DESIGN OF COMPRESSION MEMBERS

Many Structural members are in compression. verticle compression members in buildings are called coloumns, stanchions, posts. compression members in trusses are called as "struts". The Jib of crane which carries compression is called a "Boom"

Buckling class of cross section :- Imperfections of fabrication resulting into essentricity largely depends upon the cross section of the compression member. based on such imperfection buckling tendency where is. for IS 800 2007 (table 10) (pg-44) gives various cross sections into 4 buckling classes A, B, C and D.

Slenderness Ratio :- It is defined as the ratio of effective length to it least radius of Gyration of the section

$$\text{Slenderness Ratio} = \frac{L_e}{r} , \frac{K.L}{r_{min}}$$

Where K = Factor for effective length depends on end support conditions

(Given by table - 11 Pg-45)

Design compressive stress and strength :- (pg - 34, cl- 7.1.2)

Design compressive stress (f_{cd}) for axially loaded compression members

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{m0}}$$

where $\phi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$

and λ = Non dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

Where $f_{cc} = \frac{\phi^2 E}{}$

f_{cc} = Eulers buckling stress

α = Imperfection factor. (given by table - 7 pg - 35)

$\gamma_{m0} = 1.1$

The design compressive strength

$$\boxed{P_d = A_g * f_{cd}}$$

Eg:- Determine the design axial load capacity of the column ISHB 300 @ 577 N/m, If the length of the column is 3m and its both ends are pinned.

Sol:- Given that

ISHB 300 @ 577 N/m

L = 3m , both ends hinged

Step1 :- Data + Assumptions.

ISHB 300 @ 577 N/m , L = 3m

Assume $f_u = 410\ N/mm^2$, $f_y = 250\ N/mm^2$

$E = 2 \times 10^5\ N/mm^2$

Step2 :- Section properties

Sectional Area (A) = 7485 mm^2

depth of the section (d)(or (h)) = 300 mm

breadth of the flange = 250 mm

Thickness of the flange = 250 mm

r_{min} = 54.1 mm.

Step3 :- Buckling class classification (Table - 10, pg-44)

$$\frac{h}{b_f} = \frac{300}{250} = 1.2 \quad , \quad t_f = 10.6\ mm$$

buckling about z-z → b - 0.34 (Pg - 35, T - 7)

y-y → c - 0.49

Step4 :- Design compressive strength (Cl- 7.1.2 Table 9) pg-3

$$P_d = A_c * f_{cd}$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi\,[\phi^2 - \lambda^2]^{0.5}} \le \frac{f_y}{\gamma_{m0}}$$

$$\phi = 0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$$

"α" from table "7" $\alpha = 0.34$ $z z$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r_{min}}\right)^2} = \frac{\pi^2 \times 2\times10^5}{\left[\frac{3000}{54.1}\right]^2} = 641.9 \text{ N/mm}^2$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.9}} = 0.62$$

and $\phi = 0.5\,[1 + 0.49(0.62 - 0.2) + 0.62^2]$

$\phi = 0.79$

$$f_{cd} = \frac{\frac{250}{1.1}}{0.79 + [0.79^2 - 0.62^2]^{0.5}}$$

$= 189.48 \text{ N/mm}^2$

design strength $P_d = 7485 \times 189.4 = 1418.19$ KN

Applied load (or) working load $= \frac{1418.19}{1.5} = 945$ KN

(or)

Method (2):- Design compressive strength (P_d) (cl-7.1.2 pg 34)

$P_d = A_e \times f_{cd}$

from table 9(b); $f_{cd} = ?$ $\rightarrow$ $\frac{KL}{r_{min}} = \frac{1 \times 3000}{54.1}$

$\frac{KL}{r_{min}} = 55.45$

$f_y = 250 \text{ N/mm}^2$

table 9(b):-

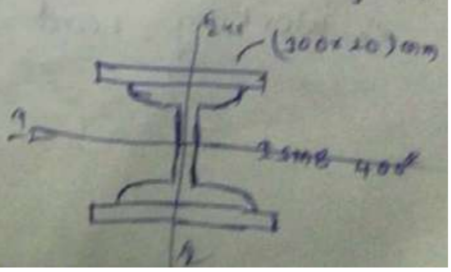
$\frac{KL}{r}$	f_{cd}
50	194
55.45	?
60	181

$f_{cd} = 186.9 \text{ N/mm}^2$

$\therefore P_d = 7485 \times 186.9 = 1398.9$ KN

$\therefore$ Working load $= \frac{1398.9}{1.5} = 932.2$ KN

(*) Determine the load carrying capacity of the column shown in figure if its actual length is 4.5 m. Its one end may be assumed to be fixed and other end hinged. assume Necessary Data.



