STRUCTURAL DESIGN-2 STUDY MATERIAL AND QUESTION BANK

CIVIL ENGINEERING DEPARTMENT

GOVERNMENT POLYTECHNIC DEOGARH

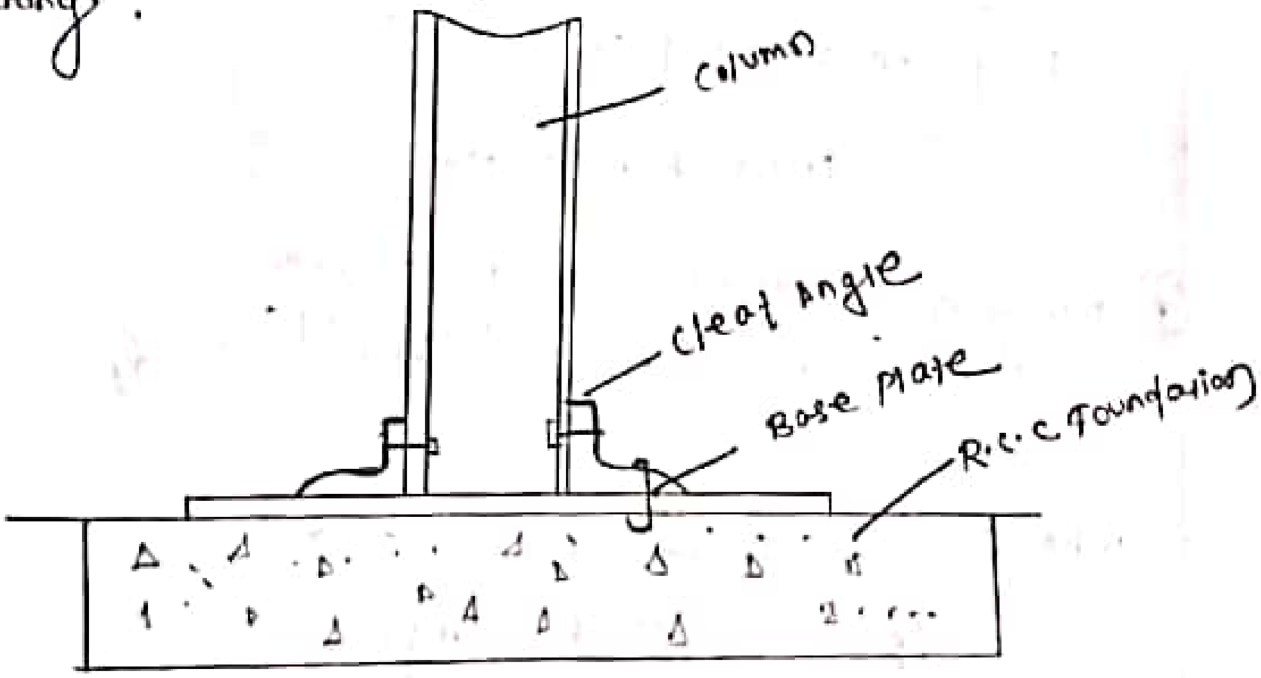
COLUMN BASES.

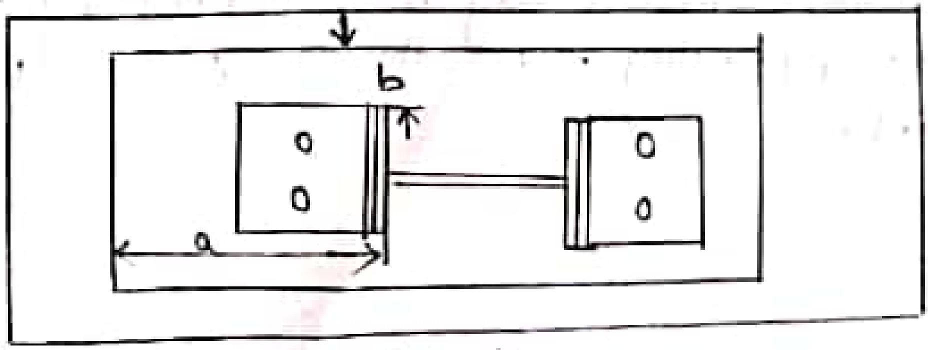
Column bases transmit the column load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing strength. There are two type of column base commonly used in practice.

1. Slab Base
2. Gusseted Base

SLAB BASE:-

These are used in column carrying small loads. In this type, the column is directly connected to the base plate through cleat angles as shown in figure. The load is transferred to the base plate through bearing.





Slab Base

DESIGN OF SLAB BASE:-

The design of slab base consists in finding the size and thickness of slab base. In the procedure given below it is assumed that the pressure is ~~linearly~~ distributed uniformly under the slab base.

state of Base plate:-

(1) Find the bearing strength of concrete which is given by $= 0.45 f_{ck}$

(2) Therefore, area of base plate required $= \frac{P_u}{0.45 f_{ck}}$, where P_u is factored load.

Select the size of base plate, for economy, as far as possible keep the projections 'a' and 'b' equal

Thickness of Base plate:-

(1) Find the intensity of pressure

$$w = \frac{P_u}{\text{Area of base plate}}$$

(2) Minimum thickness required is given by

$$t_s = \left[\frac{2.5 w (a^2 - 0.3 b^2)\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$

where

t_s = thickness of base plate

t_f = thickness of flange

The above formula may be derived by taking $\mu = 0.3$ and using plate theory finding bending moment.

Connections:-

(1) Connect base plate to foundation concrete using four 20mm diameter and 300mm long anchor bolts.

(2) If bolted connection is to be used for connecting column to base plate, use 2 ISA 6565, 6mm thick angle with 20mm bolts.

(3) If weld is to be used for connection column to base. Check the weld length of fillet welds.

Problem:-

Design a slab base for a column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 kN. M_{20} Concrete is used for the foundation. provide welded connection betn column and base plate.

Solution:-

Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 20 = 9 \, N/mm^2$

Factored load $P_u = 1000 \, kN$

$\therefore$ Area of base plate required $= \dfrac{1000 \times 10^3}{9}$

$= 111111 \, mm^2$

provided $= 360 \times 310$ size plate,

Area provided $= 360 \times 310 = 111600 \, mm^2$

Pressure $= \dfrac{1000 \times 10^3}{111600} = 8.96 \, N/mm^2$

Projection are

$$a = \frac{360 - 300}{} = 30mm$$

$$h = \frac{310 - 250}{2} = 30mm$$

$$t_s = \left[\frac{2.5 \times 8.96 (30^2 - 0.3 \times 30^2) \times 1.1}{250}\right]^{0.5}$$

$= 7.88mm$

Thickness of flange of ISHB 300 @ 577 N/m is 10.6mm

Provided 12 mm thick plate.

Connecting 360×310×12mm plate to concrete foundation use 4 bolts of 20 mm dia 300 mm long to anchor the plate.

welds:- properly machined column is to be connected to base plate using fillet weld.

Total length available for welding

$= 2(250+250-7.6+300-2\times10.6) = 1542.4mm$

Strength of weld $= \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37 \ N/mm^2$

Let s be the size of weld. Then effective area of weld $= 0.7 s L_e$.

where Le is effective ~~length~~ length

∴ The design condition is $0.7 s L_e \times 189.37 = 1000\times$

$sL_e = 7543.8$

Using 6mm weld Le = 1257 mm

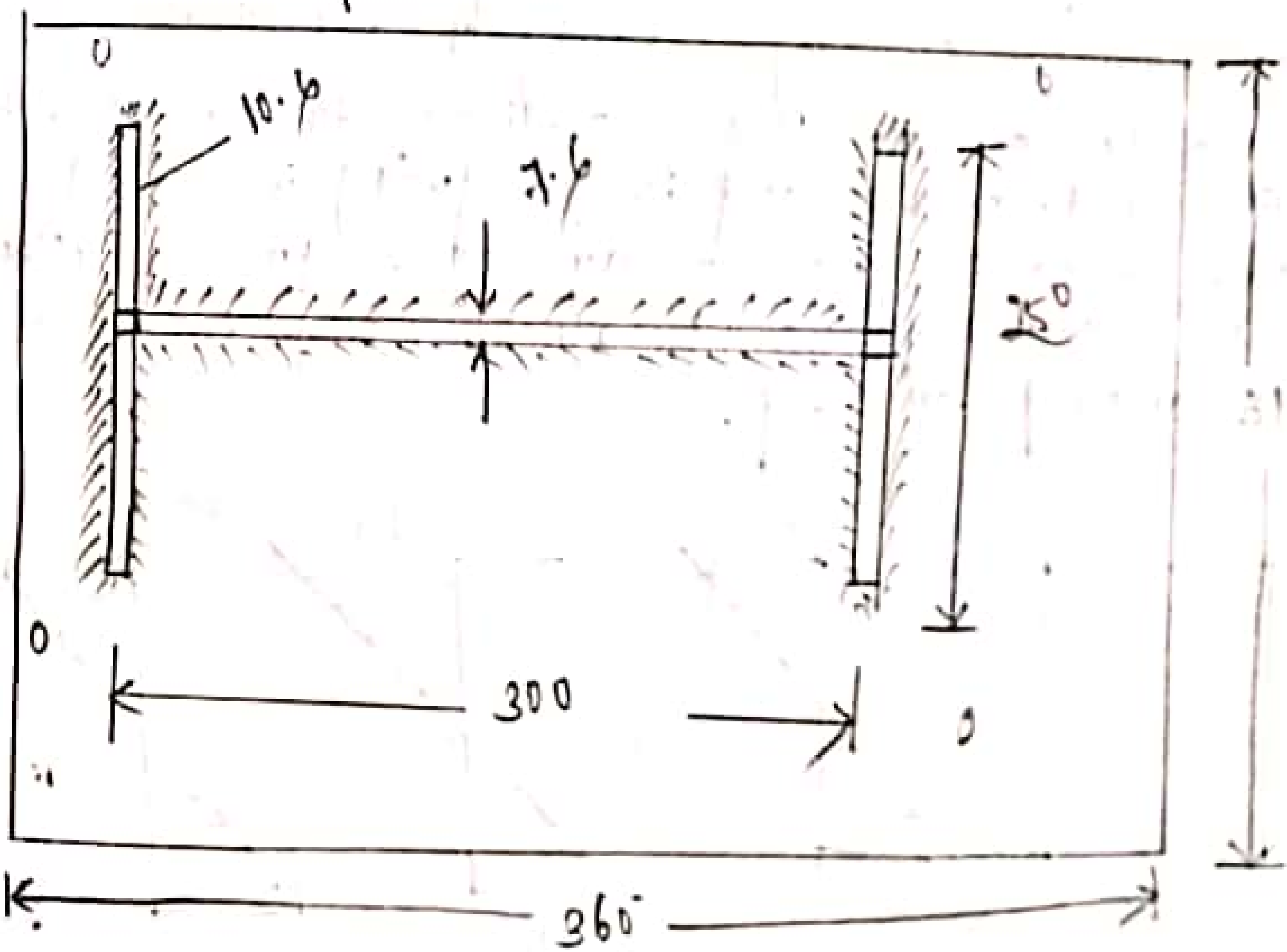
After deducting for end returns of the weld at the rate of twice the size of the weld at each end.

Available effective length = 1542.4 − 2×6×No of return

= 1542.4 − 2×6×12

= 1398.4 > 1257 mm

Hence 6mm weld is adequate:-



TIMBER

TYPES OF TIMBER:-

↳ The various types of timber that are used for structural purpose are teak, sal, deodar, rose wood etc. In addition to these, there are many other types of timbers which are also used for structural purposes.

↳ Species of timber recommended for constructional purpose are classified into 3 groups on the basis of their strength properties namely, modulus of elasticity (E) and extreme fibre stress in bending and tension (fb) as given below.

① Group-A – E above 12.6×10^3 mpa & fb above 18 mpa

② Group B – E above 9.8×10^3 mpa upto 12.6×10^3 mpa & fb above 12 mpa upto 18 mpa.

③ Group C – E above 5.6×10^3 mpa upto 9.8×10^3 mpa & fb above 8.5 mpa upto 12 mpa.

