Design of steel and Timber structure:

CE 651

1. Steel structure and their Analysis and Design.

1.1 Introduction to steel structure
1.2 Structural steel and classification
1.3 Method of analysis & Design
1.4 Design process and Basis

A steel structure is an assemblage of a group of member or elements expected to sustain their share of applied forces.

Design of steel structure involves.
a) functional design
b) structural design

a) functional design: The planning of structure for specific purpose such as ventilation, lighting, asthetic view etc.

b) structural design: It consists of proportioning various elements of the building in the most economical manner, so that load acting on it are transferred safely to ground without using excess material.

The members are usually subjected to axial force, bending, on torsion or the combination of all loads. Axial force is either T(+ve) or C(-ve).

Member	force
Tie	Tensile
Column or strut	compression
Beam	bending

Advantage and disadvantage of steel structure.

A) Advantage

i) Highstrength per unit weight, result in smaller section should be used

ii) Posses high ductility so it doesn't fail suddenly

iii) Structural steel are tough, having both strength & durability so during fabrication & erection steel member will not fracture easily

iv) light in weight so easy to handle and transport

v) steel is ultimate recycleable material and has higher selvage value

B: Disadvantage:

i) It is susceptable to corrosion therefore they required frequent painting & maintenance.

ii) for fabrication and erection skill manpower is required

iii). Initial installation is so costly

iv) Maintenance cost is also high.

Use of steel as a building material:

a). Bridge over a tank

b) Highrise building

c) Industrial buildings

d) Transmission towers

2. Structural steel: It is an alloy of iron & carbon. In a standard steel carbon contain is 0.2 to 0.35%, carbon contributes to strength but reduce ductility. structural steel has been classified by (BIS) based, on ultimate or yield strength.

Physical property: a) Modulus of Elasticity $E = 2 \times 10^5 \ N/mm^2$

b) Shear modulus $G = 0.769 \times 10^2 \ N/mm^2$

c) Poisson's ratio $\mu = \begin{cases} 0.3 & \text{elastic} \\ 0.5 & \text{plastic} \end{cases}$

d) Coefficient of thermal expansion : $(\alpha) = 12 \times 10^{-6}/^{\circ}C$

e). unit mass of steel (ρ) = $7850\ kg/m^3$

steel as a structural material :

structural steel is a material used for steel construction, which is formed with a specific shape following standards of chemical composition and strength. They can also be defined as hot rolled products, with a X-section of special form like angle, channel and beam/joints. Steel has always bee more preffered to concrete because steel offer better tension and compression thus result in light constuction. Usually steel uses three dimensional trusses hence making it larger than its concrete counter part.

Technique to produce wide range of structure and shape.

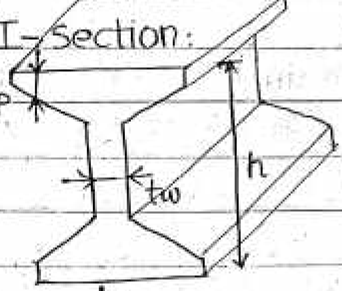
a) High precision stress analysis

b) computerized "

c) Innovative joints.

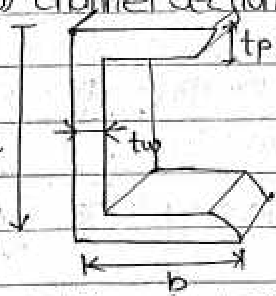
Types of steel structure :

a) I-section:



Uses:- Beam and column

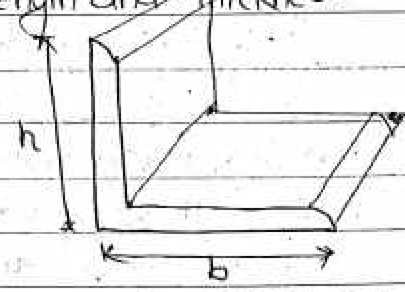
b) Channel section:



uses: Beam / column (heavy)

