span
Near side
Minimum approach
Crane girder
Hook position closest to rail
Hook + lift
Wheel carriage
Crane rail
Far side
Wheel centres



Wheel load
Surge
Wheel load
Braking
Surge
Braking
Surge
Crab weight + hook load
Weight of crane frame
Wheel load
Surge
Wheel load


c
W
W
L
W(2 – c/L)
Shear force and reaction
W
W
WL/4
Bending moment
W
W
c/4
L/2
L/2
2W/L (L/2 – c/4)²
Bending moment


Hrc = Roof Clearance
Crane truss
Crane rails
Gantry girder
Bracket
Cab
(optional)
L1
Lc
Bolts
Base plate
Pedestal
Elevation


Crane span = Lc
(c/c of rails)
C1
L1
Column
C
L
Hoist hook
Crane girder
Gantry girder
Plan

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – III - WATERTANKS- SCI1309

UNIT III

Introduction

Permissible Stresses

Thickness Specifications

Elevated Tanks

Circular Tanks

Rectangular Tanks

Pressed Steel Tanks

Introduction

The *steel tanks* are defined as vessels made of steel plates. The steel tanks are either placed on ground or placed on towers. When the steel tanks are placed on ground, then, these are supported on cement concrete foundation or on steel grillage foundation. These steel tanks with vertical cylindrical surface and flat bottom and supported on ground are known as surface tanks. When the steel tanks are placed on the towers or staging, then, the steel tanks are known as *elevated steel tanks*. The steel tanks are placed on towers in order to provide necessary, pressure head., The elevated steel tanks are generally used in connection with pumping stations. The steel tanks are used for storage and supply of water and other liquids, like petroleum, diesel, and kerosene oil.

Permissible Stresses

The steel tanks are designed conforming to code of practice for use of steel in gravity water tanks IS: 805-1968. Permissible Stresses for Tank Plates The values given in IS : 800-1962: shall be multiplied by 0.8 to derive the permissible stress in the tank plates.

Thickness Specifications

The minimum thickness of the steel plates of the tanks shall be 6 mm except for roofs. In case, the tank water contains salts, the thickness of steel plates shall be 1.5 mm more thicker than that calculated.

Nominal Plate Thickness (As per IS 805 -1968)

Nominal Plate Thickness	**Nominal Tank Diameter**
10 mm	9 m and less
12 mm	18 m and less
16 mm	36 m and less
Over 16 mm	All sizes

Elevated Tanks

Elevated tanks do not require the continuous operation of pumps, as it will not affect the distribution system since the pressure is maintained by gravity. Strategic location of the tank can equalize water pressures in the distribution system. However, precise water pressure can be difficult to manage in some elevated tanks.

The pressure of the water flowing out of an elevated tank depends on the depth of the water in the tank. A nearly empty tank probably will not provide enough pressure while a completely full tank may provide too much pressure. The optimal pressure is achieved at only one depth (which is even more specific for standpipes than for tanks elevated on legs). The length of the standpipe causes continual and highly unequal pressures on the distribution system. In addition, a significant quantity of the water in a standpipe is required to produce the necessary water pressure. The water below a certain level is therefore used only as a support, unless booster pumps are available for emergency use of this water.
While elevated tanks provide the best pressure, they are far-more expensive and generally, only used where supply is in high demand.

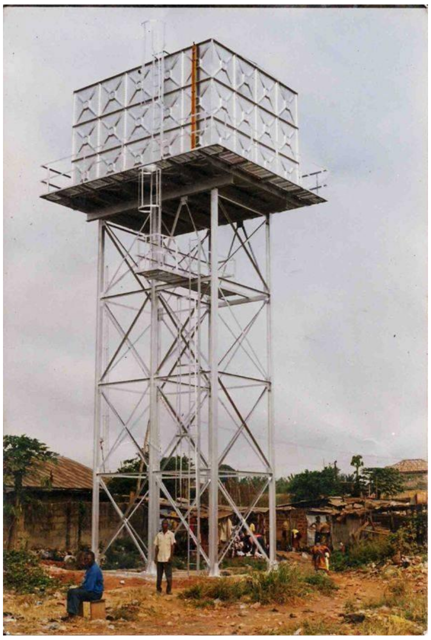
Rectangular Steel Tanks

The rectangular steel tanks are made of steel plates with flat bottom. The widths of steel plates generally adopted are 1.20 m, 1.25 m and 1.30 m depending upon availability of the plates. the thickness of steel plates should not be less than 6 mm. The bottom plates are provided in the transverse direction. These plates are turned up at the ends. These plates and the tee sections are provided in the inner-side to cover to form a butt-joint with the side plates of the tanks.

Elevated Circular Steel Tank

The elevated circular steel tanks are made with suspended bottoms. The circular steel tanks are made with hemispherical, segmental and conical bottoms. The hemispherical, segmental or elliptical bottoms are commonly used. The conical bottom is rarely used. It is difficult to make-a satisfactory connection of conical bottom with the circular girder. The conical bottom tanks are used in railways. For the hemispherical bottom tanks, the theoretical drop of the bottom from spring line is one half the diameter of the ,tank. For the elliptical bottom tanks, this drop is one-fourth the diameter, The steel plates used for the sides of the cylindrical shell are kept slightly different in diameters, so that the courses are placed overlapping each other inside and outside, alternately. The shell plates are shaped to suit the curvature of the tank. The thickness of the plates in the cylindrical shell should not be less than 6 mm. The nominal plate thickness as recommended in IS: 805-1968 for the different nominal tank diameter is adopted as given in Table in IS 805. The minimum thickness of plates in the suspended bottom should not be less than the thickness of plates in the lowest course of the cylindrical part of the tank. The plates are sheared or planed to a suitable bevel along the edges by caulking.

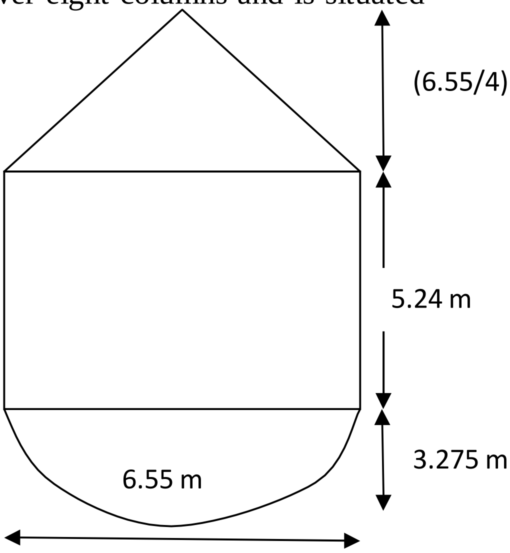
Pressed Steel Tank



Pressed Steel Tank are rectangular in shape. The pressed steel tank have come into existence because of their ease of erection, facility in transport, standard construction and ease in dismantling and re-erection. The pressed steel tank are made of mild steel palters. These plates are heated uniformly in the furnace and formed in press. When these plates are pressed depressions are formed in the plates and the plates gain extra strength.