DEFECTS IN TIMBER:-

Any irregularity or dis order that occurs in a specimen of timber affecting its strength, durability and appearance is known as defect. The timber being a natural product, it is seldom free from defects. The defect in timber may be discussed under following two heads.

① natural defects ② other defects

① NATURAL DEFECTS:- The defects that are developed in growing tissues of living tree are called natural defects.

↳ knots and wanes are typical natural effects. A knot is formed when a portion of a branch or limb embedded in the body of tree is cut. ordinarily, the knots are composed of inferior wood than the normal wood having irregular or distorted grains. which reduce the strength of the timber. But wane is a bark or lack of wood on the edge or corner of a piece of timber. wane affects the strength of a timber by reducing the cross sectional area.

② OTHER DEFECTS:-

- Such defects develop because of the activity of external agencies during subsequent treatment of sawn timbers like faulty seasoning technique or defective method of storing etc.
- Leading to permanent distortion of timbers. This results in various types of warping and rupture of tissues. Warping includes cupping, twisting and bowing whereas rupture of tissues includes checks, shakes and splits.

GRADING OF TIMBER:-

- The method of designating the quality of a piece of timber is known as grading. While grading the timbers, defects in timbers are taken into consideration with respect to their size, numbers and location. Also the effect of density & slope of grains are taken into account while grading the timbers.
- The timbers is mainly graded into three categories namely, select timbers, standard timbers and common timbers.
- However, the cut sizes of structural timbers are graded after seasoning into three grades; select grade, grade I (standard) & grade II (ordinary).
- The selected timbers is either free from the defects or it has minimum defects. The standard timbers has defects within specified limits. The common timbers is inferior to standard timbers.
- The safe working stresses in compression, shear and bending for the selected timbers is 1.16 times the safe working stresses for the timbers of the standard grade. The safe working stresses for timbers of common grade in compression, shear and bending is 0.84 times the safe working stresses for timbers of standard grade.

Exam:- If the permissible stress in bending and compression parallel to the grains of a standard timber for inside location are 18.2 mpa and 12mpa respectively determine the corresponding stresses for the timber of select grade and common grade (grade II).

Solⁿ :- For Standard Grade

$f_b = 18.2$ mpa

$f_{cp} = 12.0$ mpa

For select grade = The values for standard grade are to be multiplied with 1.16.

$f_b = 18.2 \times 1.16 = 21.11$ mpa.

$f_{cp} = 12.0 \times 1.16 = 13.92$ mpa.

For common grade - The values for standard grade are to be multiplied with 0.84

$f_b = 18.2 \times 0.84 = 15.29$ mpa

$f_{cp} = 12.0 \times 0.84 = 10.08$ mpa.

Example:- The permissible stress for a grade I timber in compression perpendicular to grains for inside and out-side location are 6.0 mpa and 4.0 mpa respectively. If the slope of the grain is 1:12 and is likely to be subjected to wind and earth-quake loading, find the safe working stress.

Solⁿ :- Here the slope of the grain is more than 1:15 and the modification factor $K_1 = 0.9$

For wind and earthquake loading modification factor $K_2 = 1.33$

Safe working stress in compression perpendicular to grains = $f_{cr} \times K \times K_2$

f_{cm} (inside) $= 6.0 \times 0.9 \times 1.33 = 7.18$ mpa

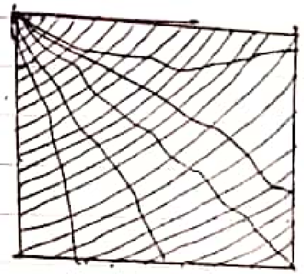
f_{cm} (outside) $= 4.0 \times 0.9 \times 1.33 = 4.79$ mpa

TIMBER COLUMNS :-

↳ The timber columns are structural members which primarily support load by including compressive stress along the grains.

↳ Solid wood columns consist of single pieces of wood & these may be of square, rectangular or circular in cross section. Built up columns, box columns and spaced columns are also used for more effective use of timber.

Solid columns :-



(a)

(b)

↳ Solid timber columns are the simplest to provide. The slenderness ratio of columns plays an important role in calculating the strength of solid wood columns.

Classification of solid columns :-

Solid columns shall be classified into short, intermediate and long columns depending upon their slenderness ratio (S/d) as follows.

(1) Short column - where S/d does not exceed 11.
(2) Intermediate column - where S/d is betn 11 and K_8.
(3) Long column - where S/d is greater than K_8.

where K_8 is a constant $= 0.584\sqrt{E/f_{cp}}$, E is the modulus of elasticity in bending in N/mm^2 and f_{cp} is the permissible stress in compression parallel to the grain in N/mm^2.

PERMISSIBLE COMPRESSIVE STRESSES IN COLUMNS :-

1. Short column - The permissible compressive stress sh be calculated as $f_c = f_{cp}$ (f_{cp} is as defined above).
2. Intermediate column - The permissible compressive stress is given by - $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{K_{9}d}\right)^4\right]$.
3. Long column - The permissible compressive stress is calculated by . $f_c = \frac{0.329E}{(s/d)^2}$

Classification of Box Column :-

The box column is classified into short, intermediate and long column as follows :-

1. Short column :- where $\frac{s}{\sqrt{d_1^2+d_2^2}}$ is less than 8.
2. Intermediate column :- where $\frac{s}{\sqrt{d_1^2+d_2^2}}$ is betn 8 and K_9.
3. Long column :- where $\frac{s}{\sqrt{d_1^2+d_2^2}}$ is greater than K_9.

where K_9 is a constant $= \frac{\pi}{2}\sqrt{\frac{U \times E}{5q\, f_{cp}}}$, U and q being constants for particular thickness of plank as given below :

t mm	U	q
25	0.80	1.00
50	0.60	1.00

PERMISSIBLE COMPRESSIVE STRESSES IN BOX COLUMNS:

The permissible compressive stresses are calculated by the following expressions :-

1. Short column $f_c = 2\,f_{cp}$

2. Intermediate column $f_c = 2\,f_{cp}\left[1-\frac{1}{3}\left(\frac{S}{K_8\sqrt{d_1^2+d_2^2}}\right)^4\right]$

3. Long column $= f_c = \cancel{0.329} \; \frac{0.329\,UE}{\left(\frac{S}{\sqrt{d_1^2+d_2^2}}\right)^2}$

(i) Unsupported length of column $S = 1.5m = 1500\,mm$

Least lateral dimension of the column $d = 150\,mm$

$\therefore$ max^m slenderness ratio $= \frac{S}{d} = \frac{1500}{150}$

$= 10 < 11 \Rightarrow$ Short column

Allowable stress $f_c = f_{cp} = 10.6$ mpa

Safe axial load = cross sectional area x allowable stress

$= (150 \times 200) \times 10.6 = 318000\,N = 318\,kN$

(ii) Unsupported length of the column $S = 2.8m = 2800\,mm$

Here $S/d = \frac{2800}{150} = 18.67 > 11$

$K_8 = 0.584\sqrt{\frac{E}{f_{cb}}} = 0.584\sqrt{\frac{12670}{10.6}} = 20.19$

$\therefore 11 < \frac{S}{d} (= 18.67) < K_8 = (= 20.19) \Rightarrow$ Intermediate column.

Allowable stress $f_c = f_{cp}\left[1-\frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

$= 10.6\left[1-\frac{1}{3}\left(\frac{2800}{20.19 \times 150}\right)^4\right]$

$= 8.02$ mpa

$\therefore$ Safe axial load $= A \times f_c = (150 \times 200) \times 8.02$

$= 240600\,N$

$= 240.6\,kN$

(iii) unsupported length of the column $s = 4.0m = 4000$ m
$\therefore s/d = \frac{4000}{150} = 26.67 > K_8 (= 20.19) \Rightarrow$ Long column

Allowable stress $f_c = \frac{0.392E}{(s/d)^2}$

$= \frac{0.392 \times 12670}{(26.67)^2}$

$= 5.86$ mpa

$\therefore$ Safe axial load $= A \times f_c = (150 \times 200) \times 5.86$

$= 175800N$

$= 175.800N$

Example:- Determine safe axial load on a circular column of 180mm dia made up of deodar (H.P) wood. unsupported length of the column is 3.0 m being situate in out site location.

Soln:- For deodar wood and outside location
Permissible compressive stress parallel to the grain
$f_{cp} = 6.9$ mpa
$E = 9180$ mpa

Sectional area of the circular column $A = \frac{\pi}{4} d^2$

$= \frac{\pi}{4}(180)^2$

$= 25447 \; mm^2$

Size of equivalent square column $d = \sqrt{25447}$

$= 159.52$ m.m

unsupported length of the column $s = 3000$ m.m

$\therefore \frac{s}{d} = \frac{3000}{159.52} = 18.81 > 11$

$K_8 = 0.584\sqrt{\frac{E}{F_{cb}}}$

$= 0.584\sqrt{\frac{9480}{6.9}}$

$= 21.65$

$\therefore 11 < \frac{S}{d}$ $(=18.81) < K_8 \Rightarrow$ Intermediate column

Allowable stress $F_c = F_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

$= 6.9\left[1 - \frac{1}{3}\left(\frac{3000}{21.65 \times 159.52}\right)^4\right]$

$= 5.39$ mpa

Safe axial load $= A \times F_c$

$= 26447 \times 5.39$

$= 142.28 \times 10^3$ N

$= 142.28$ kN

Example-2 Three planks 150mm X 50mm are glued together to form a built up column. If the timber used is babul. Find the maximum load it can carry. If the un supported length of column is 2.5 m. Given that $F_{cp} = 11.2$ mpa and $E = 10800$ mpa.

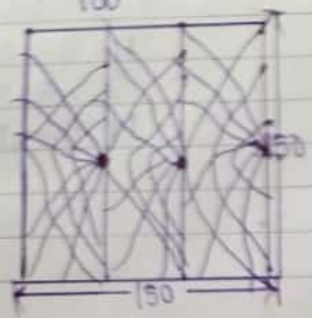
Sol:- unsupported length $S = 2500$ m.m

Slenderness ratio $\frac{S}{d} = \frac{2500}{150} = 16.67 > 11$

$K_8 = 0.584\sqrt{\frac{E}{F_{cb}}}$

$= 0.584\sqrt{\frac{10800}{11.2}}$

$= 18.13$



$\therefore 11 < \frac{l}{d} < 18.13 \Rightarrow$ intermediate column

Allowable stress $f_c = f_{cb}\left[1 - \frac{1}{3}\left(\frac{l}{kd}\right)^4\right]$

$$= 11.2\left[1 - \frac{1}{3}\left(\frac{2500}{18.13 \times 150}\right)^4\right]$$

$$= 8.536 \text{ mpa}$$

Safe axial load $= A \times f_c$

$= 150 \times 150 \times 8.536$

$= 172.06 \times 10^3 N$

$= 172.06 kN$

Example:- If four planks 160 mm × 40 mm are to be formed in the shape of a box, find the maximum load for the mango timber with unsupported length of 2.5 m and inside location.

Sol^n:- For mango timber

$E = 9120$ mpa.

$f_{cp} = 7.3$ mpa (for inside location)

$d_1 = 160+40 = 200$ mm - least overall width of box column.

$d_2 = 160-40 = 120$ mm = least overall dimension of core in box column.

Plank thickness $(t) = 40$ mm

$\therefore q = 1.0$

For $t = 25$ mm $\quad U = 0.8$

$t = 50$ mm $\quad U = 0.6$

By interpolation $U = 0.8 \left(\frac{0.8-0.6}{50-25}\right) \times (40-25)$

$= 0.8 - 0.12$

$= 0.68$

Here $\frac{l}{\sqrt{d_1^2+d_2^2}}$

$= \frac{2500}{\sqrt{200^2+120^2}}$

$= \frac{2500}{\sqrt{40,000+14,400}}$

$= 10.0078$

$K = \frac{\pi}{2}\sqrt{\frac{U \times E}{5 q f_{cp}}}$

$= \frac{\pi}{2}\sqrt{\frac{0.68 \times 9120}{5 \times 1.0 \times 7.3}}$

$= 20.48$

$\therefore 8 < \frac{l}{\sqrt{d_1^2+d_2^2}} < K \Rightarrow$ intermediate column

Allowable stress $f_c = q f_{cp}\left[1-\frac{1}{3}\left(\frac{l}{K\sqrt{d_1^2+d_2^2}}\right)^4\right]$

$= 1 \times 7.3\left[1-\frac{1}{3}\left(\frac{15}{20.48}\right)^4\right]$

$= 6.6$ mpa

Safe load $= A \times f_c = (200^2-120^2) \times 6.6$

$= 168.96 \times 10^3$ N

$= 168.96$ kN.