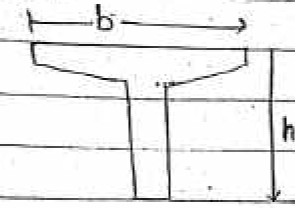
c) Angle section :

length and thickness



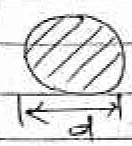
uses- compression and Tension member

d) T-section:

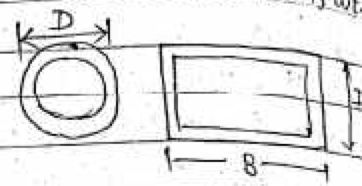


uses:- compression and tension member of door & window.

e) Rolled steel Bars: round bar designated by diameter



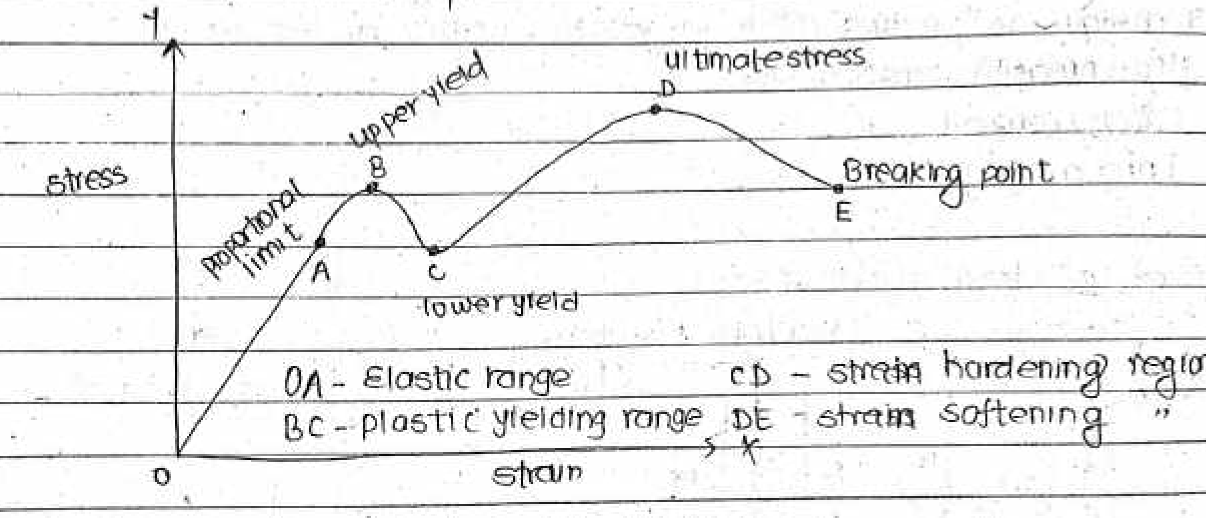
f) Rolled steel, Tubular section: designated by outside diameter and self wt.



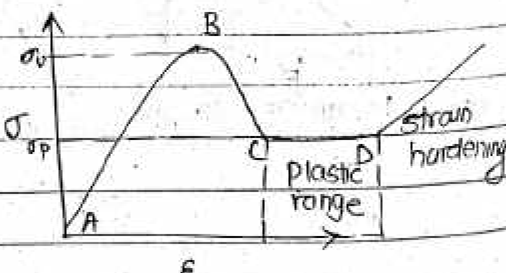
uses: compression member of roof & truss.

Loads: The force that act on a structure are called loads. for safe design of structure it is essential to have a knowledge of various material or man made or combined load acting on it.

Stress strain relationship:



since, plastic range is sufficiently large and it seems resonable to extent it without limit that is to ignore the effect of strain hardening.



strain hardening : or work hardening.

It is the strengthening of a metal by plastic deformation, which occurs because of dislocation movements and dislocation generation within the crystal structure of materials. Many non-brittle metals having high melting point can be strengthened by this effect.

Design philosophy:

=> Design of steel structure consists of design of steel members and their connections, so that they can safely and economically resist and transfer applied load to ground. The design process begins with selection of trial section & checking its safety.

The design are based on:-

a) attainment of full yielding
b) Tensile strength
c) Critical buckling
d) Max. deflection permitted
e) stress concentration
f) fatigue
g. Brittle fracture.