Design Example:

Design a circular elevated Water tank for capacity of 2.5×10^6 liters. The height of the tank bottom below the ground is 8.7 m. The tank is supported over eight columns and is situated at the railway station of Allahabad



Capacity of Tank – 250 m^3

$$\frac{\pi}{4} \times D^2 \times 0.8\ D + 0.5 \times \frac{4}{3} \times \pi \times (D/2)^3 = 250$$

Gives D = 6.55 m

<u>Thickness of the plates</u>

Let us provide 16 mm Φ power driven rivets for making the connections. Also let the efficiency of the joint be 75 %.

Thickness of Shell Plates

$$t = \frac{wHD}{2\eta\sigma_{at}} = \frac{9.81 \times 10^{-6} \times 5.24 \times 10^{3} \times 6.55 \times 10^{3}}{2 \times 0.75 \times (0.8 \times 0.6 \times 250)} = 1.87 \text{ mm}$$

Add 1.5 mm for corrosion t = 1.87 + 1.5 = 3.37 mm ≮ 6 mm

Thickness of suspended bottom plates

h = [5.24 + (6.55/2)] = 8.515 m

$$t = \frac{wHD}{4\eta\sigma_{at}} = \frac{9.81 \times 10^{-6} \times 8.515 \times 10^{3} \times 6.55 \times 10^{3}}{4 \times 0.75 \times (0.8 \times 0.6 \times 250)} = 1.52 \text{ mm}$$

Add 1.5 mm to account for Corrosion.

t = 1.52 + 1.5 = 3.02 mm ≮ 6 mm

Provide 6 mm thick plates in the hemispherical bottom of tank.

Conical Roof

Provide 5 mm thick plate for the conical roof. The pitch of the roof may be kept as 1 in 4.

Connections

Power driven 16 mm Φ (gross diameter = 17.5 mm) field driven rivets and double riveted lap joints have been used all through.

Cylindrical Shell plates Hoop stress per linear vertical height

$$F_1 = \frac{wHD}{2} = \frac{9.81 \times 10^{-6} \times 5.24 \times 10^{3} \times 6.55 \times 10^{3}}{2} = 168.35 \text{ N/mm}$$

Strength of the revit in single shear = $\frac{\pi}{4}$ x 17.5^2 x 0.8 x 90 = 17318.03 N

Strength of revits in bearing = 17.5 x 6 x 0.8 x 270 = 22680 N.

Revit Value R_v = 17318.03 N

Pitch of Revits = 2 x $\frac{17318.3}{168.35}$ = 205.74 mm k 60 mm (10t = 10 x 6 = 60 mm)

Revits are provided for horizontal joints are provided as that for vertical joints

Hemispherical Bottom Plates

Hoop stress per unit length in radial Joints

$$F_2 = \frac{wHD}{4} = \frac{9.81 \times 10^{-6} \times 8.515 \times 10^3 \times 6.55 \times 10^3}{4} = 136.78 \text{ N/mm}^2$$

There are no inclined or compressive stresses on the hemispherical portion and therefore the connection of the two shells need not be designed again. Hence, provide 16mm power driven field revits at a pitch of 60 mm and double revit lap joint.

Circular girder.

Weight of Water, W_1 = 10 x 250 = 2500 kN= 2500 x 10^3 N

Self Weight of Tank, $W_2 = [\pi \times 6.55 \times 10^3 \times 5.24 \times 10^3 \times 6 + \frac{1}{2} \times 4\pi \times \langle\frac{6.55 \times 10^3}{2}\rangle^2 \times 6] \times 7.9 \times 10^{-5}$ = 83052.77 N

Self Weight of Conical Roof,

$W_3 = \frac{\pi}{2}$ x 6.55 x $10^3 \sqrt{\langle\frac{6.55 \times 10^3}{2}\rangle^2 + \langle\langle\frac{6.55 \times 10^3}{4}\rangle^2\rangle} \times 5$ x 7.9 x 10^{-5} = 14880.74 N

The self weight of the tank and that of the conical roof may be increased by 20 % to account for additional weight of overlap of plates and connectors

$W_4 = 1.2(W_2 + W_3)$ = 1.2 (83052.77 + 14880.74) = 117520.21 N

Total weight of Girder, W = 1500 x (π x 6.55) = 30866.14 N

Total load on the girder, W= 2500 x 10^3 + 117520.21 +30866.14 = 2648386.35 N = 2650.0 x 10^3 kN

For Circular girder supported over eight Columns

Maximum Bending Moment (at supports) = -0.00827 WR

$$= -0.00827 \times 2650 \times 10^3 \times \frac{6.55}{2} = -71733.26 \text{ Nm.}$$

Maximum torsional Support (at 9^0 33' from supports) = 0.00063WR

$$= 0.00063 \times 2650 \times 10^3 \times \frac{6.55}{2} = 5647.6 \text{ Nm}$$

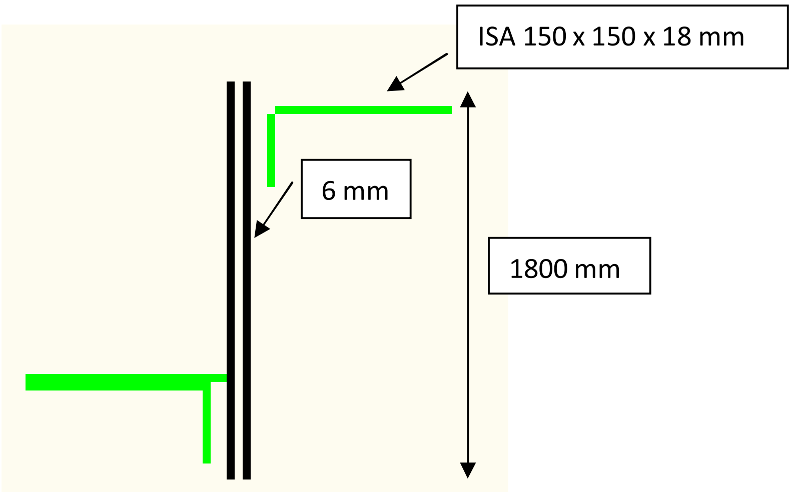
Maximum torsional Support (at 9^0 33' from supports) = 0.00063WR

$$= 0.00063 \times 2650 \times 10^3 \times \frac{6.55}{2} = 5647.6$$

Nm.