DESIGN OF BEAM

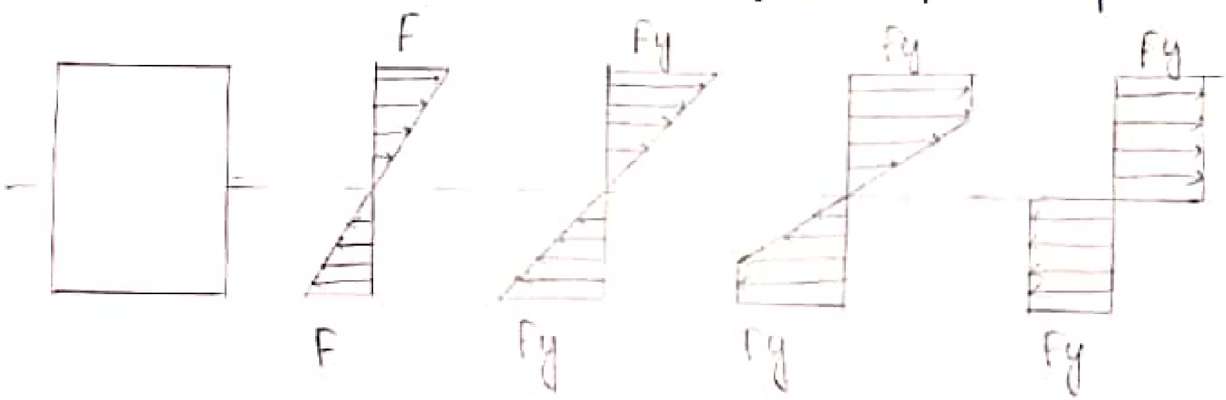
Beam is a structural member with length considerably larger than cross sectional dimensions subjected to lateral loads which give rise to bending moment, shear force in the member.

Based on the lateral supports to compression flange there are mainly two types of beams.

a. Laterally support Beam
b. Laterally unsupport Beam.

PLASTIC MOMENT CARRYING CAPACITY OF A SECTION :-

We know stress is proportional to strain which elastic limit. As the load is gradually increased stressess increase proportionally till extreme fibre stress subjected to yield stress. But interior fibres are not yielded. and



Hence additional loads are resisted by unyielded portion. Then the load is applied gradually when all fibres at a section yield is called formation of plastic hinge.

Let the area of the section in compression be A_c. in tension A_t and total area A.

Equating the horizontal force for equilibrium we get,

$$A_c \cdot f_y = A_t f_y$$

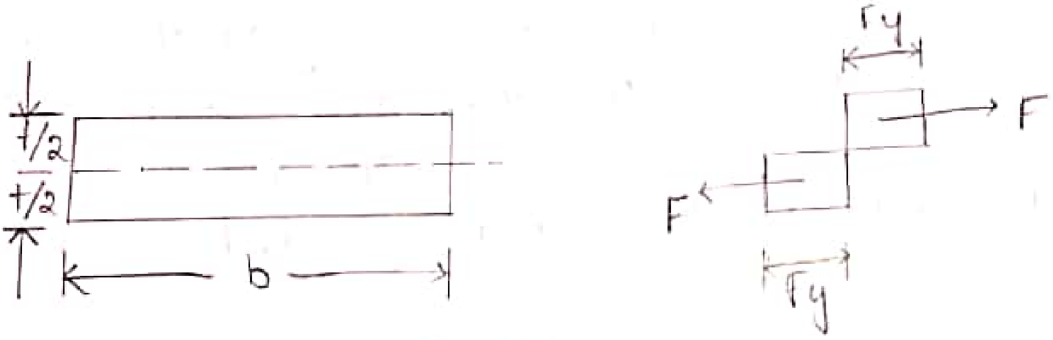
$$\therefore A_c = A_t = \frac{A}{2}$$

Determine the plastic moment capacity and plastic section modulus of.

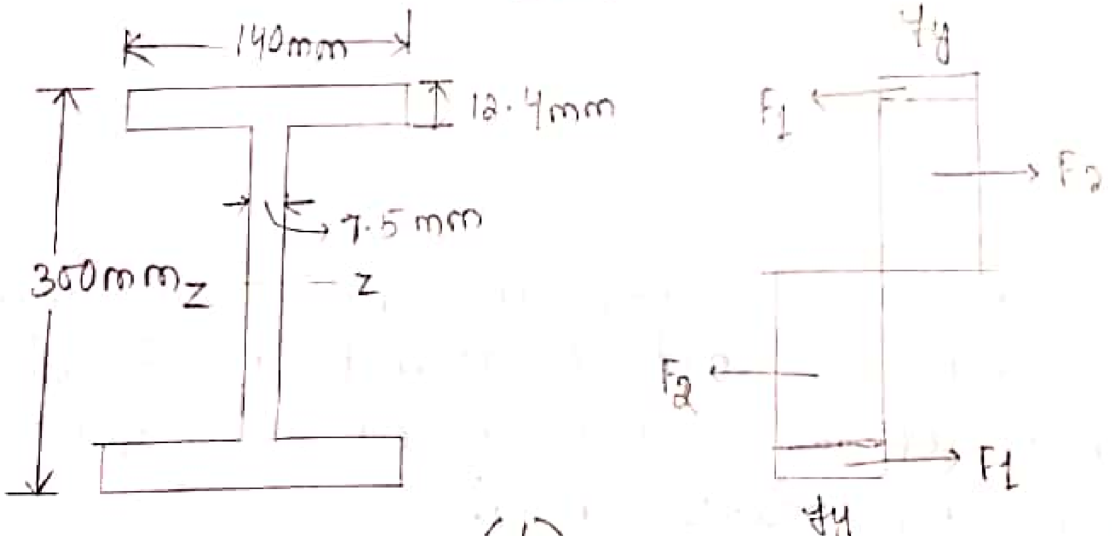
a. The rectangular section of size b×t about z-z axis shown in fig. (a).

b. The I-section about z-z axis as shown in fig (b).

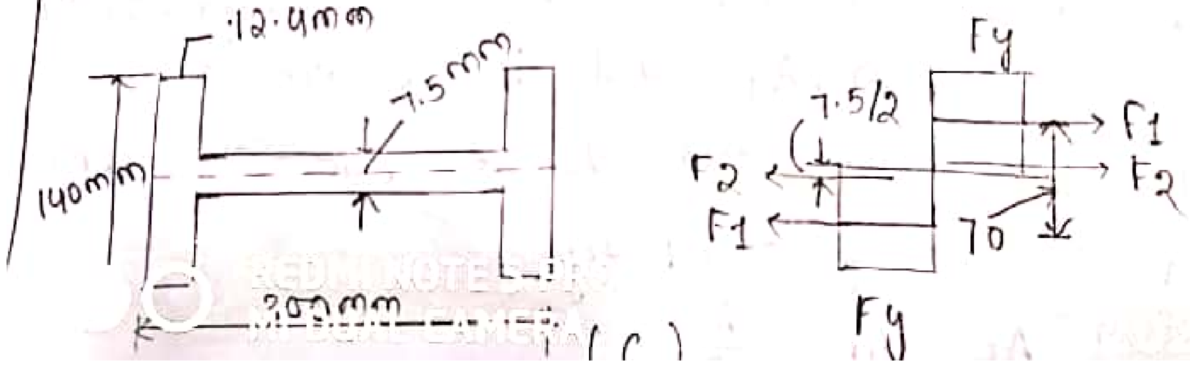
c. The I-section about y-y axis as shown in fig (c).



(a)



(b)



(c)

a. RECTANGULAL SECTION :—

Due to symmetry plastic neutral axis (axis of equal area) is at mid depth.

$$\therefore A_t = A_c = \frac{b}{2} \times t$$

$$F = \frac{b}{2} t \, F_y$$

The distance between tension and compressive force $= \frac{t}{2}$

$$\therefore M_p = F \cdot \times \frac{t}{2}$$

$$= \frac{b}{2} t \, F_y \frac{t}{2}.$$

$$= \frac{1}{4} bt^2 \, f_y$$

$$\therefore Z_p = \frac{M_p}{F_y} = \frac{1}{4} bt^2.$$

b. I-SECTION - ABOUT Z-Z AXIS :—

Plastic N-A is at mid depth. When plastic hinge is formed

Force in flanges,

$$F_1 = 140 \times 12.4 \times F_y$$

Force in Webs

$$F_2 = \frac{1}{2}(150 - 12.4) \times 7.5 \, F_y$$

Distance between F_1 force $= 300 - 12.4 = 287.6$ mm

Distance between F_2 force $= 150 - 12.4 = 137.6$ mm

$\therefore M_p = F_1 \times 287.6 + F_2 \times 137.6$

$= 140 \times 12.4\ F_y \times 287.6 + \frac{1}{2} \times 137.6 \times 7.5\ F_y \times 137.6$

$= 499274\ F_y + 71001.6\ F_y$

$= 570275.6\ F_y$

$\therefore Z_p = 570.276 \times 10^3\ mm^3$.

C. I-SECTION - ABOUT Y-Y AXIS :—

Plastic N-A is in mid depth. Let F1 be force in flange of size 140 x 12.4 mm and F_2 be force in ~~web~~ web of size (300 − 2 x 12.4) x 7.5 mm. Then

$F_1 = 140 \times 12.4\ F_y$

$F_2 = 275.2 \times 7.5\ F_y$

Distance between F1 force $= \frac{140}{2} = 70mm$

Distance between F_2 Force $= \frac{7.5}{2} = 3.75mm$

$\therefore M_p = 140 \times 12.4 \times F_y \times 70 + 275.2 \times 7.5 F_y \times 3.75$

$= 121520 F_y + 7740 F_y$.

$= 129260\ F_y$

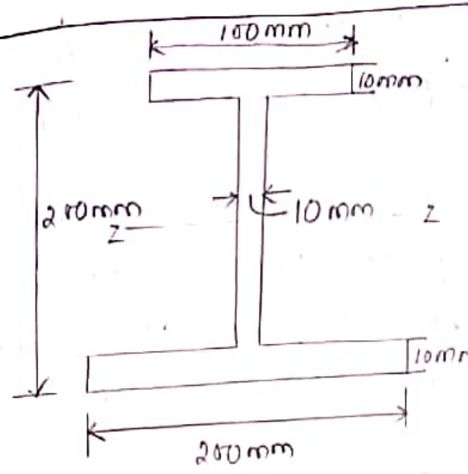
$\therefore Z_p = \frac{M_p}{F_y} = 129260 mm^2$

Note :—

Contribution of flange is $\frac{121520}{129260} \times 100 = 94\%$.

EXAMPLE :— 1

Determine the plastic moment capacity and plastic modulus of section of the unsymmetric section shown in figure.



Solution :—

Total area $= 100 \times 10 + 200 \times 10 + (200 - 20) \times 10 = 4800 mm^2$

$$A_c = A_t = \frac{4800}{2} = 2400 mm^2$$

Plastic N.A is at a depth 'h' from top fibre where h is given by

$$100 \times 10 + (h - 10) \times 10 = 2400$$

$$\therefore h = 150 mm$$

When plastic hinge is formed, one half is subjected to compressive stresses fy and another half to tensile stresses Fy. Taking moment of all such forces about plastic NA, we get.

$$M_p : \left[100 \times 10 \times (150 - 5) + 10 \times (150 - 10) \frac{(150 - 10)}{2} + 10 \times (50 - 10) \cdot \frac{(50 - 10)}{2} + 200 + 10 (50 - 5) \right] F_y$$

$= 341000 \, f_y \, mm^2$ (Ans)

$$\therefore Z_p = \frac{M_p}{f_y} = 341000 \, mm^2$$

(Ans).