Philosophy:

a) Working stress method
b) Limit state method
c) Ultimate stress method.

i) Ultimate tensile strength: It is max. stress that the material can withstand while being stretched or pulled before failure

ii) Yield strength:- stress at which stress-strain curve for axial loading deviates the strain of 0.2% from linear elastic to non-linear curve.

CH:2

Working stress Method:

a) It is an elastic method of design. According to this method, member are designed on the basis of working stress and those will never exceed the permissible stress.

A permissible stress is defined as the ratio of yield stress according to factor of safety.

$$\text{Permissible stress} = \frac{\text{yield stress}}{\text{FOS}}$$

Basic assumptions:

a) Material is homogenous and isotropic

b) Material is assumed to be linearly elastic.

c) It is a deterministic approach:

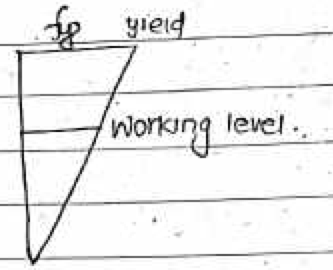
suppose $f_{ck} = 20\ N/mm^2$ at least $20\ N/mm^2$ हुनुपर्छ।

d). Working or service load is not modified.

e). Permissible stress $= \frac{\text{yield stress}}{\text{FOS}}$ (obtained by modification)

f. FOS : in steel: 1.6 to 1.92 avg. value 1.78

in concrete : FOS = 3



Permissible or allowable stress

a) Tension / compression $= 0.6 f_y$

b) Bending $= 0.66 f_y$

c) Shear $= 0.45 f_y$

limitation:- i) failure load is factor of safety x working load, which is not true

ii) actually it is more because a material can resist load after yield occurs at failure

iii) In structure just formation of plastic hinge is not the collapse criteria, still can resist more load till collapse mechanism is formed

iv) It gives uneconomical section

v) deals only elastic behaviour.

CH:3

advantage:

This method is simple and resonably reliable.

working stress is low, serviceability requirement are satisfied

Limit state method:

It is similar to plastic design which consider most critical limit state of strength and serviceability.

⇒ The acceptable limit for the safety and serviceability requirements before the failure occurs is called limit state:

The section design should satisfy serviceability requirements such as limitation of deflection & vibration and shouldn't collapse under the accidental loads.

There are two limit states:

a) Limit state of strength:

for checking the strength and stability of structure, the load are multiplied by relevant load factor (γ_f) Table: 4. IS: 800 - 2007.

The modified load are called factor loads account for uncertainties involved in estimating the magnitude of dead and live loads.

The design strength of members or its connections are determined by dividing ultimate strength w.r.to partial safety factor. (γ_m) for materials: IS 800-2007 table 5.

b) Limit state of serviceability:

It is the limit state beyond which the service criteria such as deflection. vibration, repairable damage due to fatigue, corrosion, fire resistane.

load factor (γ_f) is used for all load to check serviceability requirements:

Codes for loads:

IS 875 - Part 1 Dead load

IS 875 - Part 2 or 4 live load

IS 875 - part 3. wind load

IS 1893 earthquake load.

Mechanical property of steel:

a) Elasticity b) Plasticity c) Ductility d) Brittleness e) Hardness
f) fatigue g) creep h) Malleability. i) slow deformation

Malleability: Property of material due to which it can be rolled into thin sheet of metal without toughness. It can be stretch, bend, twisted under a high stress before failure.

Assumptions of limit state design.

a) Material is homogenous and isotropic

b) Material behaviour is elastoplastic

c) It is a probabilistic approach

d) working load or service load is modified by partial safety factor for load: PSF = 1.5

e) Design load/action $Q(d) = \gamma_f \times Q$

γ_f - partial safety factor

Q - Service/working load.

f) Design strength is obtained by modifying (yeild to ultimate) stress by PSF

ie, Designed strength $(S_d) = S/\gamma_m$

γ_m - partial safety factor for material

S - yield to ultimate stress

for yield :- $\gamma_{m0} = 1.1$

$\gamma_{m0} = 1.1$ (buckling)

Difference between :

WSM	LSM.
a) It is a traditional method	a) It is a modern method
b) It is not economical	b) It is more economical
c) It is deterministic approach	c) It is probabilistic approach
d). Material is linearly elastic	d) Material is elastoplastic
e) Working or service load is not modified and reduced	e) Modified by partial safety factor
f) Designed stress is modified by FOS	f) Designed stress is modified by partial safety factor
g. Simple and reliable method calculation	g. Design is more complex as compared to WSM.