Given data is

ISMB 400

actual length = 4.5 m

fixed – Hinged

Step 1 :- Data & Assumptions

ISMB 400 section, $L = 4.5m$

Assume $f_u = 410\ N/mm^2$, $f_y = 250\ N/mm^2$

$E = 2 \times 10^5\ N/mm^2$

Step 2 :- Sectional properties

ISMB 400 : $h = 400mm$, $b_f = 140mm$, $t_f = 16mm$,

$t_w = 8.9mm$ $A = 7846\ mm^2$, $I_{zz} = 20458.4 \times 10^4\ mm^4$

$I_{yy} = 422.1 \times 10^4\ mm^4$

Step 3 :- Buckling classification "c"

$\cancel{I_{zz}} = \cancel{20458.4}$

for Builtup section: I_{zz}, I_{yy}

$I_{zz} = 20458.4 \times 10^4 + 2[(300 \times 20) \times 210^2]$

$= 733.78 \times 10^6\ mm^4$

$I_{yy} = 422.1 \times 10^4 + 2\left[\frac{20 \times 300^3}{12}\right] = 94.22 \times 10^6\ mm^4$

$I_{yy} < I_{zz}$

Step 4 :-

$r_{min} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{94.22 \times 10^6}{(7846 + 2(300 \times 20))}}$

$\therefore r_{min} = 68.9\ mm^4$

$\frac{KL}{r} = \frac{0.8 \times 4500}{68.9} = 52.23$

$K = 0.8 \rightarrow$ for given supp

$f_y = 250\ N/mm^2$

$P_d = A_c * f_{cd}$

$P_d = 19846 * 179.64$

$\therefore P_d = 3565.13\ KN$

$\therefore$ Working Load $= 2376.75\ KN$

$50 \rightarrow 183$

$52.24 \rightarrow ?\ 179.64$

$60 \rightarrow 168$

② - ① $\rightarrow 10 \quad -15$

③ - ① $\quad 2.24 \quad (x - 183)$

$-33.6 = 10x - 1830$

$f_{cd} = 179.64\ N/mm^2$

DESIGN OF COMPRESSION MEMBERS:-

*(i) Design stress in compression is to be assume

(a) For rolled steel beam section the slenderness ratio where as 70 to 90. Hence corresponding design stress assumed to be 135 N/mm^2

(b) Angle struts slenderness ratio varies 110 to 180 corresponding design stress asumed to be 90 N/mm^2

(c) Large members are built up members due to consider 200 N/mm^2

(ii) Effective sectional area required

$$Pd = A_c * f_{cd} \Rightarrow A_c = \frac{Pd}{f_{cd}}$$

(iii) choose (or) select a section corresponding to required effective area depend on $\frac{KN}{R}$ an end condition check the section feasible (or) Not. for with respect to the applied load (or) given load.

⊛ A coloumn 4m long has to support a factored load of 6000 KN. The coloumn is effectively held at both ends and restrained in direction at one of the end. design the coloumn using beam section and plate.

Sol:-

Step1:- Assume the design stress in compression.

assume $f_{cd} = 200\ N/mm^2$

$$A_c = \frac{Pd}{f_{cd}} = \frac{6000 \times 10^3}{200} = 30,000\ mm^2$$

Section properties of ISHB @ 450

Sectional area $(A) = 117.89\ cm^2 = 11789\ mm^2$

$b_f = 250\ mm$

$t_w = 11.3\ mm$

$t_f = 13.7\ mm$

Total Area of section $= 30000 - 11789 = 18211\ mm^2$

Select 20 mm plates

$26 \times 20 = 18211$

$\therefore b = 455$ mm

choose (500 x 20) mm plate.

breadth of the plate $b_p = 500$

Thickness of the plate $t_p = 20$

$$= \frac{b_p - b_f}{t_p} = \frac{500 - 250}{20} = 12.5$$

$$\frac{KL}{r} = \frac{0.8 \times 4000}{r}$$

where $r = \sqrt{\frac{I_{yy}}{A}}$

For builtup section :-

$I_{yy} = 3045.0 \times 10^4$ mm^4

$I_{xx} = 40349.9 \times 10^4$ mm^4

$$I_{yy} = 3045.0 \times 10^4 + 2\left[\frac{20(500)^3}{12}\right]$$

$= 447.11 \times 10^6$ mm^4.

$\text{Area} = 11789 + (2(500 \times 20)) = 31789$ mm^2

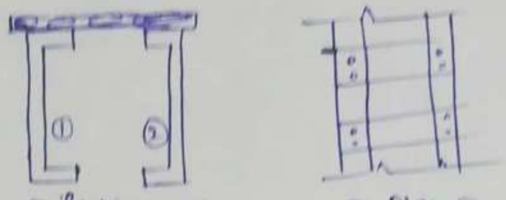
$$r = \sqrt{\frac{447.1 \times 10^4}{31789}} = 11.85$$

$$\Rightarrow \frac{KL}{r} = \frac{3200}{11.85} = 26.82$$

(pg - 42)

* The objective of providing lateral system is to keep the main member of the coloumn away from principle axis then the lacings are subjected to shear force due to horizontal forces on Colomn.

battens :- Instead of lacings we can use battens to keep member of coloumn at required distances.