CLASSIFICATION OF CROSS-SECTION :—

On the basis of IS 800 : 2007 ; Various cross sections are :—

a. Class - 1 (Plastic) cross-section.
b. Class - 2 (Compact) cross-section.
c. Class - 3 (Semi-Compact) cross-section.
d. Class - 4 (Slender) cross-section.

DESIGN PROCEDURE :—

1. A trial section is selected assuming it is going to be plastic section (class 1 section).
2. Then it is checked for the class it belongs.
3. Check for bending strength.
4. Check for shear strength
5. Check for the deflection.

If any check fails the section is revised.

BENDING STRENGTH OF A LATERALLY SUPPORTED BEAM :—

If $\frac{d}{t_w} \leq 67\varepsilon$ IS 800 : 2007 considers two cases one with design shear strength less than $0.6 V_d$ and other with design shear strength more than $0.6 V_d$ where V_d is design shear. When $\frac{d}{t_w} > 67\varepsilon$ shear buckling of web is likely to take place for such case.

a. If $V \leq 0.6 V_d$:

The design bending strength M_d shall be taken as :-

$$M_d = \beta_d Z_p f_y \times \frac{1}{\gamma_{m0}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}}$$ for simply supported beam.

$\leq 1.5 Z_e \frac{f_y}{\gamma_{m0}}$ for cantilever beam

Where,

$\beta_b = 1.0$ for plastic and compact section.

$= \frac{Z_e}{Z_p}$ for semi - compact sections.

Z_p, Z_e = plastic and electric section modulii of the cross-section, respectevely.

b. If $V > 0.6 V_d$

In such cases,

$$M_d = M_{dv}.$$

Where M_{dv} is design bending strength under high shear. This reduced, value is recommended to account for the effect of higher shear on the bending strength of the section, M_{dv} is to be calculated as given below.

a. Plastic or Compact section :-

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e \times F_y \times \frac{1}{\gamma_{m0}}$$

where,

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

M_d: Plastic design moment of the whole section.

V : Factored applied shear force.

V_d : Design shear strength.

M_{fd} = Plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{m0} (for finding shear area).

b. **Semi-compact section :—**

$$M_{dv} = \frac{Z_e F_y}{\gamma_{m0}}$$

SHEAR STRENGTH OF A LATERALLY SUPPORED BEAM:—

The design shear strength of a section is given by :—

$$V_d = \frac{A_v F_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{m0}}$$

where,

A_v = Shear area

F_{yw} = Yield strength of wed.

Deflection check :—

The deflection should be calculated by elasti theory for working load. The maximum deflection in beam should not exceed limit specified.

EXAMPLE :— 2

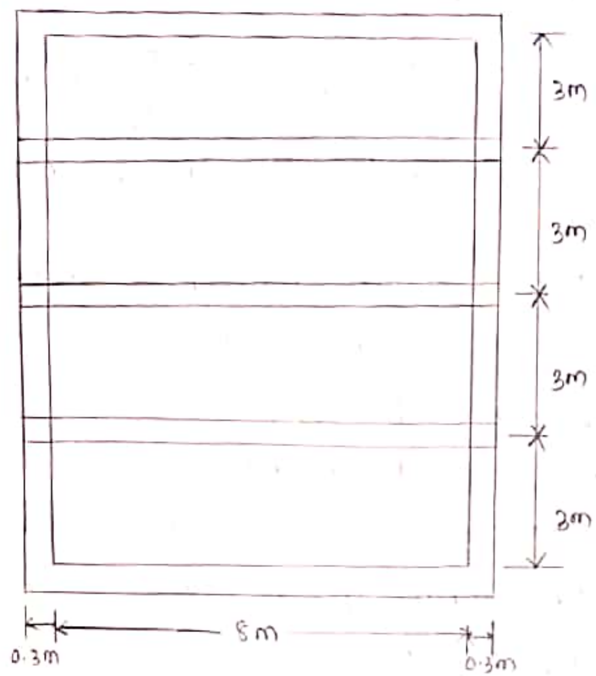
A roof of a hall measuring 8m × 12m consists of 100mm thick R.C slab supported on steel I. bear spaced 3m apart as shown in figure. The finishi load may be taken as 1.5 kN/m² and live lo as 1.5 kN/m². Design the steel beam.

Solution :—

Each beam has a clear span of 8m and ta care of 3m width of slab. Hence the load per meter length of the beam is as follow

weigth of R.C slab = $0.1 \times 1 \times 3 \times 25 = 7.5$ kN/m

finishing Load = $1.5 \times 3 = 4.5$ kN/m.



Self weight (assumed) = 0.8 kN/m .

∴ Total dead Load = 12.8 kN/m .

Live load = 1×3×1.5 = 4.5 kN/m .

∴ ~~Factored Live load = 1.5×4.5 = 6.75 kN/m~~ .

∴ factored dead load = 1.5 × 12.8 = 19.2 kN/m .

Factored live load = 1.5 × 4.5 = 6.75 kN/m .

∴ Total factored load = 25.95 kN/m .

Effective span of the simply supported beam = centre to centre distance of supports.

Assuming width of support = 0.3m .

Effective span = 8 + 0.3 = 8.3m .

$$\therefore \text{Design moment, } M = \frac{wl^2}{8}$$

$$= \frac{25.95 \times 8.3^2}{8} = 223.46 \text{ kN.m}$$

Design shear force $V = \frac{25.95 \times 8.3}{2} = 107.69$ kN.

$\therefore$ Section modulus required $= \frac{M}{f_y} \times \gamma_{m0}$.

$$Z_p = \frac{223.46 \times 10^6 \times 1.1}{250} = 983224 \text{ mm}^2.$$

Try ISMB 400 which has $Z_p = \cancel{H6}\,1176.163 \times 10^3$ mm³

The properties of the section are as follows:

width of flange $b = 140$mm

Depth of section $h = 400$mm

sectional area $A = 7845.58 \text{mm}^2$.

Thickness of flange $t_f = 16.0$mm.

thickness of web $t_w = 8.9$mm.

Depth of web $d = h - 2(t/2)$

$= 400 - 2(32.8) = 333.4$ mm.

moment of inertia about z-z axis.

$I_{zz} = 20458.4 \times 10^4 \text{ mm}^4$.

Elastic section modulus $Z_e = 1022.7 \times 10^3 \text{ mm}^4$.

Outstanding leg of comp. flange, $b = \frac{140}{2} = 70$

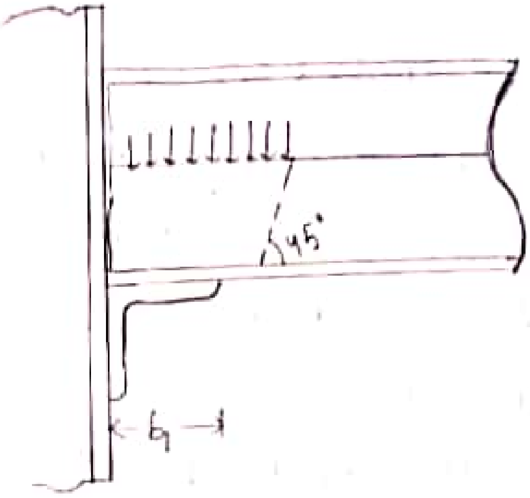
WEB BUCKLING STRENGTH :—

Certain portion of beam as supports act as column to transfers the load from beam to the support. Hence under this compressive force the web may be buckle.

So check for buckling is done as per IS 800

$F_{cdw} = (b_1 + n_1)\, t_w\, f_c$

F_c can be calculated by interpolation by using table 9 of IS 800



Slenderness ratio $= 2.5 \frac{d}{t_w}$

$d = D - 2(t_f + r)$

where,
r = radius at root

WEB CRIPPLING STRENGTH :—

$F_w = (b_1 + n_2) \cdot \frac{f_{yw}}{\gamma_{m0}}$

n_2 : Length obtained by dispersion through flange to web junction 1:2.5

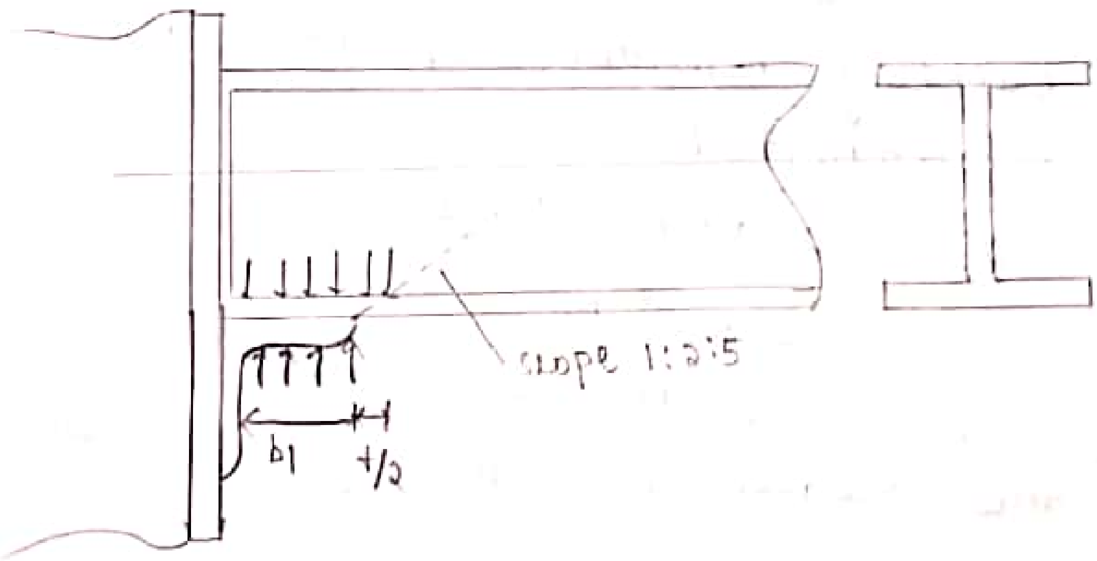
$n_2 = 2.5(t_f + r)$

where, r = radius at root

In the design F_w > load transferred by bearing.

EXAMPLE :— 3

Check the section in problem -2 for web buckling & web crippling if $b_1 = 75mm$.



Solution :—

Section selected was ISMB 400

End reaction = End shear = 107.61 kN

Shift bearing at ends = 75mm.

From steel table,

$t_w = 8.9mm$

$t_f = 16.0mm$.

radius at root = 14.0mm.

Depth of section $h = 400mm$.

$\therefore$ Depth of web $= h_1 = 334.2$.

Check for web buckling :

Slenderness ratio.

$$\lambda = 2.5\frac{h_1}{t_w}$$

$$= \frac{2.5 \times 334.2}{8.9} = 93.88$$

Since, cross-section of web is rectangle, it falls under buckling class C.

$$F_c = 121 - \frac{3.88}{10}(121 - 107) = 115.568\ N/mm^2.$$

$$n_1 = \frac{400}{2} = 200mm.$$

$\therefore$ web buckling resistance of the section.

$$F_{cdw} = (b_1 + n_1)\, t_w\, F_c$$

$$= (75 + 200) \times 8.9 \times 115.568$$

$$= 282.852 \times 10^3\ N.$$

$$= 282.852\ kN > 107.61\ kN$$

Hence the section is safe against web buckling.

Check for web crippling :—

Flange thickness $= 16.0$.

radius of root $= 14.0$.

$\therefore n_2 = 2.5\,(t/2) = 2.5 \times 32.8$

$= 82mm$.

$\therefore$ Strength of web against web crippling.

$$F_w = (b_1 + n_1)\, t_w\, F_{yw} \times \frac{1}{\gamma_{m0}}.$$

$$= (75 + 82)\, 8.9 \times 250 \times \frac{1}{1.1}$$

$$= 317.568 \times 10^3\ N.$$

$= 317.568\ kN >$ Load transferred by bearing in this case $(107.61\ kN)$.

Tubular Steel Structure.

Tubular steel structures are used in truss members, scaffolding of building, stadium, exhibition halls, transmission towers.

Codes Required:- IS 1161.1998, IS 806.1968.

Advantages:-

(i) These have small self weights. Also because of direct connections, gusset plates are eliminated further reducing dead loads.