Safety and serviceability requirement of structure

The design of steel structure should satisfy the safety and serviceability requirement regarding strength, serviceability, brittle fracture, fatigue, fire and durability even during worst combination of load. Structure should meet following requirements:-

a) Remain fit and adequate reliable and able to sustain all loads
b) Have adequate durability under normal maintenance
c) Do not suffer overall damage or collapse disproportionality under accidental loads like explosions, impact, human error.

The catastrophic damage can be limited by:-
a) avoiding the exposure to hazard
b) choosing structural forms, layout & details during designing
c) Choosing suitable materials, designing and detailing procedure.

Design process and Basis for Design:

It is a process to decide shape, size and connection details of both concrete and reinforcement so that structure being designed will perform satisfactory during its intended life, with an required degree of safety.

⇒ Basis of Design of structure

a) Sustain all loads expected on it

b) Sustain deformation during and after construction

c) Should have adequate durability

d) Should " " resistance to misuse and fire

e. Structure should be stable and have alternate load path to prevent overall collapse under accidental loadings.

Grade of steel:

Steel grade classify various steels by their composition & physical properties.

As per IS 2062: 2006. 9 grades of steels are designated as E165, E250 A, E250 B, E250 C, E300, E350, E410, E450(D), E450(E).

Numerical value - yield strength in Mpa.

These grades of steel have their own chemical composition & mechanical properties.

Classification of structural steel as per IS.

a) Hot rolled section:- various products made using this methods are plate, strip, shape & section, flat, bars as per IS:

1. Beam: (ISJB, ISLB, ISMB, ISWB)
2. Column (ISSC, ISHB)
3. Parallel flange Beam / column - ISNPB, ISWPB
4. Channel: ISJC, ISLC, ISMC, ISMCP
5. Angle. ISA
6. T section. ISNT, ISDT, ISLT, ISMT, ISHT
7. Tubular
8. Rectangular / Hollow

cold formed light-gauge:

cold formed light gauge are used where thicker hot rolled section become uneconomical in small building subjected to lighter load. They are produced from steel strips, generally not thicker than 8mm. for mass production they are produced by cold rolling. They are avilable in the form of equal angle, unequal angle, channel, hot section, denoted by depth x width x thickness.

* Classification of steel according to moment rotation capacity.

plastic X-section.	compact X-section	semicompact	Slender X section
develop full plastic moment allowed suffrient rotation redistribution of moment take place until failure mechonism occured	develop full plastic moment local buckling prevent required rotation	stress in extreme fiber limited to yield local buckling prevent development of full plastic moment	yield in the extreme fibre can't be attended Premature local buckling occurs in the elastic range.

advantages of steel structures:

Design in tension, compression, Bending and shear.

while designing a structural member subjected to Tension, compression, Bending & shear, allowable stress are as follows:-

tensile stress (σ_{at}) = $0.6 f_y$

compressive, (σ_{ac}) = λf_y

λ = Slenderness ratio

bearing stress = 185 Mpa

max. bending tension / compression = $0.66 f_y$

max. Shear stress (τ_{vm}) = 0.42 to $0.45 \times f_y$

Different limit state for Steel design:

A) Limit state of strength:

a) loss of equilibrium of whole or part of structure

b) loss of stability

c) failure by excess of deformation

d) fracture by fatigue

e) Brittle fracture

B) limit state of serviceability.

i) Deformation & deflection affect the appearance of structure

ii) vibration cause loss of functional effectiveness

iii) repairable damage or crack due to fatigue

iv) Corrosion

v) fire

CH: 4 Structural Steel Connections

Connection :

a) Permanent connection (i) Welded ✓

(ii) Riveted (Introduction)

b) Temporary connection (i) Bolted ✓

(ii) Nailed ×

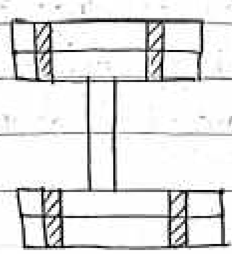
Various elements of steel structure like Tension, compression and flexural members are connected fastners or connectors.