~ Sections ~ ~ Plan ~

DESIGN OF LACED COLOUMNS :- As per code IS 800-2007 (pg-48, cl-7.6)

* The thickness of flat lacing bars, shall not be less than $\frac{1}{40}$ times of its effective length for single lacing. and $\frac{1}{60}$ times of its effective length for double lacing. (7.6.3)
* Lacing bars shall be placed at inclained at 40°-70° to the axis of the built up member. (7.6.4)
* The distance b/w the two main members should be kept, as to get "$r_{yy} > r_{zz}$" (7.6.8)
* Maximum spacing of lacing bars shall be such that the maximum slenderness of the main member b/w consecutive lacing connections is not greater than 50 (or) 0.7 times the most unforble slenderness ratio of member as a whole (6.5.1 (or) 7.6.5.1)
* The lacing shall be designed to resist transverse shear (V_t) is equal to 2.5% axial force in the coloumn, if there are two transverse parllel system then each system has to resist "$\frac{V_t}{2}$" shear force. (7.6.5.2)
* If the coloumn is subjected to bending also (V_t) then the $\boxed{\text{transerves shear} = \text{bending shear} + 2.5\% \text{ colo force}}$
* Effective length of single laced system is equal to length b/w the inner end fastener. (7.6.6.1) for welded joints and double laced, effectively conn

at intersection, effective length may be taken as 0.7 times of the actual length

* The slenderness ratio "$\frac{KL}{r}$" for lacings bars should not exceeds 145 (7.6.6.3)

* The effective length of slenderness ratio $\left(\frac{KL}{r}\right)$ of Laced coloumns shall be taken as "1.05" times the actual maximum slenderness ratio, in order to account for shear deformations effects. (7.6.1.1)

DESIGN OF BATTENS :- (Cl- 7.7, Pg-51)

* B

Problems on Laced coloumns – Design :-

⊛ Design a Laced coloumn with two channels back to back of length 10 m, to carry an axial factored load of 1400 KN. the coloumn may be assumed to have restrained in position but not in direction at both ends

sol:-

Step1:- design of channel section

Assume $f_{cd} = 135\ N/mm^2$

$$\text{Area required } (A_c) = \frac{P_d}{f_{cd}} = \frac{1400 \times 1000}{135}$$

$$\therefore A_c = 10370\ mm^2$$

Take ISMC @ 350 ; 49.4 kN/m

Area provided = $2 \times 5366 = 10732\ mm^2$; $r_{zz} = 136.6$

distance will be maintained so as to get $r_{yy} \geq r_{zz}$

actual $\frac{KL}{r} = \frac{1 \times 10000}{136.6} = 73.206$ (for builtup section)

Since it is laced coloumn $\frac{KL}{r} = 1.05 \times 73.206 = 76.87$

from table. (9cc) pg-42) from buckling class "c" (pg-44)

by interpolating $f_{cd} = 141\ N/mm^2$

ⓐ load carrying capacity of laced coloumn

$$P_d = f_{cd} * \text{Area}.$$

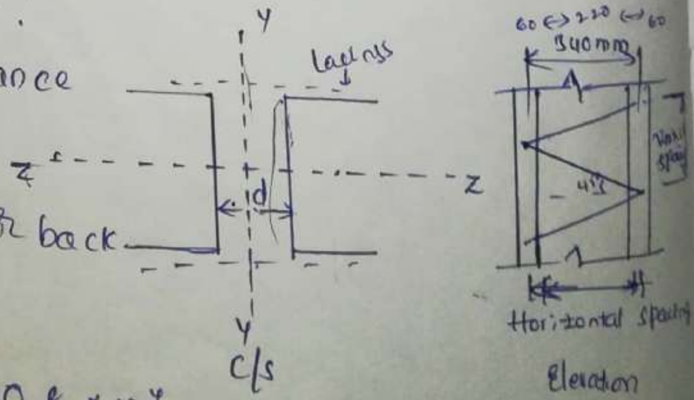
$$P_d = 141 \times 10732 = 1513.2 > 1400\ KN$$

Hence the design load carrying capacity is good

ⓑ Spacing b/w the channels :-

Let "d" be "clear distance b/w channels.