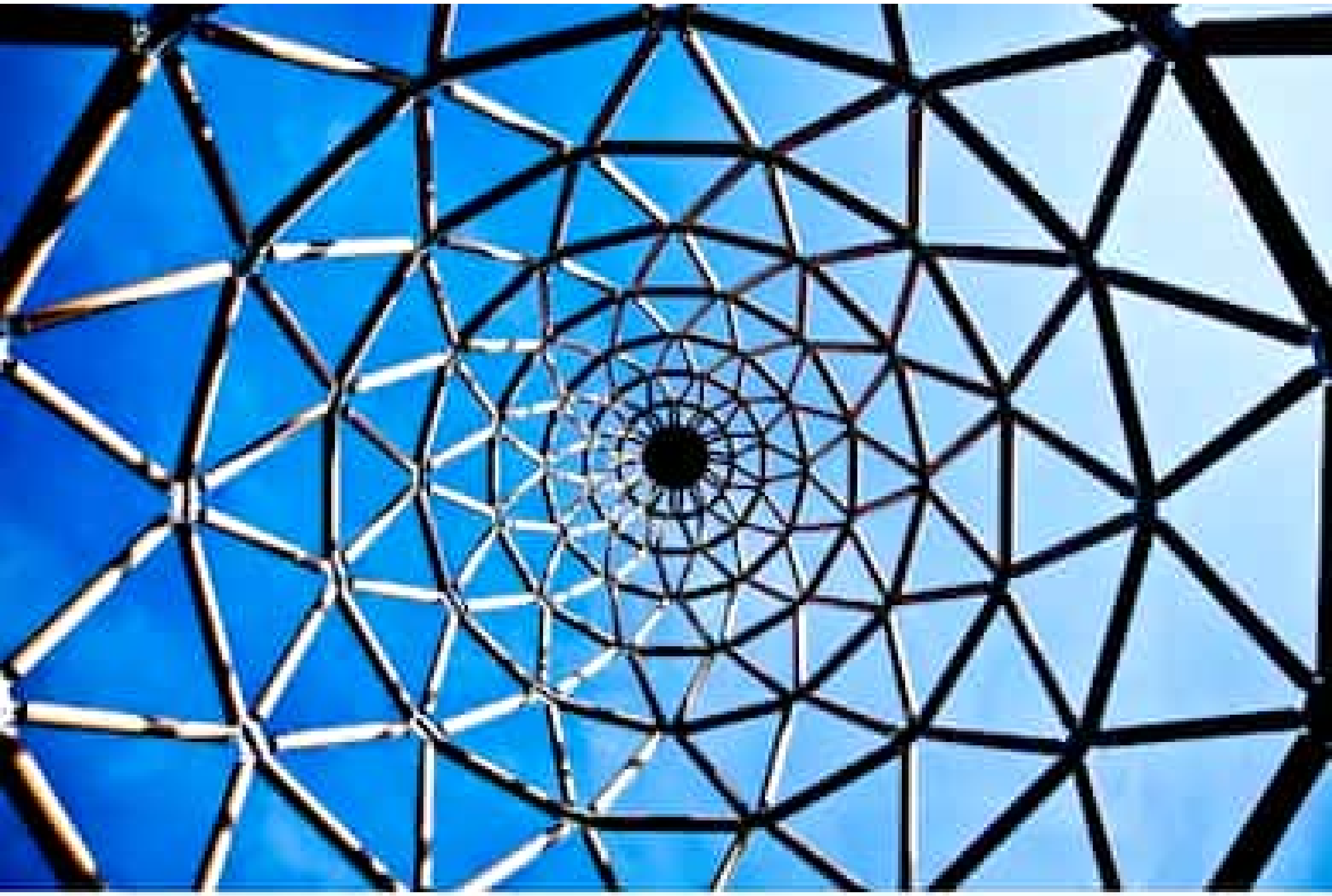
(2) Torsional strength of these structures is more than any other rolled section.

(3) For the same load, the surface area of a tube is about 60 to 70% of that for other rolled sections. Because of less area economy is achieved in maintenance, painting & fire proofing.

(4) Due to smooth finished surface, dirt & moisture do not collect over the surface, reducing the possibility of corrosion.

(5) ~~Due to~~. The change in load with the floor levels can be accomodated by varying the tube thickness & the external tube dimension may be maintained.

(6) The internal hollow space of tubular columns may be used for carrying drain pipes, wires, cable etc. Also these spaces may



be filled with concrete to increase the load carrying capacity & to improve fire resistance.

Disadvantage :-

(1) They pose difficulty in connection among themselves or to any plate element due to their shape problems.

(2) Bolting & riveting on those sections are not convenient.

(3) Their light weight some times become responsible for the structural instability.

(4) Highly skilled man power & special welding techniques are required for their connection.

Designation of steel tubes :-

Steel tubes are designated by their nominal bore size.

classification of steel tube. →

→ based on wall thickness
- light
- medium
- heavy.

→ based on yield stress
- YSt 22 (YSt 210) ↳ Mpa
- YSt 25 → YSt 240 ↳ Mpa
- YSt 32. → YSt 320 ↳ Mpa

YS → Yield stress

t → tube.

22, 25, 32 are on kgf/mm 2,

Note

(i)* 1 Mpa = 1 N/mm^2 = 0.102 kgf/mm^2

Tensile properties of steel tubes for Structural purposes:-

(table-2, clause-11.2.1., IS code 1161).

Grade	tensile strength (Min)	Yield stress (Min).
Yst 210	330	210
Yst 240	410	240
Yst 310	450	310.

* The standard sizes of tubular sections, their mass/weight, relevant geometrical properties are given in table 1 as per IS:1161:1998.

Behaviour of tubular Sections:-

(1) compression member:-

depends upon slenderness ratio.

slenderness ratio $\lambda = \frac{l_{eff.}}{r}$

$L_{effective}$ → depends of end conditions.

$= kL$ (table-7, cl:-6.4, IS 806

r = we will get from section properties given is IS 1161:1998.

maximum limit of slenderness ratio :-

type of member.	$\lambda = \frac{l}{r}$
(a) carrying loads resulting from dead load / super-imposed load.	180
(b) carrying loads resulting from wind or seismic forces only provided the deformation of such members does not adversely, affect the stress in any part of the structure.	250
(c) Normally acting as a tie in a roof truss but subject to possible reversal or stress resulting from the action of wind.	350.

Axial stress in compression :- The direct stress in compression on the cross-sectional area of axially loaded steel tubes shall not exceed the value of Fc given in table -(2) of IS 806:1968.

Permissible axial stress in compression $\rightarrow$ (fc).
(table 2, clause 5.2)
(IS 806)

the cross-sectional area of axially loaded steel tubes shall not exceed the values of F_c given in Table 2 in which l/r is equal to the effective length of the member divided by the radius of gyration.

*Specification for steel tubes for structural purposes (*revised*) (*Second revision* in 1968).

†Specification for covered electrodes for metal arc welding of mild steel (*revised*) (Third revision in 1970).

‡Code of practice for structural safety of buildings: Loading standards (*revised*).

TABLE 2 PERMISSIBLE AXIAL STRESS IN COMPRESSION

(*Clause* 5.2)

l/r	F_c		
	Grade YSt 22 kgf/cm²	Grade YSt 25 kgf/cm²	Grade YSt 32 kgf/cm²
(1)	(2)	(3)	(4)
0	1 250	1 500	1 900
10	1 217	1 448	1 821
20	1 175	1 400	1 760
30	1 131	1 352	1 679
40	1 088	1 303	1 610
50	1 046	1 255	1 539
60	1 002	1 207	1 468
70	970	1 155	1 375
80	929	1 088	1 263
90	876	1 003	1 128
100	814	910	989
110	745	813	869
120	674	721	758
130	603	638	665
140	540	565	584
150	490	503	517
160	432	443	450
170	381	392	396
180	339	348	353
190	304	311	315
200	271	278	280
210	243	249	250
220	219	225	227
230	198	204	205
240	180	185	187
250	162	167	167
300	106	106	106
350	71	71	72

Note 1 — Intermediate values may be obtained by linear interpolation.

Note 2 — The formula, from which these values have been derived, is given in Appendix A.

5.3 Bending Stresses — In tubes, the tensile bending stress and the compressive bending stress in the extreme fibres shall not exceed the values of F_b given in Table 3.

TABLE 3 PERMISSIBLE BENDING STRESS IN EXTREME FIBRES IN

6.4 Compression Members

6.4.1 *Effective Length of Compression Members* — Effective length (l) of a compression member for the purpose of determining allowable axial stresses shall be assumed in accordance with Table 7, where L is the actual length of the strut, measured between the centres of lateral supports. In the case of a compression member provided with a cap or base, the point of lateral support at the end shall be assumed to be in the plane of the top of the cap or bottom of the base.

TABLE 7 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

Type	Effective Length
Effectively held in position and restrained in direction at both ends	0·67 L
Effectively held in position at both ends and restrained in direction at one end	0·85 L
Effectively held in position at both ends but not restrained in direction	L
Effectively held in position and restrained in direction at one end, and at the other end effectively restrained in direction but not held in position	L
Effectively held in position and restrained in direction at one end, and at the other end partially restrained in direction but not held in position	1·5 L
Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end	2·0 L

Table 1 Sizes and Properties of Steel Tubes for Structural Purposes

(*Clauses* 3.1, 6.1, 6.1.1 *and* 6.1.2)