Maximum Shear force (at Support) = $\frac{71773.26 \times 10^3}{0.66 \times 250}$ = 165625.00 N

Section Modulus $Z_{req} = \frac{71733.26 \times 10^3}{0.66 \times 250} = 434989.8 \text{ mm}^3$



The Section is selected as shown in figure

$$I_{xx} = \frac{6 \times 1800^3}{12} + 2 \times 2588.7 \times 10^4 + 2 \times 6881 \times (900-56.1)^2 = 1276861.9 \times 10^4 \text{ mm}^4$$

$= 1276861.9\ mm^4$

$z_{prov} = \frac{127681.9 \times 10^4}{900} = 14187354\ mm^3 > 434989.8\ mm^3$ which is alright.

Torsional Constant,

$J = 2 \times \left[\frac{1}{3} \times 200 \times 18^3 + (200-18) \times 18^3\right] + \frac{1}{3} \times 1800 \times 6^3 = 3030048\ mm^4$

Shear Stress due to torsion $= \frac{5467.6}{3030048} \times (18+6) = 43.30\ N/mm^2$

Shear Force due to shear force $= \frac{165625.00}{2\times6881+1800\times6} = 6.743\ N/mm^2$

Total Shear Stress $= 43.30 + 6.743 = 50.04\ N/mm^2 < 100\ N/mm^2$ (0.4 x 250) which is safe.

Bending Stress $\sigma_{bct} = \frac{71773.26 \times 10^3}{14187354} = 5.059\ N/mm^2$

Hoop Stress $\sigma_{at1} = \frac{wHD}{2\eta t} = \frac{9.81 \times 10^{-6} \times 5.24 \times 10^3 \times 6.55 \times 10^3}{2 \times 0.75 \times 6} = 37.41\ N/mm^2$

Maximum principal Stress,

$\sigma_1 = \frac{37.41+5.059}{2} + \sqrt{\left(\frac{37.41+5.059}{2}\right)^2 + 50.04^2} = 75.59\ N/mm^2 < 165\ N/mm^2$ (0.6 x 250)

This is alright

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV - BUNKERS AND SILOS- SCI1309

UNIT IV BUNKERS AND SILOS

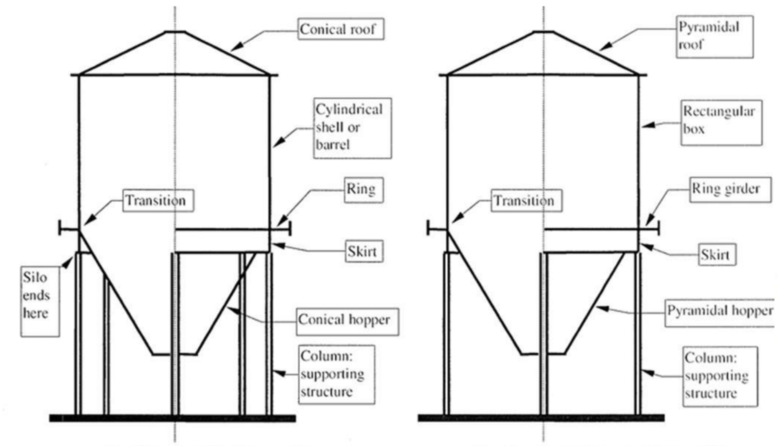
Refer code book – IS9178(i):1979

Introduction

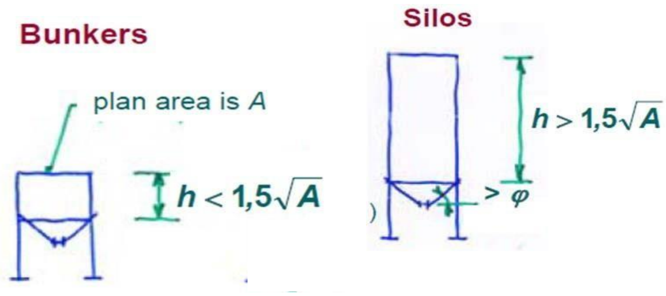
The bunkers are large size shallow bins to store grains, coal and cement. In bunkers, the plane of rupture intersects the free surface of the stored material. Generally, steel bunkers are used to store coal at power plants and loco-running sheds. Generally, these are square or rectangular shaped. The silos are the deep bins for storage. They are circular in shape. The plane of rupture intersects the opposite side of the container. Objectives After studying this unit, you should be able to understand Airy's theory, understand Janssen's theory, know the components of bunkers and silos, design the bunkers, and design the silos.

Silos

- Silos are used by a wide range of industries to store bulk solids in quantities ranging from a few tones to hundreds or thousands of tones.
- The term silo includes all forms of particulate solids storage structure, that might otherwise be referred to as a bin, hopper,grain tank or bunker.
- They can be constructed of steel or reinforced concrete and may discharge by gravity flow or by mechanical means.
- Steel bins range from heavily stiffened flat plate structures to efficient unstiffened shell structures.
- They can be supported on columns, load bearing skirts, or they may be hung from floors.
- Flat bottom bins are usually supported directly on foundations.



a) Circular planform silo b) Rectangular planform silo



Components of Bunkers

1) Main girder, 2) Cross girder, 3) Beam, 4) Sloping plates, 5) Stiffeners, and 6) Openings.

The sectional elevation and plan of the bunker are shown Fig.4.1

Main Girders : The main girders are provided parallel to the longitudinal sides. These are supported on cross girders**.**

Cross Girders : These are provided parallel to the width.

Sloping Plates : These are provided in the bottom portion of the bunker. The inclination is more than the angle of repose of the material for self cleaning.

Openings : These are provided at the bottom of the bunkers. The size is 500 mm square.