For built up section for back to back channels.



$$I_{xx} = I_{zz} = 2 \times 10008 \times 10^4$$

$$= 20016 \times 10^4\ mm^4$$

$$I_{yy} = 2\left[430.6 \times 10^4 + 5366\left(d/2 + 24.4\right)^2\right]$$

$I_{xx} = I_{yy}$

$$2\left[430.6 \times 10^4 + 3266\left(\frac{d}{2}+244\right)^2\right] = 20016 \times 10^4$$

$$\left(\frac{d}{2}+244\right)^2 = 17848.3$$

$$\therefore \boxed{d = 218.4\,mm}$$

take distance as b/w two channel section back to back $\boxed{d = 220\ mm}$

design of Lacings:-

Let Lacings can provided 45° horizontal and ~~that~~ horizontal distance of the lacings = 220+60+60 = 340 mm

Verticle spacing = 340 tan 45° × 2 = 680 mm

ryy of minimum of channel = 28.3 mm

from that $\frac{KL}{r}$ channel ~~[illegible]~~ = ~~[illegible]~~

$$\frac{KL}{r} = \frac{680}{28.3} = 24.02$$

Transverse shear resisted by lacing systems

= 2.5% Actual force

Shear to be resisted by each lacing system $= \frac{35000}{2}$

= 17500 N

Length of lacing $= 340 \times \frac{1}{\cos 45°} = 480.83\,mm$

(or) $\boxed{(\text{HP})^2 = (\text{OP})^2 + (A)^2}$

$\therefore$ thickness of lacings $= \frac{1}{40} \times 480.83$

= 12.02 mm

$\therefore$ Width of Lacings = 3 times (*) Nominal dia of bolting (or) riveting

= 3 * 20 = 60 mm

Lacings ~~at the~~ choose indian standard (IS) flats "60 mm" ISF

⊛ Design a Battend coloumn with two channels back to back of length 10 m, to carry an axial factored load of 1000 kN the coloum may be assumed to have restrained in position and also in direction at both ends

Sol:-

Step1 :- design of channel section

Assume $f_{cd} = 135 \; N/mm^2$

Required Area $(A_c) = \frac{P_d}{f_{cd}} = \frac{1000 \times 1000}{135} = 7407.4$

$\therefore A_c = 7407.4 \; mm^2$

take ISMC @ 400 ; 494 N/m

provided Area = $2 \times 6293 = 12586 \; mm^2$

$r_{zz} = 136.6 \; mm^4$

distance will be maintained so as to get $r_{yy} > r_{zz}$

actual $\frac{KL}{r} = \frac{0.65 \times 10000}{136.6} = 47.58$ (for builtup sectn)

for battens $1.1 \times \frac{KL}{R} = 1.1 \times 47.58 = 52.33$

Properties of ISmc @ 400 @ 494 N/m is

depth of the Section (h) = 400 mm

width of the flange (b) = 100 mm

thickness of the flange (t_f) = 15.3 mm

thickness of the web (t_w) = 8.6 mm

table 9(c) — pg-42

$\therefore f_{cd} = 191 \; N/mm^2$

40 - 198
47.58 - ?
50 - 183

$10 \rightarrow$ 15
7.58 x-15
$10x = 1980 = 113$

Load Carrying Capacity :-

$P_d = f_{cd} * \text{Area}$

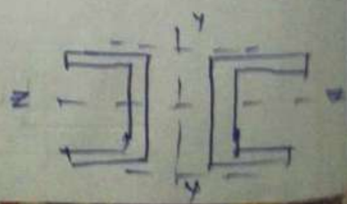
$P_d = 191 * 7407.4$

$\boxed{P_d = 1414.8 \; kN} > 1000 \; kN$

Hence the design load carrying capacity is high hence section can be safe against load carrying.

Spacing b/w the two channels :-

Let "d" be the clear distance b/w two channels.



$I_{xx} = I_{zz} = 15082.8 \times 10^4 \; mm^4$

We know that $I_{xx} = I_{yy}$

$$\frac{(15082.8 \times 10^4) - (430.6 \times 10^4)}{2 \times 6293} = \frac{d}{2} + 24.2$$

$$11641.66 = \frac{d + 48.4}{2}$$

$$d = 236.7 \simeq 240 \text{ mm}$$

Design of battens :-

Let "c" be spacing of battens Longitudinally, for that

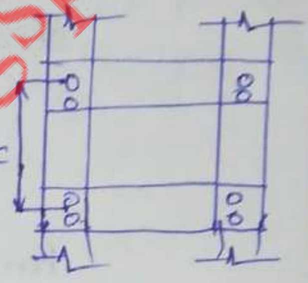
Radius of Gyration of channel section = 23.8 mm

2 back to back ISMC 250 @ 297.2 N/m

$$\frac{KL}{r} = 1.1 \times \frac{0.65 \times 10000}{99.4} = 71.9$$

(i) $\frac{c}{23.8} < 50 \Rightarrow c < 1190$ mm

(ii) $\frac{c}{23.8} < 0.7 \times 71.9 \Rightarrow c < 1197.8$ mm



Let us consider 1100 mm as distance b/w batten to batten

$\therefore \boxed{c = 1100 \text{ mm}}$

transverse shear $V_t = \frac{2.5}{100} \times 1000 \times 10^3 = 25000$ N

$$V_b = \frac{V_t\, C}{N S} = \frac{25000 \times 1100}{2 \times 230} = 59782.60$$

Where

V_t = transverse shear force

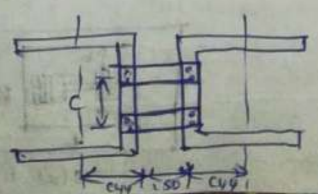
C = distance b/w c/c of battens Longitudinally,

N = Number of parallel planes of battens.