Nominal Bore	Outside Diameter	Class	Thickness	Weight	Area of Cross Section	Internal Volume	Surface		Moment of Inertia	Modulus of Section	Radius of Gyration	Square of Radius of Gyration
							External	Internal				
mm	mm		mm	kg/m	cm^2	cm^3/m	cm^2/m	cm^2/m	cm^4	cm^3	cm	cm^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
15	21.3	Light	2.0	0.947	1.21	235		543	0.57	0.54	0.69	0.47
		Medium	2.6	1.21	1.53	203	669	506	0.69	0.64	0.66	0.44
		Heavy	3.2	1.44	1.82	174		468	0.75	0.70	0.55	0.42
20	26.9	Light	2.3	1.38	1.78	390		700	1.36	1.01	0.87	0.76
		Medium	2.6	1.56	1.98	370	845	681	1.48	1.10	0.86	0.74
		Heavy	3.2	1.87	2.38	330		644	1.70	1.26	0.84	0.71
25	33.7	Light	2.6	1.98	2.54	638		895	3.09	1.83	1.10	1.21
		Medium	3.2	2.41	3.06	585	1 059	857	3.61	2.14	1.08	1.17
		Heavy	4.0	2.93	3.73	518		807	4.19	2.48	1.05	1.11
32	42.4	Light	2.6	2.54	3.25	1 086		1 168	6.47	3.05	1.41	1.98
		Medium	3.2	3.10	3.94	1 017	1 332	1 130	7.62	3.59	1.39	1.93
		Heavy	4.0	3.79	4.82	929		1 080	8.99	4.24	1.36	1.86
40	48.3	Light	2.9	3.23	4.13	1 418		1 335	10.70	4.43	1.61	2.59
		Medium	3.2	3.56	4.53	1 378	1 517	1 316	11.59	4.80	1.59	2.54
		Heavy	4.0	4.37	5.56	1 275		1 265	13.77	5.70	1.57	2.47
50	60.3	Light	2.9	4.08	5.23	2 332		1 711	21.59	7.16	2.03	4.13
		Medium	3.6	5.03	6.41	2 213		1 667	25.88	8.58	2.00	4.02
		Heavy	4.5	6.19	7.88	2 066		1 611	30.90	10.2	1.98	3.92
65	76.1	Light	3.2	5.71	7.32	3 814		2 189	48.79	12.82	2.58	6.66
		Medium	3.6	6.42	8.20	3 727	2 391	2 163	54.02	14.20	2.57	6.60
		Heavy	4.5	7.93	10.1	3 534		2 107	65.12	17.1	2.54	6.43
80	88.9	Light	3.2	6.72	8.61	5 343		2 591	79.23	17.82	3.03	9.19
		Medium	4.0	8.36	10.7	5 138	2 793	2 540	96.36	21.68	3.00	9.00
		Heavy	4.8	9.90	12.7	4 936		2 490	112.52	25.31	2.98	8.88
90	101.6	Light	3.6	8.70	11.1	6 995		2 964	133.27	26.23	3.47	12.03
		Medium	4.0	9.63	12.3	6 877	3 192	2 939	146.32	28.80	3.45	11.91
		Heavy	4.8	11.5	14.6	6 644		2 889	171.44	33.75	3.43	11.76
100	114.3	Light	3.6	9.75	12.5	9 004		3 363	192.03	33.60	3.92	15.36
		Medium	4.5	12.2	15.5	8 704	3 591	3 306	234.3	41.0	3.89	15.10
		Heavy	5.4	14.5	18.5	8 409		3 250	274.5	48.0	3.85	14.86
110	127.0	Light	4.5	13.6	17.3	10 930		3 705	325.3	51.2	4.33	18.78
		Medium	4.8	14.5	18.4	10 819	3 990	3 686	344.58	54.27	4.32	18.69
		Heavy	5.4	16.2	20.6	10 599		3 649	382.0	60.2	4.30	18.52
125	139.7	Light	4.5	15.0	19.1	13 410		4 104	437.2	62.6	4.78	22.87
		Medium	4.8	15.9	20.3	13 287	4 389	4 085	463.44	66.35	4.77	22.76
		Heavy	5.4	17.9	22.8	13 043		4 047	514.5	73.7	4.75	22.58
135	152.4	Light	4.5	16.4	20.9	16 142		4 503	572.2	75.1	5.23	27.37
		Medium	4.8	17.5	22.2	16 008	4 788	4 484	606.92	79.65	5.22	27.25
		Heavy	5.4	19.6	25.0	15 740		4 446	674.5	88.5	5.20	27.05
150	165.1	Light	4.5	17.8	22.7	19 128		4 902	732.6	88.7	5.68	32.27
		Medium	4.8	18.9	24.2	18 981	5 187	4 883	777.32	94.16	5.67	32.14
		Heavy	5.4	21.3	27.1	18 690		4 845	864.7	105.0	5.65	31.92
150	168.3	Light	4.5	18.2	23.1	19 921		5 002	777.2	92.4	5.79	33.56
		Medium	4.8	19.4	24.7	19 771	5 287	4 983	824.78	98.01	5.78	33.42
		Heavy 1	5.4	21.7	27.6	19 473		4 946	917.7	109.0	5.76	33.21
		Heavy 2	6.3	25.2	32.0	19 030		4 889	1 053	125.0	5.73	32.85
175	193.7	Light	4.8	22.4	28.5	26 606		5 781	1 271.71	131.31	6.68	44.63
		Medium	5.4	25.1	32.0	26 260	6 085	5 743	1 417	146	6.66	44.36
		Heavy	5.9	27.3	34.8	25 974		5 712	1 535.2	158.65	6.64	41.11
200	219.1	Light	4.8	25.4	32.3	34 454		6 578	1 856.51	169.47	7.58	57.45
		Medium	5.6	29.5	37.5	33 930	6 883	6 528	2 141	195	7.55	57.02
		Heavy	5.9	31.0	39.5	33 734		6 509	2 247	205	7.54	56.86
225	244.5	Heavy	5.9	34.7	44.2	42 507	7 681	7 307	3 149	258	8.44	71.21
250	273.0	Heavy	5.9	38.9	49.5	53 557	8 578	8 202	4 412	323	9.45	89.30
300	323.9	Heavy	6.3	49.3	62.8	76 073	10 177	9 775	7 992	493	11.2	125.44
350	355.6	Heavy	8.0	68.6	87.3	90 533	11 173	10 663	13 111	737	12.3	151.29

Q-1 A tubular steel column of 4.8m length is hinged at both ends. It has nominal dia of 225mm & of Yst 25 grade. Determine the safe load carrying capacity of the column.

solution :-

step-1 Given data & section properties:-

thickness = 5.9 mm, weight = 34.7 kg/m.

Area of cross section = 44.2 cm^2

radius of gyration (r) = 8.44 cm.

nominal dia = 225mm. Yst = 25

outside dia = 244.5mm.

length of column (L) = 4.8m.

as both the ends are hinged, effective length (le) = KL

= 1XL = 4.8m.

(table-7, IS806).

Step(2)

calculation of slenderness ratio :-

$\lambda = \frac{l_{eff}}{r}$, $\lambda = \frac{4.8 \times 10^3}{8.44 \times 10} = 56.87 < 180$ (Cl-6.4.2) (IS 806) (ok)

Step-(3) permissible stress in compression.

(for Yst = 25) f_c (kgf/cm^2) (from table-2) (IS 806)

$\lambda_1 = 50 \longrightarrow 1255$

$\lambda = 56.87 \longrightarrow ?$

$\lambda_2 = 60 \longrightarrow 1207$

by interpolating we will get value of permissible compressive stress.

$$\frac{56.87-60}{50-60} \times (1255-1207) + 1207$$

$\Rightarrow f_c = 1222.024\ kg/cm^2$ $\quad$ ($1\ kgf = 9.81\ N \simeq 10\ N$)

$= \cancel{1222.2024\ N/cm^2} =$ $12220.24\ N/cm^2$ $\quad$ $1\ cm^2 = 100\ mm^2$

Safe load carrying capacity :-

$A \times f_c$

$P = 44.2 \times \cancel{1222.2024}.\ 12220.24$

$= \cancel{5401.35\ N}$

$= 540{,}134\ N \simeq 540.134\ kN.$

Assignment :-

Q-2) A tubular column consists of IS 1161 grad Yst 32 steel. The column is hinged in both the ends. The outside diameter of tube is 219.1mm. The weight of 1m length of tube is 310N. The length of column is 9.5m. Determine the safe load carrying capacity of column.

Minimum thickness of material:

For tubes painted with one prime coat of red oxides then periodically painted, the thickness should not be less than:-

(i) For construction exposed to weather -> ~~3.2 mm~~ 4mm

(ii) For construction not exposed to weather -> 3.2 mm.

(iii) For members not readily accessible for maintenance = 5mm.

For tubes painted with one coat of zinc primer followed by two coats of paint, the thickness should not be less than:-

(i) For construction exposed to weather = 3.2 mm

(ii) For construction not exposed to weather = 2.6 mm.

Permissible axial stress in tension (table-1, IS 806)

Grade	Ft (kgf/cm²)
Yst 22	1250
Yst 25	1500
Yst 32	1900

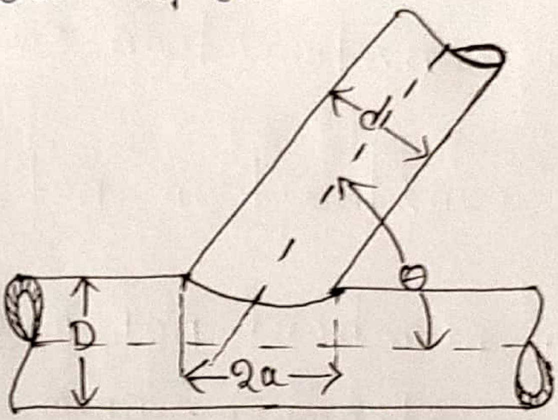
permissible stresses as per IS 806 (table-3, 4, 5)

Grade	Permissible bending stress (fb) (kgf/cm²)	Permissible maxm shear stress (Fs)	permissible maximum bearing stress (Fp)
Yst 22	1400	900	1700
Yst 25	1655	1100	1900
Yst 32	2050	1350	2500

Connection:- cl-5.7.2, appendix B, IS 806).

Determination of the Length of the curve of intersection of a tube with another tube or with a flat plate:-

The Length of the curve of intersection may be taken as:-



$P = a + b + 3\sqrt{a^2 + b^2}$

$a = \frac{d}{2} \operatorname{cosec} \theta$

$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$

↳ for intersection with a tube.

$= \frac{d}{2}$ for intersection with a flat plate.

d = outside diameter of branch. tube

θ = angle between branch & main tube

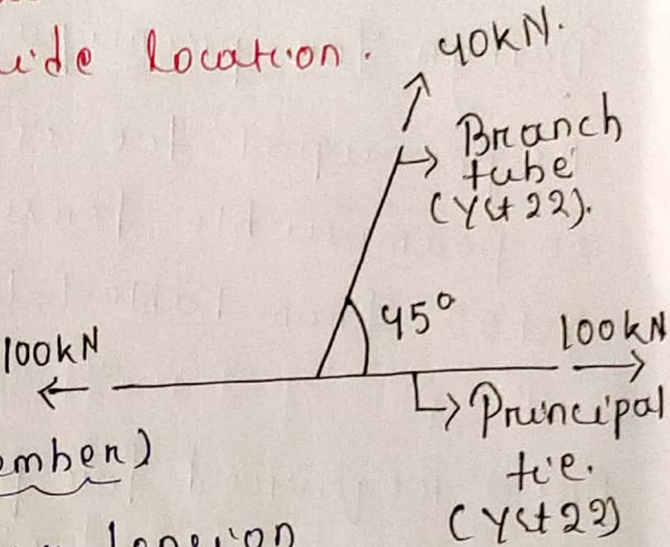
D = outside diameter of main tube.

Permissible stresses in welds:- (cl:- 5.7).

* For butt weld, the allowable tensile, compressive & shear stresses shall not exceed the stresses respectively permissible in the stresses respectively permissible in Yst 25 tubes or in the parent metal, whichever is less.
* In a fillet weld or in a fillet butt weld, the permissible stress shall not exceed the permissible in Yst25 tubes the shear stress permissible in Yst25 tubes or in the parent metal, whichever is less.

Scanned with CamScanner

Q A tension member carrying a force of 40kN meets the principal tie of tubular truss at an angle of 45°. If the force in principal tie is 100kN, Design the members & welded joint between the two tube. Use Yst 22 grade of steel. Assume these member are placed around inside location.



Design of Branch tube (40kN member)

As the branch tube is a tension member, so permissible tensile stress for Yst 22 (f_t) = 1250 kgf/cm² (table-1 IS 806)

$= 125\ N/mm^2$

→ Load coming on the branch tube (P) = $40kN = 40 \times 10^3 N$.

→ Area of the branch tube $= \frac{P}{f_t} = \frac{40 \times 10^3}{125}$

$= 320 mm^2$

let choose cross-sectional area = 373 mm²

so let us provide steel tube of nominal bore size 25mm (class heavy) & outside diameter of 33.7mm & area of cross section 373mm².

check the minimum thickness from durability consideration ->

We provide thickness of member = 4mm > 3.2mm.

(as this member is located inside, minimum thickness should be 3.2 mm as per cl 6.3.1).

Design for principal tie (100kN tie member)

As principal tie is a tension member so permissible tensile stress (f_t) for Yst 22 from table-1, IS 806 = 1250 kgf/cm^2

= 125 N/mm^2

Area required for principal tie :-

$$= \frac{\text{load on principal tie}}{f_t}$$

$$= \frac{100 \times 10^3}{125} = 800\text{mm}^2$$

Let provided area = 820mm^2 (choose from table-1, IS 1161).

So let us provide a steel tube of nominal bore of 65mm (medium class) & outside dia of 76.1 mm & provide thickness of 3.6 mm.

check for thickness.

provided thickness (3.6mm) > 3.2mm.

↳ minimum thickness required as per cl 6.3.1 for durability consideration.

so it is ok.

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Design of connection :- (refer appendix B of IS 806).

Length of connection =

$$p = a + b + 3\sqrt{a^2 + b^2}$$

$a = \frac{d}{2} \operatorname{cosec} \theta$

d = outside dia of branch tube = 33.7 mm.

$= \frac{33.7}{2} \operatorname{cosec} 45$

$= 23.83$.

D = outside dia of principal tie = 76.1 mm.