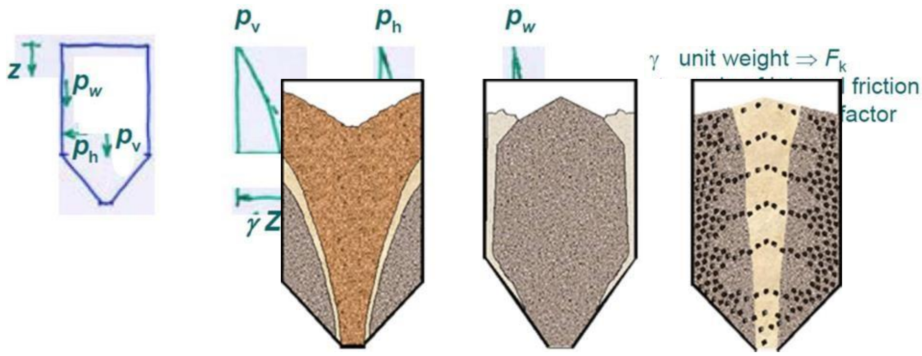
Stiffeners : These are provided with the inclined plates. At top, there are connected with the main - girder. At bottom, these are connected with the bottom plates.

Hopper Silos

Storage of grains (cereals, seeds, legumes, industrial products and other products) that require special storage conditions

Flat Bottom Silos Used for long-term storage of largequantities of grain, seeds and granular products

Actions on silos



Segregation patterns due to different mechanisms

Segregation

For stored material with a wide range of density, size and shape, the particles tend to segregate. The greater the height of free fall on filling, the greater the segregation.

Segregation may create areas of dense material. More seriously, coarse particles may flow to one side of the bin while fine cohesive particles remain on the opposite side. An eccentric flow channel may occur, leading to unsymmetrical loads on the wall. The concentration of fine particles may also lead to flow blockages.

Rapid Filling and Discharge

The rapid discharge of bulk solids having relatively low permeability to gasses can induce **negative air pressures** (internal suction) in the bin. Rapid filling can lead to greater consolidation, and the effects are discussed above.

Powders

The rapid filling of powders can aerate the material and lead to a temporary decrease in bulk density, cohesiveness, internal friction and wall friction. In an extreme case, the pressure from an aerated stored material can be hydrostatic.

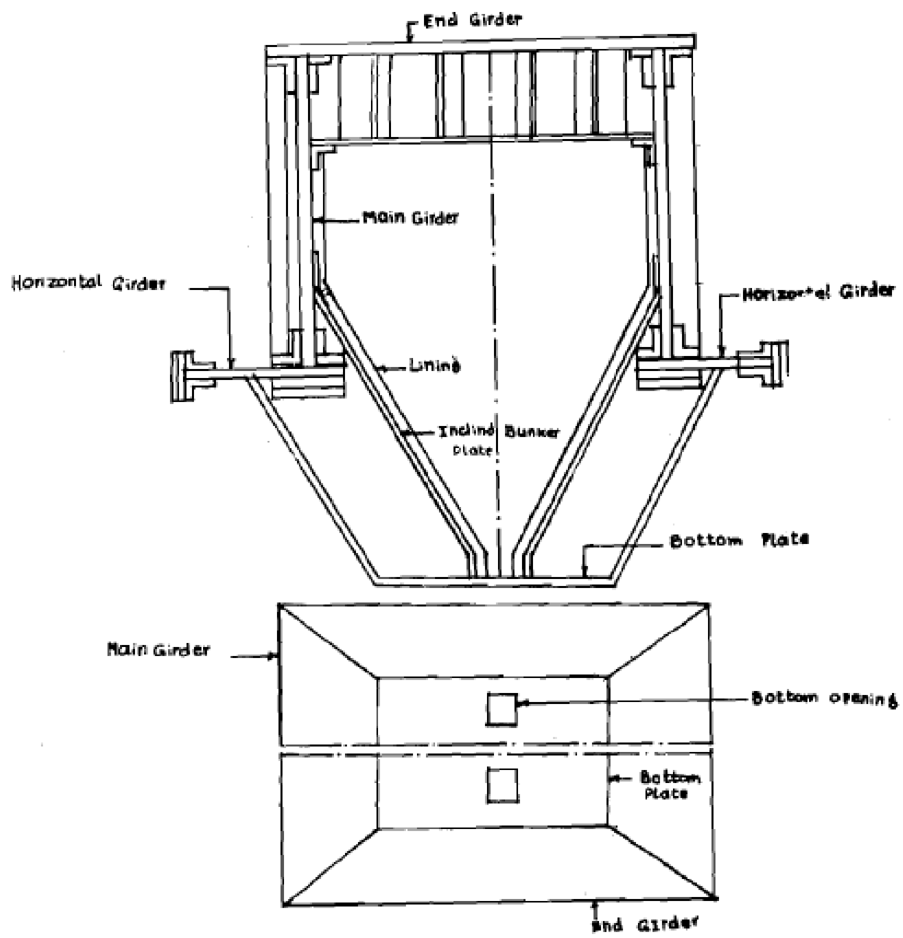


Fig. 4.1 Section and plan of steel bunker

Airy's Theory

By using this theory the horizontal pressure per unit length of periphery and position of plane of rupture can be determined. The Airy's theory is actually based on Coulomb's wedge theory of Earth Pressure.

Janssen's theory

Assumptions

1) Most of the weight of the material stored in the bin is supported by friction between the material and the vertical wall.

2) Weight transferred to the hopper bottom is very less. (Hence Rankine or Coulomb's lateral pressure theory cannot be applied.

3) The vertical wall of the bin is subjected to vertical force and horizontal pressure.

Design of Bunkers

Step 1: Force analysis

a) Calculate the vertical forces.

b) Calculate the horizontal forces using code specification.

c) Calculate the bursting forces H,1, H2, H3 and H4 Using equation of equilibrium.

d) Calculate the pressure p_v, p_w, p_h on trough walls.

e) Calculate the normal and tangential pressures.

f) Calculate the normal load on trough.

Step 2: Design of trough plates

a) Span = spacing of stiffeners.

b) Considering truss-way bending, calculate the maximum bending,

$M_1 = pL^2/ (2x12)$

where, p = maximum normal pressure and L = span of trough plate.

c) Calculate the thickness required
$t= [6M_1/(\sigma_{bc}.L)]^{1/2}$ but Min. thickness = 6 mm.