S = Minimum transverse distance b/w Centroid of rivet/bolt connecting of batten to the main member.

$$\text{Bend-Moment } (M) = \frac{V_c * C}{2N} = \frac{25000 \times 1100}{2 \times 2}$$

$$\boxed{M = 6875000 \text{ N-m}}$$



Size of the batten :-

effective depth of end batte

$\Rightarrow (150 + 2(23) = 1$

∴ Let, us take effective depth 210 mm

for intermittent battens

effective depth ⊄ $\frac{3}{4} \times 196 = 147$ mm

∴ provide effective depth of intermittent battens = 160 mm

give edge distance = 40 mm

∴ overall depth of battens = 240 mm

∴ Thickness of the batten shall not less than 1/50th of c/c distance

$$T \not< \frac{1}{50} \times 230 \not< 4.6$$

take 8mm thickness of batten plates

check for stresses in batten plates:-

$$\text{shear stress} = \frac{597820}{8 \times 210} = 35.5 \text{ N/mm}^2.$$

$$< \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} = 131.21 \text{ N/mm}^2$$

$$\text{Bending stress} = \frac{6 \times M}{t d^2} = \frac{6 \times 6875000}{8 \times 210^2}$$

$$= 116.92 \text{ N/mm}^2 < \frac{f_y}{1.1} \times 1.2 = 272.72 \text{ N/mm}^2$$

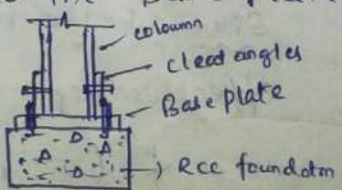
∴ The designed battens are safe against bending and shear.

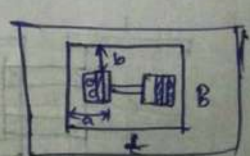
Coloumn Bases:-

Coloumn bases transmit the coloumn load to the concrete (or) masonry foundation work. The coloumn base spreads the load on wider area. so that the intensity of bearing pressure on the foundation block is with in the bearing strength. There are 2 types of coloumn bases

① Slab base
② Gusseted base.

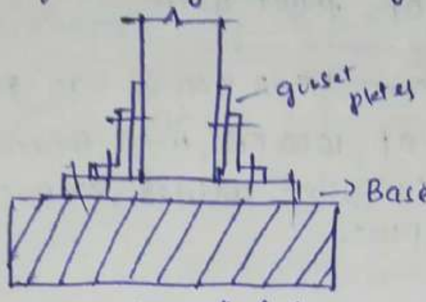
Slab base:- These are used in coloumns to carrying small loads. The coloumn is directly connected to the base plate through cleat angles. as shown in figure the load is transfered to the base plate through bearing





Slab base

Gusseted Base :- For column carries heavy loads gusseted bases are used. In this the column is connected to the base plate through gussets. The load is transfered to the base plate partly through bearing and partly through gussets.



gusseted base

DESIGN OF SLAB BASE :-

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

① Step1 :- ① Size of base plate

find the bearing strength of concrete by $0.45 f_{ck}$

Step2 :- Area of base plate required

$$A_{base\ plate} = \frac{P_u}{0.45 f_{ck}} \quad (P_u = \text{factored load})$$

Step3 :- Select the size of the base plate

for economy as far as possible the projections a and b are equal.

② Thickness of the base plate

Step1 :- find the intensity of pressure

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

Step2 :- minimum thickness of base plate

$$t_s = \sqrt{\frac{2.5\, w\, (a^2 - 0.3 b^2)\, \gamma_{mo}}{f_y}} > t_f$$

t_s = thick of base plate

t_f = thickness of flange of ~~section~~ column ~~of colomn~~

③ connections

Step1 :- connect base plate to foundation concrete using #4, 20 mm ϕ and 300 mm long anchor Bolts.

Step 2:- If bolted connection is to be used for connecting coloumn to base plate ...

Step 3:- If weld is to be used for connecting coloumn to base plate, check the weld length of fillet weld.

Design a slab base for a coloum ISHB 300 @ 577 N/m carrying an axial factored load of 1000 KN, M20 concrete can be used to the foundation. Provided welded connection b/w the coloumn and base plate.

Sol:- Given data is

ISHB 300 @ 577 N/m

Factored Load $(P_u) = 1000$ KN.

Grade of Concrete $(f_{ck}) = 20\ N/mm^2$

Assume $(F_e) = 410\ N/mm^2$

Step 1:- ~~Bearing Strength~~ Size of the base plate.

(i) Bearing Strength of concrete $= 0.45 f_{ck}$

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

(ii) Area of base plate required.