$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$

$$\Rightarrow \frac{33.7}{3} \times \frac{3 - \left(\frac{33.7}{76.1}\right)^2}{2 - \left(\frac{33.7}{76.1}\right)^2} = 17.46$$

$$P = a + b + 3\sqrt{a^2 + b^2}$$

$$= 23.83 + 17.46 + 3\sqrt{(23.83)^2 + (17.46)^2}$$

$$= 129.91 \text{ mm.}$$

Let us assume fillet weld :-

permissible shear stress in weld :-

permissible shear stress in

(i) permissible shear stress in parent material (Yst 22) = 90 N/mm² (table-4)

(ii) for Yst 25, permissible shear stress = 110 N/mm²

minimum will be taken.

so permissible shear stress in weld :- 90 N/mm²

Required area of weld = $\frac{40 \times 10^3}{90} = 444.44 \text{ mm}^2$

Area of weld = effective throat thickness × Length of weld

$444.44 = t_e \times 129.91$

$\Rightarrow t_e = 3.42$ mm.

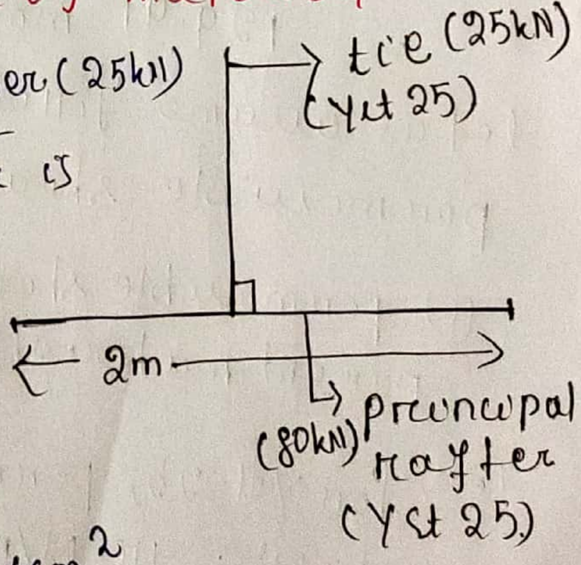
effective throat thickness (t_e) = 0.7 × size of weld

$3.42 = 0.7 \times S$

$\Rightarrow S = \frac{3.42}{0.7} = 4.88$ mm.

So provide size of weld = 4.88 mm.

Q. The principal rafter in round tubular truss (compression member). carries maximum force of 80kN. A tie member meeting at a joint at right angle to carry a force of 25 kN. If the panel length of principal rafter is 2m. Design the member as well as welded joint. Assume outside location & Yst 25 grade of material.



Design of branch tie member (25kN)

As the branch tie member is tension member, so permissible tensile stress for Yst 25 from table 1 (IS 806):-

$f_t = 1500 \text{ kgf/cm}^2$
$= 150 \text{ N/mm}^2$

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Area required for branch tie :-

$$= \frac{\text{Load on branch tie}}{f_t}$$

$$= \frac{25 \times 10^3}{150} = 166.67 \text{ mm}^2.$$

So provide a section of 373 mm^2 (class heavy) of nominal bore size 25 mm & of outside diameter of 33.7 mm & thickness 4mm to satisfy durability requirements.

Design of principal rafter.

let assume slenderness ratio $\lambda = 100$

so for $\lambda = 100$, the safe compressive stress $(f_c) = 91 \text{ N/mm}^2$ (from table-2)

$$\text{Required area} = \frac{\text{load on principal rafter}}{f_c}$$

$$= \frac{80 \times 10^3}{91} = 879.12 \text{ mm}^2$$

Letus provide a steel tube of nominal bore of 76.1mm of heavy class having cross section area 1010 mm^2, outside dia 76.1 mm & the thickness 4.5 mm which is more than 4mm. as per 3.6.3.1. (hence ok).

as nothing is given about end support condition, let assume effective length of principal rafter $(l_{eff}) = 0.85L$.

so, $L_e = 0.85 \times 2$
$= 1.7 m.$

radius of gyration of choosen section(r) = 2.54 cm.

slenderness ratio (λ) $= \frac{1.7 \times 100}{2.54}$
$= 66.92$

permissible stress according to $\lambda = 66.92$. (from table-2 IS 806).

$\lambda = \frac{l}{r}$	f_c (kgf/cm^2) (for Yst 25)
60	~~1002~~ 1207
$\lambda = 66.92$ →	?
70	~~970~~ 1155

by interpolating,

$f_c = \frac{66.92 - 70}{60 - 70} \times (1207 - 1155) + 1155$

$= 1171.016$ ~~N/mm²~~ $kgf/cm^2 = 117.1016\ N/mm^2$

Load $= f_c \times A_{provided}$

$=$ ~~1171.016 × 1010~~

$= 117.1016 \times 1010$

$= 118273\ N = 118.273\ kN > 80\ kN$ (ok).

Design of connection:- (appendix B of IS 806).

length of connection :-

$P = a + b + 3\sqrt{a^2 + b^2}$

$d = 33.7\ mm$

$D = 76.1\ mm.$

$a = \frac{d}{2} \csc\theta$

$= \frac{33.7}{2} \csc 90°$

$= 16.85$

$$b = \frac{d}{3} \times \frac{3-(d/D)^2}{2-(d/D)^2}$$

$$= \frac{\cancel{76.1}\ 33.7}{\cancel{3}\ \ 3} \times \frac{3-\left(\frac{33.7}{76.1}\right)^2}{2-\left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a+b+3\sqrt{a^2+b^2}$

$$P = 16.85+17.46+3\sqrt{(16.85)^2+(17.46)^2}$$

$$= 107.104 \text{ mm}$$

Let us assume fillet weld:-

permissible shear stress in weld:-

(i) permissible shear stress in parent material (Yst 25) = 110 N/mm^2 (table-4)

(ii) for Yst 25, permissible shear stress = 110 N/mm^2

minimum will be taken.

So permissible shear stress in weld = 110 N/mm^2

Required area of weld = $\frac{\text{Load on branch tube}}{\text{permissible shear stress in weld}}$

$$= \frac{25\times10^3}{110} = 227.27 \text{ mm}^2$$

Area of weld = effective throat thickness × Length of weld

$$227.27 = t_e \times 107.14$$

$$\Rightarrow t_e = 2.12 \text{ mm}$$

$t_e = 0.7 \times$ size of weld (s)

$\Rightarrow 2.12 = 0.7 \times s \quad \Rightarrow s = 3.028$ mm.

So provide size of weld = 3.028 mm

Tubular beam :-

limiting deflection of beam :-

The maximum deflection should not exceed 1/325 of the span for simply supported members. This requirements may be satisfied if the bending stress in compression or tension does not exceed $\frac{31500 D}{L}$ kgf/cm², where 'D' is the outer diameter of the tube in cm & 'L' is the effective Length of beam in cm.

Q. A medium steel tubular section of 200 mm nominal diameter is simply supported as a flexural member over effective span of 4.5 m. Determine the safe uniformly distributed super imposed load which can be placed over it? Assume Yst 25 grade of steel).

Section properties of 200 mm nominal dia medium class steel tube :-

(from table-I)

thickness (t) = 5.6 mm., γ = 29.5 kg/m.

= 289.395 N/m.

$E = 2 \times 10^5$ Mpa.

Area of cross section (A) = 37.5 cm²

= 3750 mm²

modulus of section (Z) = 195 cm³

= 195000 mm³.

moment of Inertia (I) = 2141 cm⁴

= 2141×10^4 mm⁴

$$b = \frac{d}{3} \times \frac{3-(d/D)^2}{2-(d/D)^2}$$

$$= \frac{\cancel{76.1}\ 33.7}{3} \times \frac{3-\left(\frac{33.7}{76.1}\right)^2}{2-\left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a+b+3\sqrt{a^2+b^2}$

$$P = 16.85+17.46+3\sqrt{(16.85)^2+(17.46)^2}$$

$$= 107.104 \text{ mm}$$

Let us assume fillet weld:-

permissible shear stress in weld:-

permissible shear stress in parent

(i) permissible shear stress in parent material (YSt 25) = 110 N/mm² (table-4)

(ii) for YSt 25, permissible shear stress = 110 N/mm²

minimum will be taken.

So permissible shear stress in weld = 110 N/mm²

$$\text{Required area of weld} = \frac{\text{Load on branch tube}}{\text{permissible shear stress in weld}}$$

$$= \frac{25\times10^3}{110} = 227.27 \text{ mm}^2$$

Area of weld = effective throat thickness × Length of weld

$227.27 = t_e \times 107.14$

$\Rightarrow t_e = 2.12$ mm.

$t_e = 0.7 \times$ size of weld (s)

$\Rightarrow 2.12 = 0.7 \times s \Rightarrow s = 3.028$ mm.

(i) Load carrying capacity based on bending stress:-

permissible bending stress for Yst 25 (f_b) = 1655 kgf/cm^2

$= 165.5 \text{ N/mm}^2$

allowable bending stress due to load applied (f_{ab}) $= \frac{M}{Z}$.

for simply supported beam, bending moment due to uniformly distributed load (M) $= \frac{w l_{eff}^2}{8}$ (l_{eff} given = 4.5m)

$= \frac{w(4.5)^2}{8} = 2.53 w \text{ N mm}$.

section modulus (Z) from section properties = 19500mm^3,

$f_{ab} = f_b$

$\frac{M}{Z} = f_b \Rightarrow \frac{2.53 w \times 10^3}{195000} = 165.5$

$\Rightarrow w = 12755.928 \text{ N/m}$.

$= \boxed{12.756 \text{ kN/m}}$

(2) based on shear stress:-

for Yst 25, permissible maximum shear stress (f_s) = 110 N/mm^2

allowable shear stress due to applied load (f_{as}) $= 2 \times \frac{\text{maximum shear force (V)}}{\text{Area of section}}$

Maximum shear force (V) due to applied load $= \frac{w\,l_{eff}}{2} = \frac{w \times 4.5}{2} = 2.25w$ N (Let w is in N/m)

Area of section from section properties:-

$A = 3750 mm^2$

Now, $f_{as} = f_s$

$\Rightarrow 2 \times \frac{V}{A} = f_s \Rightarrow 2 \times \frac{2.25w}{3750} = 110$

$\Rightarrow w = 91667$ ~~[illegible]~~ N/m

$= \boxed{91.667 \text{ kN/m.}}$

(B) Load carrying capacity in view of deflection:-

Maximum permissible deflection for simple supported beam $\delta_{max} =$

$\frac{1}{325} \times l_{eff}$

$= \frac{1}{325} \times 4.5 \times 10^3$

$= 13.85$ mm.

(w is in N/mm)

Deflection due to applied load:-

$\delta = \frac{5}{384} \cdot \frac{w(l_{eff})^4}{EI}$

$= \frac{5}{384} \frac{\times w \times (4.5 \times 10^3)^4}{2 \times 10^5 \times 2141 \times 10^4 \times 10^3}$

$=$ ~~[illegible]~~ $1246.93w$

Now $\delta = \delta_{max}$

$\Rightarrow 1.25w = 13.85 \Rightarrow w = 11.08$ N/mm.

$= \boxed{11.08 \text{ kN/m.}}$

load carrying capacity = minimum of above 3 loads calulated

w = 11.08 kN/m.

self wt. of member (dead load) or = 289.395 N/m.

= 0.289 kN/m.

w = super imposed load (live load) + dead load.

=> super imposed load = w - dead load

= 11.08 − 0.289

= 10.79 kN/m.

Question Bank Of Structural Design - II

2 mark Questions

1) (a) For bolts of property class 4.6, what do the numbers 4 and 6 indicate.

(b) What is the angle between fusion faces for fillet weld?

(c) Define bolt value.

(d) How are the connections classified?

(e) Define pitch.

(f) Define radius of gyration.

(g) What are the advantages of butt joint over lap joint?

(h) Define staggered pitch.

(i) Write 2 advantages of welding over bolting.

(j) Two plates of 8mm and 18 mm thickness are to be joined using longitudinal fillet weld. Suggest a suitable size of weld.

(k) What is the recommended throat thickness for incomplete penetration butt welds welded from one side only?

(l) What is the objective of providing tack rivets in steel structural members.

(m) State the types of bolts used in structure.

(n) Sketch the basic sections and symbols for single V-butt weld.

(o) Sketch the basic sections and symbols for double V-butt weld.