Step 3: Design of inclined stiffeners in trough

a) Calculate the maximum BM and (M2) and direct tension at mid-span.
b) Choose suitable T-section with plate.
c) Calculate A, Ixx and Zxx
d) Check for tensile stress and bending stress.

Step 4: Design of plate stiffeners for trough

These are provided perpendicular to the T-stiffeners.

a) Calculate the maximum BM.

$M_3 = pL^3/ (2x12)$

b) Calculate the section modulus Zrequired.

c) Assuming thickness (t), find the depth of plate

$td^2/6 = Z$

Step 5: Design of vertical plate

a) Calculate the maximum BM, $M_4 = pL^2/ (2x12)$

b) Calculate the thickness required

$t= [6M_4/(\sigma_{bc}.L)]^{1/2}$ but Min. thickness = 6 mm.

Step 6: Design of vertical stiffeners

a) Calculate the max. BM, $M_5 = pL^2/ (2x8)$

b) Calculate Zrequired = M_5/ σ_{bc}

c) Choose a standard T-section with plate

d) Calculate A, Ixx , Zxx

e) Check for bending stress.

Step 7: Design main (longitudinal) girder

a) Calculate the moment due to Hl at top, $M_6 = H_1L^2/8$

b) Calculate Zrequired = M_6/ σ_{bc}

c) Choose the suitable section.

Step 8: Design of horizontal girder

a) Calculate the moment due to H_3

$M_7 = H_3L^2/8$

b) Calculate Z required.

c) Select the suitable section.

Design of Silos

Generally, the Silos are circular in shape and is shown in fig 4.2. These are designed similar to bunkers.

Design Procedure

Step 1: Calculation of horizontal pressure
Refer code book IS 9178(i) – Pg no 12, table 3,

Pg no . 14,

$Z_o = \min(Z_{of}, Z_e)$

$P_i(Z) = P_{i\,max}(1 - e^{-z/Zoe})$

By using the codal provisions, find the horizontal and vertical pressures at different depths at some intervals say 3 m, 4 m/5 m.

Step 2: Calculation of max. hoop tension

HOOP tension, $H_1 = (p_h)$ max .D/2

Step 3: Design of wall plate

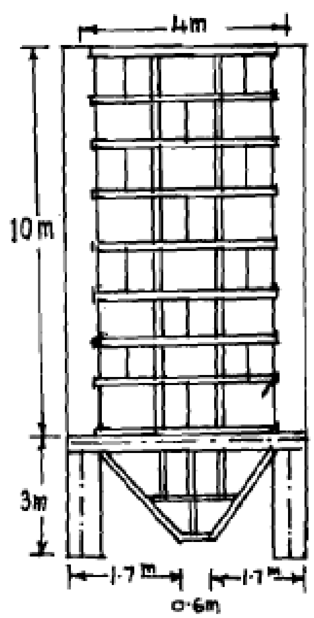


Fig.4.2 Sectional elevation of Silo

Calculate total vertical load, self weight, weight of lining, weight of top cover.
Calculate the vertical load.
Calculate thickness of plate from combined loading.
Step 4: Design of hopper
Calculate the total vertical load.
Calculate the direct tension.
Calculate the thickness = Direct tension/(σ_{at} x 1000 mm)

Step 5: Design of ring beam

Calculate the weight of stored material, self-weight of silos lining cover, platform.
Calculate the reaction, SF, BM, torsion and compression.
Calculate σ_{ac}, $\sigma_{ac,cal}$, σ_{bc} , $\sigma_{bc,cal}$
Check for combined stresses.

Failure of silos

The major causes of silo failures are due to shortcomings in one or more of four categories:

- Failure due to design
- Failure due to construction
- Failure due to usage
- Failure due to maintenance.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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – V - LIGHT GAUGE SECTIONS - SCI1309

UNIT 5

LIGHT GAUGE SECTIONS

USE IS 811 : 1987 code book – Properties of light section

USE IS 801 : 1975 code book – Basic Design Steps

Introduction

Thin sheet steel products are extensively used in building industry, and range from purlins to roof sheeting and floor decking. Generally these are available for use as basic building elements for assembly at site or as prefabricated frames or panels. These thin steel sections are ***cold-formed,*** i.e. their manufacturing process involves forming steel sections in a cold state (i.e. without application of heat) from steel sheets of ***uniform*** thickness. Sometimes they are also called ***Light Gauge Steel Sections*** or ***Cold Rolled Steel Sections***. The thickness of steel sheet used in cold formed construction is usually *1* to *3* mm. Much thicker material up to *8 mm* can be formed if pre-galvanised material is not required for the particular application. The method of manufacturing is important as it differentiates these products from ***hot rolled steel*** sections. Normally, the yield strength of steel sheets used in cold- formed sections is at least $280\ N/mm^2$, although there is a trend to use steels of higher strengths, and sometimes as low as $230\ N/mm^2$. Manufacturers of cold formed steel sections purchase steel coils of *1.0* to *1.25 m* width, slit them longitudinally to the correct width appropriate to the section required and then feed them into a series of roll forms. These rolls, containing male and female dies, are arranged in pairs, moving in opposite direction so that as the sheet is fed through them its shape is gradually altered to the required profile. The number of pairs of rolls (called ***stages***) depends on the complexity of the cross sectional shape and varies from *5* to *15*. At the end of the rolling stage a flying shearing machine cuts the member into the desired lengths.

An alternative method of forming is by press - braking which is limited to short lengths of around *6 m* and for relatively simple shapes. In this process short lengths of strip are pressed between a male and a female die to fabricate, one fold at a time and obtain the final required shape of the section. Cold rolling is used when large volume of long products are required and press breaking is used when small volume of short length products are produced.

Galvanizing (or zinc coating) of the preformed coil provides very satisfactory protection against corrosion in internal environments. A coating of $275\ g/m^2$ (total for both faces) is the usual standard for internal environments. This corresponds to zinc coating of 0.04 *mm*. Thicker coatings are essential when moisture is present for long periods of time. Other than galvanising, different methods of pre-rolling and post-rolling corrosion protection measures are also used.

Advantages of Cold formed Sections

Cold forming has the effect of increasing the yield strength of steel, the increase being the consequence of cold working well into the strain-hardening range. These increases are predominant in zones where the metal is bent by folding. The effect of cold working is thus to enhance the mean yield stress by *15% - 30%*. For purposes of design, the yield stress may be regarded as having been enhanced by a minimum of *15%*.