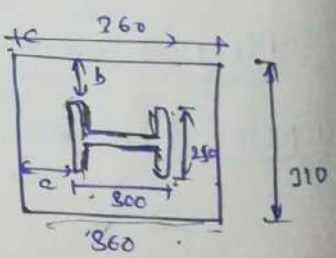
$$A_{baseplate} = \frac{P_u}{0.45 \times f_{ck}}$$

$$A_b = \frac{1000 \times 10^3}{0.45 \times 20} = 111111\ mm^2$$

(iii) Select the size of the plate

choose 360 mm × 310 mm base plate

∴ provided Area = 111600 mm²



$a = \frac{360 - 300}{2} = 30$

$b = \frac{310 - 250}{2}$

Step 2:- Thickness of base plate

(i) Intensity of pressure

$$(w) = \frac{P_u}{\text{Area of base plate}} = \frac{1000 \times 10^3}{360 \times 310}$$

$(w) = 8.96\ N/mm^2$

(ii) minimum thickness of base plate

$$t_s = \sqrt{\frac{2.5 \times w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}} > t_f$$

$$t_s = \left[\frac{2.5 \times 8.99 \times (25^2 - 0.3 \times 2^2) \times 1.1}{250}\right]^{0.5} > 10.6$$

$t_s = 7.88\ mm > 10.6$

choose t_f as 12 mm thickness

Base plate (360 x 310 x 12)

step3 :- connections.

(i) connect base to foundation concrete using #4, 20 mm ϕ and 300 mm long anchor bolts.

(ii) connection of coloumn to base plate using fillet welds

Total length available to welding

$= 2(250 + 250 - 7.6 + 300 - 2 \times 10.6)$

$= 1542.4\ mm$

(iii) Strength of weld $= \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = \boxed{189.37}\ N/mm^2$

Let 's' be the size of the weld, the effective area of weld

Area () stress

eff Area $= 0.7 \times S \times L_e$ * eff stress = load

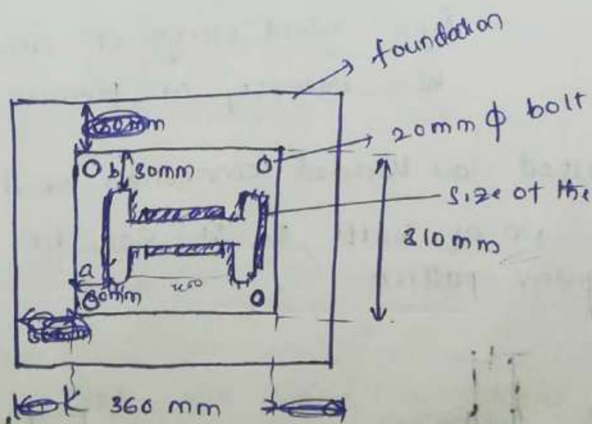
where L_e = eff length

provide S = 6 mm

$0.7 \times 6 \times L_e * 189.37 = Load$

$\therefore L_e = \frac{1000 \times 10^3}{0.7 \times 6 \times 189.37} = 1257.3\ mm$

Diagram :-



Gusset base design :-

Step 1 :- Assume suitable grade of concrete, if not given.

(i) The bearing strength of concrete $= 0.45 \times f_{ck}$.

(ii) The area required of base plate is $(A) = \frac{P_U}{0.45 \times f_{ck}}$

Step 2 :-

(a) The size of the base plate is calculated from the following

ⓐ The gusset plate should not be less than 16 mm

ⓑ Dimension of base plate parllel to the web can be calculated as

L = depth of section $(d) + 2 \times$ (Thickness of gusset plate + leg length of angle + overhang)

(for bolted connection)

(for) Welded connection L = depth of section $(d) + 2$ (thickness of gusset plate + over hang)

and other dimension of plate $B = \frac{\text{Area}(A)}{\text{Length}(L)}$

Step 3 :- The intensity of bearing pressure (w) from the base concrete is calculated as

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

The thickness of base plate is computed by equating the moment at the critical section to the moment of resistance of the gusset at the section

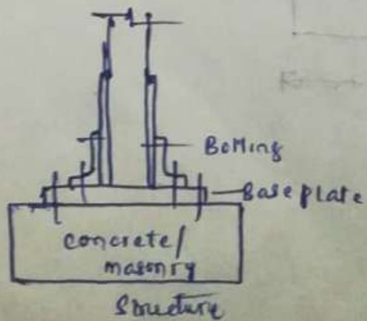
$$t = c_1 \sqrt{\frac{2.7\,w}{f_y}}$$

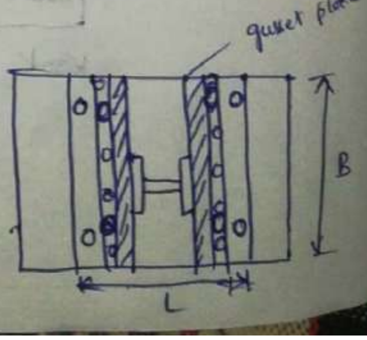
where c_1 = The portion of base plate acting as cantilever in (mm)

f_y = yield stregth of steel

w = intensity of pressure.