(p) what do you mean by structural steel?
(q) what is rolled steel section & welded steel section
(r) why load combination is necessary in steel design
(s) what is HSFG bolt.
(t) Define end distance & edge distance
(u) what are the types of bolted connections.
(v) Define welding
(w) what do you mean by slot weld or plug weld?
(x) Define net sectional area of tension member.
(y) what is the min^m & max^m value of pitch of bolts in a tension member.
(z) what base plate is required below a column section
7(a) Mention the types of buckling in a compression member
(b) where do you recommend base plate.
(c) what is the limit of slenderness ratio for a short and solid rectangular column.
(d) what is effective length of compression member for a simply supported column.
(e) How slenderness ratio influences design of steel structure
(f) State the basic difference between slab base and gusseted base.
(g) what are the types of column bases usually used.
(h) what will be the buckling class of ISHB 400 @ 907 N/m about z-z and y-y axis?
(i) Differentiate between web buckling & web crippling of beams.
(j) How do you obtain permissible stress for timber of select grade-I and grade-II, when the strength of grade-I timber is given.

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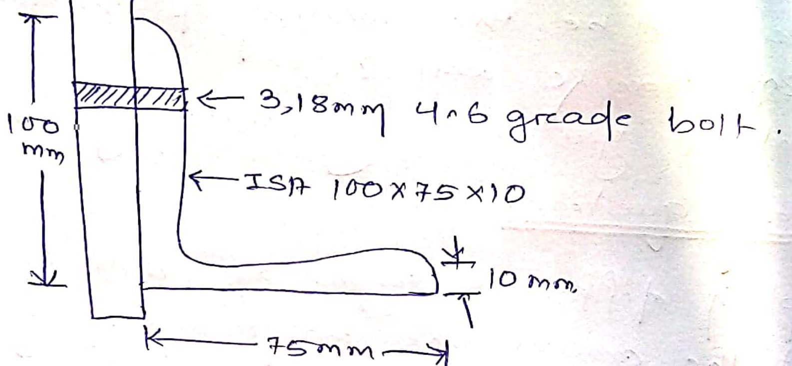
(k) How are the structural timbers graded?
(l) What do you mean by grading of timber?
(m) Write 3 classifications of mortars.
(n) What is slenderness ratio of a masonry wall.
(o) For what type of structures, tubular sections are suitable.
(p) Where will be the location of critical section of bending moment for RC wall?.
(q) What do you mean by crinkling of tubes.

5 marks questions

1) Explain different types of butt welds with neat sketch
2) List the assumptions made in design of bearing bolts.
3) Explain special consideration in steel design.
4) Explain the advantages of steel structures.
5) Discuss advantages & disadvantages of bolted connection.
6) What do you mean by slip critical sections? Explain the principle of high strength friction grip bolts.
7) Write down the properties of structural steel.
8) Describe the concept of shear lag.
9) Write down the advantages of welded connection over bolted connection.
10) Explain the concept of block shear in the design of tension member.
11) A tie member of a roof truss consists of 2 ISA 90×60 ... The angles are connected on either side of 10 mm gusset plates and the member is subjected to a factored pull of 360 KN. Design the welded connection assuming welding is to be made in the field.
12) Design a single angle tension member of a roof truss to carry a factored tensile force of 225 KN. The member is subjected to the possible reversal of

stress due to action of wind. The length of the member is 3m. Use 20mm shop bolts of grade 4.6 for the connection.

13) Determine the effective net area of the angle section shown in figure.



14) A tie member of a roof truss consists of 2, ISA 90×60×10. The tie member is subjected to a pull of 200 KN. The angles are connected to either side of a 12mm thick gusset plate. Design welded connection.

15). Design a suitable slab base for a column section ISJB 200 @ 365.9 N/m supporting an axial load of 400kN. The base plate is to rest on a concrete pedestal of M20 grade.

16) Design a suitable slab base for a column section ISHB 200 @ 365.9 N/m supporting an axial load of 400 KN. The base plate is to rest on a concrete pedestal of M20 grade. Use steel of grade Fe 410.

17) Determine the design axial load on the column section ISMB 400, given that the height of column is 3m and that is pin ended. Also assume $f_y = 250\,N/mm^2$, $f_u = 410\,N/mm^2$ & $E = 2\times10^5\,N/mm^2$.

18) Explain the buckling class of cross-sections in compression member.

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19) Find the form factor and moment of resistance of the cross-section of the beam of Dhaman wood for Rectangular section of width 200mm, depth = 350 mm.

20) Write the codal provision of design consideration for masonary walls under eccentric loading.

21) Determine the plastic moment capacity of the unsymmetrical I-section. Given size: Top flange - 100mm x 20mm, bottom flange - 200mm x 20mm and web - 200mm x 20mm.

22) What are the factors that determines the buckling class of structural elements? Determine the buckling class of ISHB 400 @ 806.4 N/m.

23) Write short note on web buckling and web crippling

24) Short note on design consideration for masonary footing

25) If four planks 160mm x 40mm are to be formed in the shape of a box, find the maximum load for the mango timber with unsupported length of 3.5m in inside location.

26) A column 1450mm x 150mm is made of babul wood. The unsupported length is 3.7m. Determine the safe axial load on the column.

27) A column 120mm in dia. is made of deodar wood. The effective length of column is 1.20m. Determine the safe axial load of the round column. The column is situated in outside location. Take safe working stress in axial compression parallel to grains for outside location. $f_{cp} = 7\ N/mm^2$ -

28) A timber column 200mm x 200mm section having an unsupported length of 3.5m. find the safe axial load for column assuming it to be sal wood.

stress
member
for
13)

29) Write short note on crinkling in tubular steel compression members.

30) Write short note on design consideration for masonary wall footing.

10 marks questions

1) Design a lap joint to connect two plates 300 mm wide and 16 mm thick using 20 mm dia bolts of grade 4.6. The applied service load is 375 KN.

2) Design a lap joint between two plates each of width 100 mm, if thickness of one plate is 16 mm & the other is 12 mm. The joint has to transfer a design load of 180 KN. The plates are of Fe410 grade and M16 bolts of 4.6 grade.

3) Design a welded lap joint for 2 plates of size 120 mm x 8 mm and 120 mm x 12 mm for maximum efficiency. Assume shop welding & Fe410 grade of steel.

4) Find the maximum force that can be transmitted through a double bolted chain lap joint consisting of 6 bolts in two rows. Given that M16 bolts are 4.6 grade & plates are of Fe410. The thickness of the plates connected are 10 mm and 12 mm.

5) Calculate the strength of a 20 mm diameter bolt of grade 4.6 for double cover butt joint each of the cover plate being 8 mm thick & main plates to be jointed are 12 mm thick.

6) ~~A~~ ~~cal~~ Design a welded lap joint for two plates of size 200 mm x 8 mm and 200 mm x 12 mm for maximum efficiency. Assume shop welding & Fe410 grade steel.

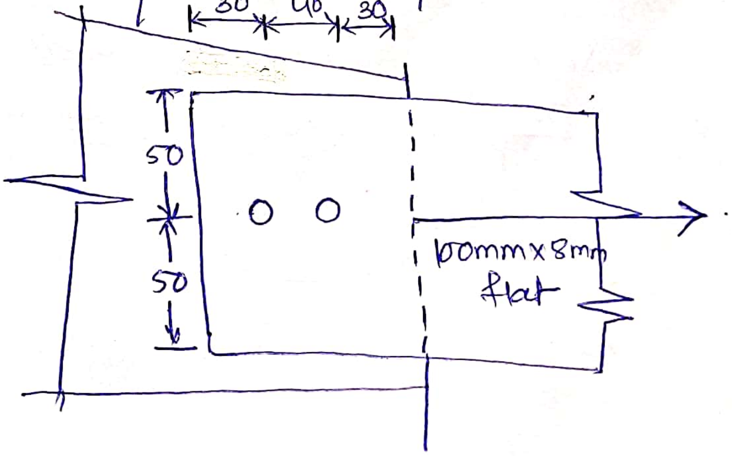
7) Find the maximum force that can be transmitted through a double bolted chain lap joint consisting of 6 bolts in 2 rows connecting 2 plates of thickness 12 mm and 10 mm. Given M16 bolts of grade 4.6 and plates of Fe410 are to be used.

8) Design a lap joint to connect two plates 300 mm wide & 16 mm thick using 20mm dia bolts of grade 4.6. The applied service load is 375 KN.

9) What are the predominant limit states in limit state method of design?

10) Two steel plates of Fe410 grade 16mm thick are to be joined by 24 mm dia bolts of property class 4.6. Assuming a pitch of 60mm and edge distance of 40mm, Calculate the strength of bolt in case of (i) Lap joint, (ii) Double cover butt joint with 10mm thick cover plate.

11) A tension member consists of a flat 100mm x 8mm which is connected to a gusset plate of 10mm thick by 2nos. of 16mm dia bolts as shown in fig. Determine the strength of the flat against yielding, rupture and block shear. Also determine the maximum load the joint can carry safely? Assume steel of grade Fe 410 and bearing bolts of property class 4.6 in the field.



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12) ~~A~~ A tie member of a roof truss consists of 2 ISA 90 x 60 x 8mm. The angles are connected on the either side of 10mm gusset plates and the member is subjected to a factored pull of 360 KN. Design the welded connection. Assume welding is to be made in the field.

13) Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225 KN. The member is subjected to the possible reversal of stress due to the action of wind. The length of the member is 3m. Use 20 mm shop bolts of grade 4.6 for the connection.

14) A tencion member 0.8m long to resist a service load of 20KN and a service ~~load~~ load of 50KN. Design a rectangular bar of standard structural steel of grade Fe 410. Assume that member is connected ~~to~~ by one line of 16mm dia bolts of grade 4.6.

15) Design a column section to carry a working axial load of 400KN. The column is 40m long and effectively held in positions and restrained against direction at both end. Consider $f_y = 250\ N/mm^2$.

16) A column ISWB 300 @ 471.8 N/m is to carry an axial factored load of 800 KN. M20 concrete is used for the foundation. Design the slab base. Provide welded connection between column and base plate. Given that the column ~~end~~ end and base plate are not machined for bearing.

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17) Design a steel column section using channel section only to carry a factored axial load of 400 KN. The column is 4m long and is effectively held in position at both ends but restrained against rotation at one end only. Consider fy = 250 MPa and assume wind / earthquake actions.

18) Calculate the design compressive load for an ISHB 250 @ 536.6 N/m, 4m high. The column is restrained in erection only at both the ends. It is to be used as an uncased column in a single storey building.

19) Design a slab base for a column ISHB 350 @ 710.2 N/m subjected to a factored load of 15000 KN. M25 concrete is used for the foundation. Provide welded connection between column and base plate.

20) Design a simply supported beam of effective span 2.5m carrying a factored concentrated load of 300 KN at mid span point assuming is to be ~~to~~ laterally supported (restrained) throughout.

21) A laterally supported beam ISMB 600 @ 1202.71 N/m is placed between two supports. Determine the safe uniformly distributed load the beam can carry for an effective span of 8m. Take fy = 250 N/mm². Neglect web buckling and web crippling.

22) Determine the safe axial load on a circular column of 180mm diameter made up of deodar (HP) wood for following cases.

(i) Unsupported length of column is 3.0m (outside location)

(ii) Unsupported length of the column is 4.5m (inside location).

23) Determine safe axial load on a circular column of 190 mm diameter made up of deodar (HP) wood. Unsupported length of column is 3.30m being situated in outside location.

24) Design a ~~steel~~ simply supported beam of effective span 2.5m carrying a factored concentrated load of 300 KN at mid-span point. Assuming it to be laterally supported.

25) Design a slab base for a column ISHB 350 @ 710.2 N/m subjected to a factored load of 15000 KN. M25 concrete is used in foundation. Provided welded connection.

26) Design a simply supported beam to carry a uniformly distributed load of 50 KN/m. The effective span of beam is 9m. The compression flange of the beam would be prevented from lateral deflection.

27) Design a gusseted base of a column consisting of ISHB 400 × 82.2 kg/m with flange plate 300mm × 16mm on each flange. The column carries a load of 2000 KN and is supported on concrete pedestal with a bearing capacity of 40 MPa.

28) A timber beam having a clear span of 6.0m. carries a UDL of 15 KN/m including the self weight of beam. Assuming the beam to be made of Deodar wood, design the beam.

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