Some of the main advantages of cold rolled sections, as compared with their hot-rolled counterparts are as follows:

- Cross sectional shapes are formed to close tolerances and these can be consistently repeated for as long as required.
- Cold rolling can be employed to produce almost any desired shape to any desired length.
- Pre-galvanised or pre-coated metals can be formed, so that high resistance to corrosion, besides an attractive surface finish, can be achieved.
- All conventional jointing methods, (i.e. riveting, bolting, welding and adhesives) can be employed.
- High strength to weight ratio is achieved in cold-rolled products.
- They are usually light making it easy to transport and erect.

It is possible to displace the material far away from the neutral axis in order to enhance the load carrying capacity (particularly in beams).

There is almost no limit to the type of cross section that can be formed.

Some typical cold formed section profiles are sketched in Fig.5.1.

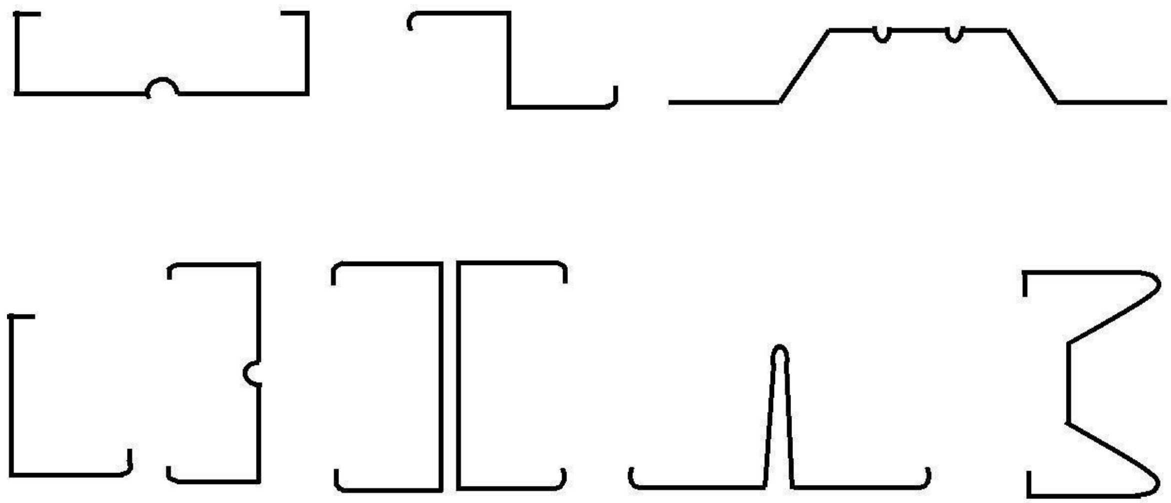


Fig. 5.1 Typical Cold Formed Steel Profiles

While the strength to weight ratios obtained by using thinner material are significantly higher, particular care must be taken to make appropriate design provisions to account for the inevitable buckling problems.

Types of Stiffened and Unstiffened Elements

As pointed out before, cold formed steel elements are either ***stiffened*** or ***unstiffened***. An element which is supported by webs along both its longitudinal edges is called a ***stiffened*** element. An ***unstiffened*** element is one, which is supported along one longitudinal edge only with the other parallel edge being free to displace. Stiffened and unstiffened elements are shown in Fig. 5.2.

An ***intermittently stiffened element*** divided into two or more narrow stiffeners is made of a very wide thin element which has been sub elements by the introduction of intermediate formed during rolling. In order that a flat compression element be considered as a ***stiffened element***, it should be supported along one longitudinal edge by the web and along the other by a web or lip or other edge stiffener, (eg. a bend) which has sufficient flexural rigidity to maintain straightness of the edge, when the element buckles on loading. A rule of thumb is that the depth of simple “lips” or right angled bends should be at least one-fifth of the adjacent plate width.

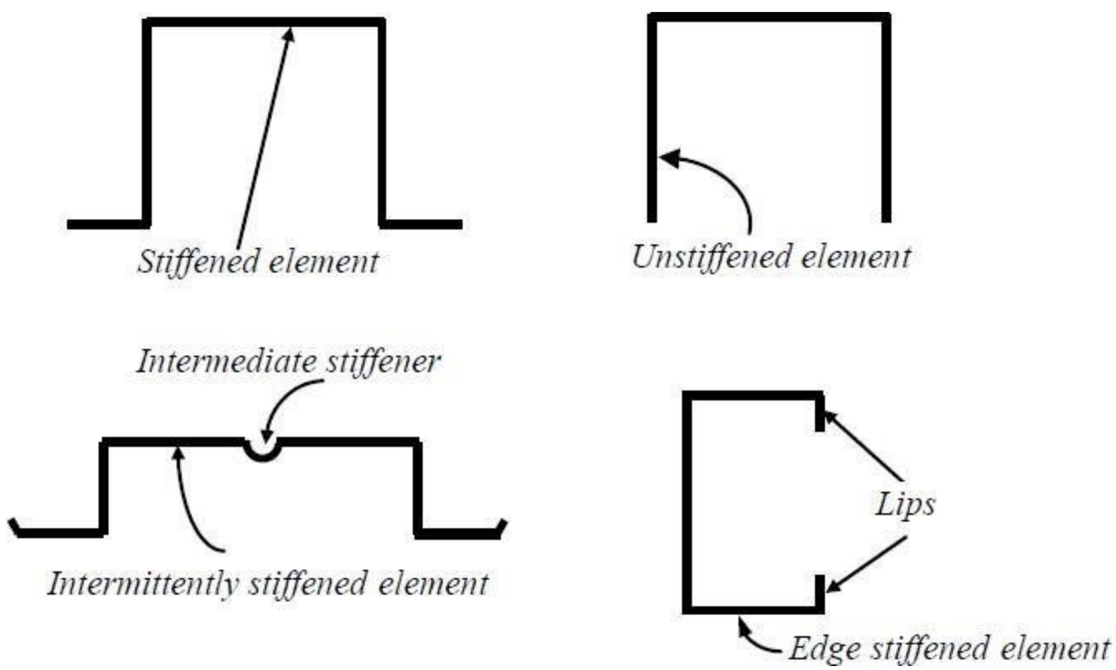


Fig. 5.2 Stiffened and Unstiffened elements

Local Buckling

Local buckling is an extremely important facet of cold formed steel sections on account of the fact that the very thin elements used will invariably buckle before yielding. Thinner the plate, the lower will be the load at which the buckles will form.