Step 4 :- Bolted (or) Welded connection are designs are increase the no of bolts, so it can be provided by in regular pattern.

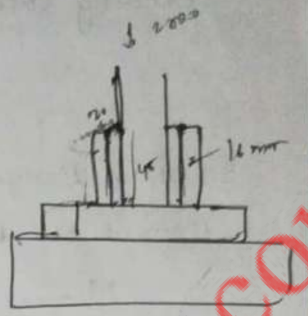




(*) Design a gusseted base for a column ISHB 350 @ 710 N/m with 2 plates of size 450 mm * 20 mm carrying a factored load of 2500 KN, the coloumn is to be supported a concrete pedastral with M20 grade of concrete?

Sol:-

Given data is.



coloum ISHB 350 @ 710 N/m

2 Gusset plates = 450 mm x 20 mm

Factored load = 2500 KN = (Pu)

Step 1:-

$f_{ck} = 20\ N/mm^2$

properties of ISHB 350 @ 710 N/m

$A = 9221\ mm^2$,

$h = 350\ mm$, $t_f = 11.6\ mm$

$b_f = 250\ mm$; $t_w = 10.1\ mm$

Area of base plate $= \frac{P_u}{0.45 f_{ck}} = \frac{2500 \times 10^3}{0.45 \times 20}$

$= 277777\ mm^2$

Total Sectional area(A) $= 9221 + 2(450 \times 20)$

(A) $= 27221\ mm^2$

Assume 16 mm Gusset plates on both sides of the coloum

Length of base plate $= 350 + 2[20 + 16] = 422$

∴ Let us take "50 mm" cover on both sides

∴ Take the base plate "550 x 550 mm"

∴ Provided area $= 302500\ mm^2$

Step 2:- Thickness of base plate

Bearing strength of concrete

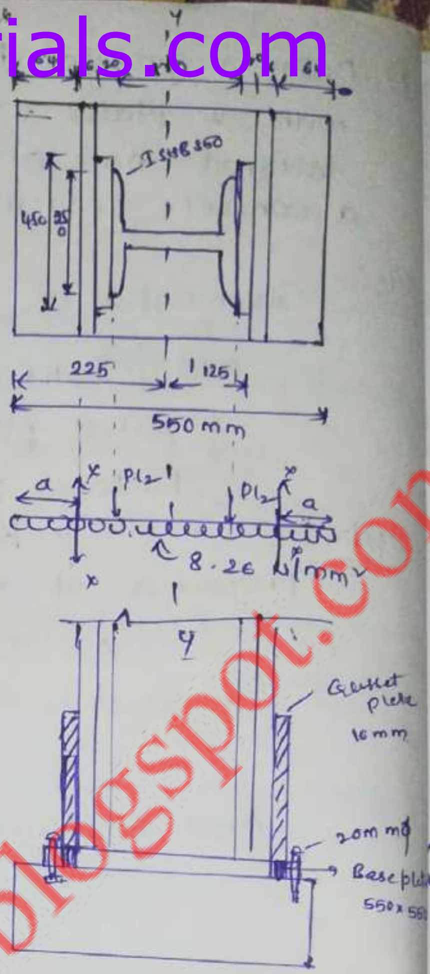
$w = \frac{2500 \times 10^3}{550 \times 550} = 8.26\ N/mm^2$

Moment @ x - x $= \frac{8.26 \times 64^2}{2} = 16916.48$ N-mm

Reaction at $P/2 = \frac{wl}{2} = \frac{8.26 \times 550}{2}$

$= 2271.5\ N.$

moment about @ y-y $= \frac{8.26 \times 275^2}{2} - (2271.5 \times 135)$



Designed Bending moment

$= 85181 \cdot 25$ N-mm

Bending strength $= \frac{f_y}{\gamma_{m0}}$

$= \frac{250}{1 \cdot 1} = 227 \cdot 27$ N/mm²

Equating Design bending moment to the moment of Resistance

Moment of Resistance $= \frac{1.2 f_y Z_e}{\gamma_{m0}}$

$Z_e = \frac{Bt^2}{6} = \frac{1 \times t^2}{6}$

Moment of Resistance $= \frac{1.2 \times 250}{1 \cdot 1} \times \frac{t^2}{6}$

$45.45 t^2 = 85181.25$

∴ Required $\boxed{t = 43.28 \text{ mm}}$

∴ Base plate $= 550 \times 550 \times 50$ mm

Step 3 :- Connections

① connection b/w base plate & foundation

4, 20mm ϕ, 300mm long anchor bolts

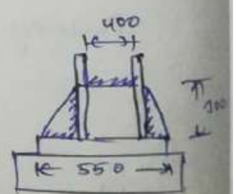
② connecting gusset plate with section

Strength of the weld $= \frac{f_u}{\sqrt{3}\,\gamma_{m0}} = \frac{410}{\sqrt{3} \times 1.1} = 189.97$ N/mm²

Min size of the weld = 5mm (20mm thick)

Take size of the weld = 6mm

Connecting Gusset plate of coloumn :-



Strength of the weld $= \frac{f_u}{\sqrt{3}\,\gamma_{m0}} = \frac{410}{\sqrt{3} \times 1.1}$

$= 189.97$ N/mm²

Size of the weld = 6mm.

Available length of the weld $= 550 + 2(300) = 1150$ mm

Height of the gusset plate 300mm

∴ Load on gusset plates = 50% of axial load

$= \frac{2500}{2} = 1250$ kN

Load on each gusset plate $= \frac{1250}{2} = 625$ kN

Strength ... 625 KN

$\boxed{L_e = 785.1 \text{ mm}}$

Connecting coloumn with base plate

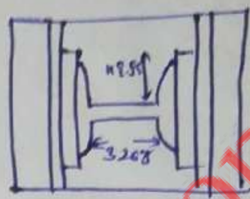
available length of weld $= 2[(3268 + 250 - 10.1) + 200]$

$= 1532.4 \text{ mm}$

Required length of weld $(\sigma) = \frac{P}{A}$

$189.37 = \frac{1250 \times 10^3}{0.7 \times 6 \times L_e}$

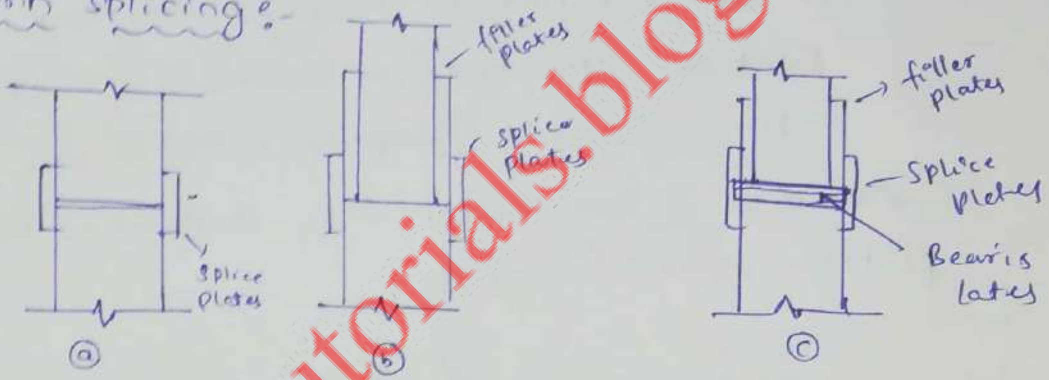
$L_e = 1571.62 \text{ mm}$



We can use extra welding length from the gusset plate connection (350 mm)

Hence we can provide 6mm size of weld through out the circumference of the guesset base coloumn section

* Coloumn splicing:-



* An upper storey coloumn ISHB 300 @ 577 N/m carries a factored load of 1200 KN and factored moment of 12 kN-m it is to be spliced with lower storey coloumn ISHB 400 @ 806 N/m. Design a suitable splice.

Sol:-

Step 1:- design of bearing plate

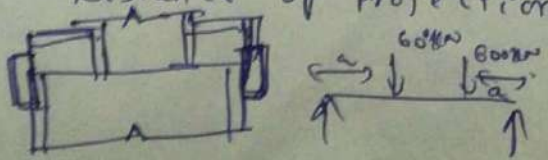
Assuming that the coloumn load is transfered by flanges only.

Load on each flange $= \frac{1200}{2} = 600 \text{ KN}$.

c/c Distance b/w two flanges of coloumn 1 $= 300 - 10.6 = 289.4 \text{ mm}$

c/c distance b/w two flanges of coloumn 2 $= 400 - 12.7 = 387.3 \text{ mm}$

Distance of projection (a) $= \frac{387.3 - 289.4}{2}$



$= 48.95 \text{ mm}$

Design bending stress = $\frac{fy}{1.1}$ = $\frac{250}{1.1}$ = 227.27 N/mm²

equating MoR with Bending moment

MoR = Section modules × stress

$= \frac{bt^2}{6} \times f_{bR} = M$

$= \frac{250 \times t^2 \times 227.27}{6} = 600 \times 48.95 \times 10^3$

$t = 55.69 \approx 56$ mm

Take the bearing plate of 250 × 400 × 56 mm

(ii) Design of splice plate :-

Coloum ends are milled for complete bearing

Hence on splice plates are designed for 50 % of axial load

Load on Splice plate = 1200 × 0.5 = 600 kN

Load on each splice plate = $\frac{600}{2}$ = 300 kN.

Assume 6 mm thick plates, distance b/w splice plates = 400 + 6 = 406 mm