Elastic Buckling of Thin Plates

A flat plate simply supported on all edges and loaded in compression (as shown in Fig. 5.3(a)) will buckle at an elastic critical stress. When one of the edges is free to move and the opposite edge is supported, (as shown in Fig. 5.3(b), the plate buckles at a significantly lower load, as K reduces dramatically to *0.425.*

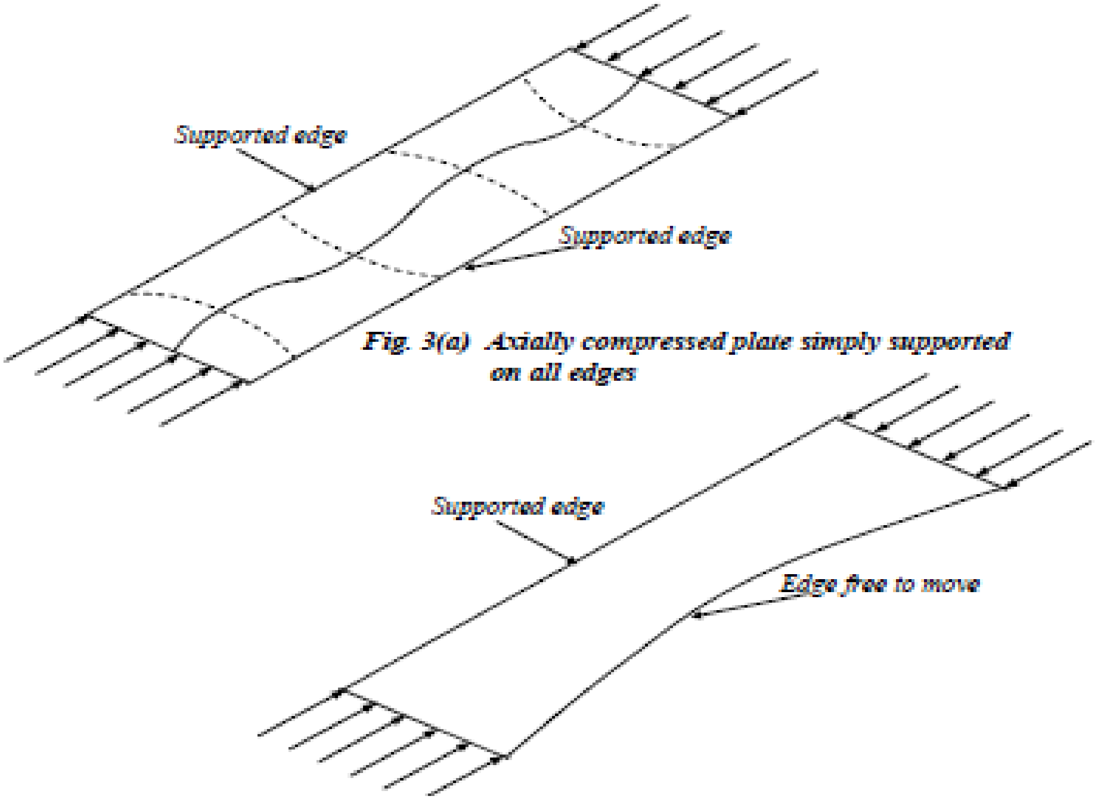


Fig. 5.3(b) Axially compressed plate simply supported on all edges

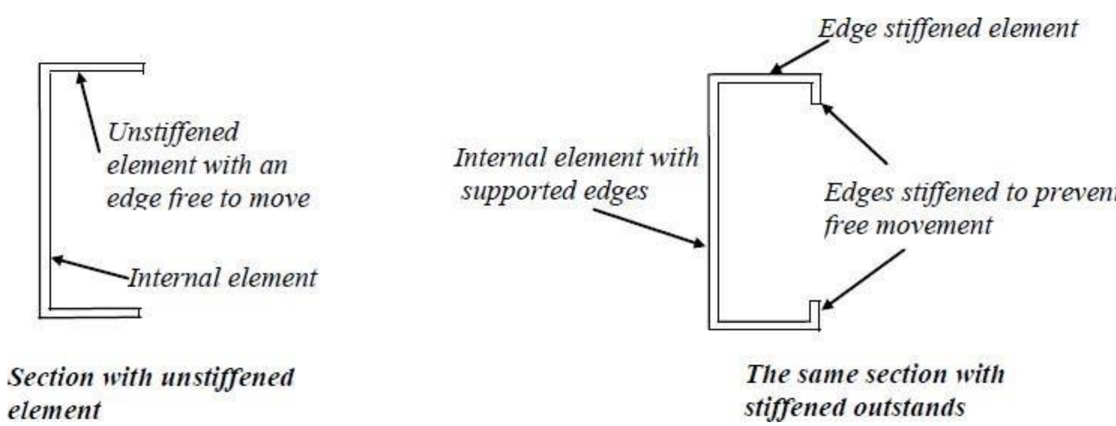


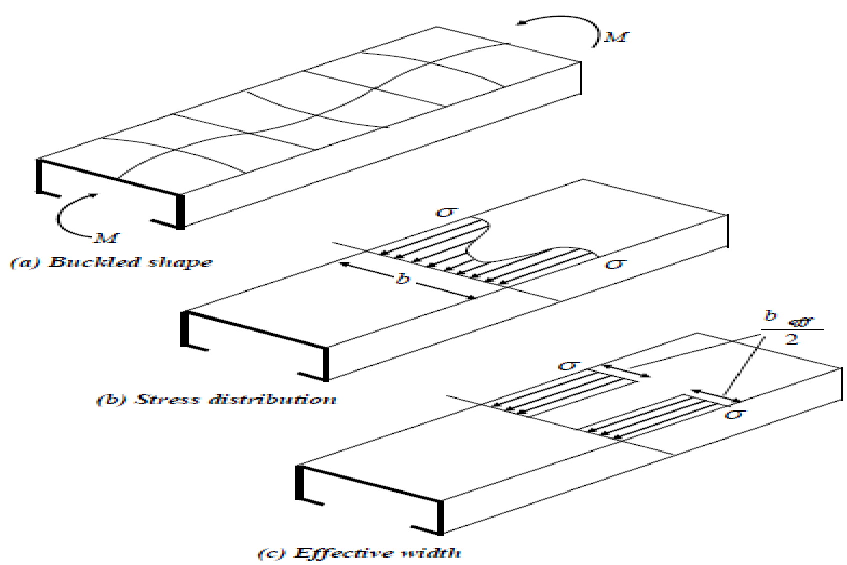
Fig. 5.4 The technique of stiffening the element

This shows that plates with free edges do not perform well under local buckling. To counter this difficulty when using cold formed sections, the free edges are provided with a lip so that they will be constrained to remain straight and will not be free to move. This concept of stiffening the elements is illustrated in Fig. 5.4.

Post Critical Behaviour:

Consider the channel subjected to a uniform bending by the application of moments at the ends. The thin plate at the top is under flexural compression and will buckle as shown in Fig.

(a). This type of buckling is characterized by ripples along the length of the element. The top plate is supported along the edges and its central portion, which is far from the supports, will deflect and shed the load to the stiffer edges. The regions near the edges are prevented from deflecting to the same extent. The stresses are non-uniform across the section as shown in Fig. 5.5 (b). It is obvious that the applied moment is largely resisted by regions near the edges(i.e. elements which carry increased stresses) while the regions near the centre are only lightly stressed and so are less effective in



resisting the applied moment.

Fig. 5.5 Local Buckling Effects

This tendency is predominant in plates having *b/t* (breadth/thickness) ratios of *30-60*. For plates having a *b/t* value in excess of *60*, the in-plane tensile stresses or the “membrane stresses” (generated by the stretching of the plates) resist further buckling and cause an increase in the load-carrying capacity of wide plates.

Effective Width Concept

The effects of local buckling can be evaluated by using the concept of ***effective width***. Lightly stressed regions at centre are ignored, as these are least effective in resisting the applied stresses. Regions near the supports are far more effective and are taken to be fully effective. The section behaviour is modelled on the basis of the effective width (b_{eff}) sketched in Fig. 5.5(c).

The effective width, (b_{eff}) multiplied by the edge stress is the same as the mean stress across the section multiplied by the total width (b) of the compression member.

The ***effective width*** of an element under compression is dependent on the magnitude of the applied stress f_c, the width/thickness ratio of the element and the edge support conditions.

How are light gauge steel products produced?

A very wide range of lightweight structural sections are produced by cold forming thin gauge strip material to specific section profiles. These are often termed light gauge or cold formed steel sections. In most cases, galvanized steel strip material is used. The cold rolling process begins with coils of galvanized strip steel that are uncoiled, slit into appropriate widths and then cold roll-formed into the final product form.

Profile shapes and section sizes do vary but most sections use lips at free edges and indented profiles to provide stiffness and avoid premature failure by local buckling. Thicknesses for load bearing products typically vary from 1.2 mm to 3.2 mm.

5.7 **Light gauge steel infill walls**

Infill walling is used across many different construction sectors; health, education, commercial, residential and leisure and is the generic name given to external walls that are built between the floors of the primary structural frame of a building, and which provide support for the cladding system. Infill walls do not support floor loads but they do resist wind loads applied to the façade, and may be used within both steel and concrete-framed buildings.

Light gauge steel load-bearing walls "As well as the benefits of fast track construction and ease of handling, light gauge steel also offers a highly sustainable method of construction. Production is energy efficient, it optimises raw material use, and lighter structures mean that footings can be less extensive," says Kingspan Insulated Panels Regional Manager Paul Grimshaw.

Light gauge steel load-bearing walls are used in light steel-framed buildings and modular construction, supporting floor loads, loads from walls above and resisting lateral wind loads. They generally include bracing to provide lateral stability to the building. Light gauge steel load-bearing walls use vertical C sections of typically 100 mm depth. Both internal and external walls may be designed as load-bearing.

Wall panels are typically pre-fabricated as storey-high units or may be site assembled from C sections that are delivered cut-to-length, but this is less common

Axially Compressed Columns

Local buckling under compressive loading is an extremely important feature of thin walled sections.

In analysing column behaviour, the first step is to determine the effective area (A_{eff}) of the cross section by summing up the total values of effective areas for all the individual elements.

The ultimate load (or squash load) of a short strut is obtained from

$$P_{cs} = A_{eff} \,.\, f_{yd} = Q.\, A.\, f_{yd}$$

where P_{cs} = ultimate load of a short strut

A_{eff} = sum of the effective areas of all the individual plate elements

Q = the ratio of the effective area to the total area of cross section at yield stress

Tension Members

If a member is connected in such a way as to eliminate any moments due to connection eccentricity, the member may be designed as a simple tension member. Where a member is connected eccentrically to its axis, then the resulting moment has to be allowed for. The tensile capacity of a member (P_t) may be evaluated from

$$P_t = A_e \,.\, P_y$$

where

A_e is the effective area of the section making due allowance for the type of member (angle, plain channel, Tee section etc) and the type of connection (eg. connected through one leg only or through the flange or web of a T- section).

p_y is design strength (N/mm^2)

The area of the tension member should invariably be calculated as its gross area less deductions for holes or openings. (The area to be deducted from the gross sectional area of a member should be the maximum sum of the sectional areas of the holes in any cross section at right angles to the direction of applied stress).

The Indian code IS: 801-1975 is in the process of revision and it is probable that a similar enhancement will be allowed for cold rolled steel sections also.

DESIGN OF BEAMS

The beam is designed as a simply supported at ends and laterally supported through out the span. The bending stress of top compression flange should be less than the basic design stress 0.6fy.

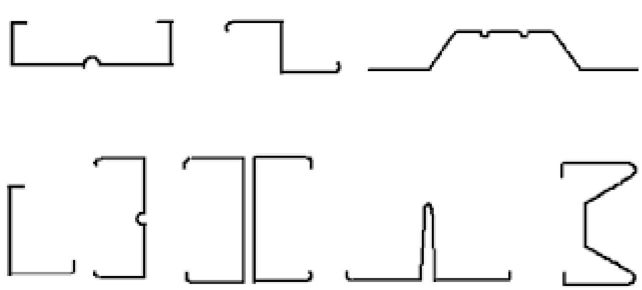
The bending stress is given by M.y/I.

5.12 To determine the allowable load:

Determine

1. Basic design stress – pg no 11, IS 801
2. Limiting flat width ratio - pg no 6, IS 801
3. Actual flat width ratio
4. Effective width
5. Effective area of span
6. Form factor $Q = A_{eff} / A$
7. Limiting slenderness ratio

8. Safe load = Aeff x Allowable compressible stress (F_{a1})



Light weight frame

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