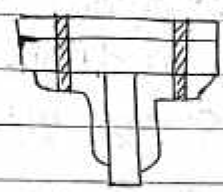
Needs of connection

a) To transmit load from one member to another

b) assemblage of Individual member to form composite section.

eg connecting Plates, angles, channels, I section



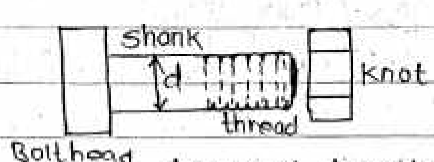


Tsection angle and plates

c) connections of two length of a member to make up a required length.

a) Bolted connection :-

A bolt may be defined as a metal pin with a thread at one end, a shank threaded at other end to receive a knot.



d-nominal diameter

Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the knot at threaded ends.

Based on load Transfer:

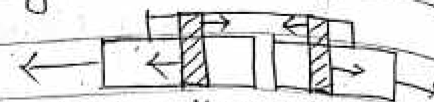
Types of Bolt — features

a) Bearing type bolt: - load transfer by shearing action
- Suitable for light structure
- not suitable for vibration, fatigue
- suitable for static loading

(Designation) Nominal diameter: 12, 16, 20, 22, 24 ---

designated as M12, M16, M20, M22 ----

eg: Slip critical connection



load transfer through bolt

b) High strength friction grip bolt (HSFG) :- Transfer load by friction
Suitable for heavy structure
Suitable for vibration and fatigue
Suitable for dynamic & static loading

(through plate)

It is made from high strength steel rod and surface is finished. Bolt are tightened by using callibrated wrenches and knots are provided by clamping devices. The shearing load is restricted by frictional force bet^{n} shank and member and washers.

Nominal diameter: 16, 20, 24, 30, 36.

Property class of bolt (Pr. Cl.)

The ratio of net tensile area at thread to nominal plain shank area of bolt is 0.78.

$$A_{nb} = 0.78 A_{sb}$$

An - area at root of threads / stress / proof area.

Commont bolt is black bolt of class 4.6 and $\overset{9150}{\wedge}$ 8.8 are avilable.

It is represented as $X \cdot Y$

$X = \frac{1}{10}^{th}$ of ultimate tensile strength of bolt in kgf/mm^2

Y = fraction of $\frac{1}{10}^{th}$ of ultimate tensile strength as yield strength of bolt.

3.6, 4.6, 4.8, 5.6 — Bearing type

9.8, 10.8, 12.9 (HSFG)

x.y

for eg:- for Pr.Cl: 4.6

ultimate tensile strength of bolt $f_{ub} = 40 kgf/mm^2$

$= 40 \times 9.81 \ N/mm^2$

$= 392.4 \ N/mm^2$

$\approx 400 \ N/mm^2$

so, Yield tensile strength of bolt $= (f_{yb}) = \frac{y}{(x+y)} \cdot f_{ub}$

$= \frac{6}{(6+4)} \times 400$

$= \frac{6}{10} \times 400$

$= 240 \ N/mm^2$

Specification for spacing:

P should not be less than 2.5d ($p \nless 2.5d$)

d - nominal diameter of bolt

P is $\ngtr$ 16t or 200mm

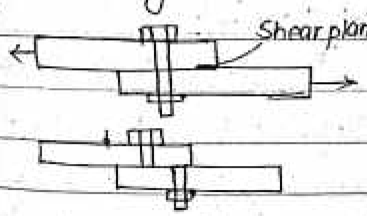
$$\left.\begin{matrix} p \ngtr 16t \\ 200 \ mm \end{matrix}\right\} \text{tension}$$

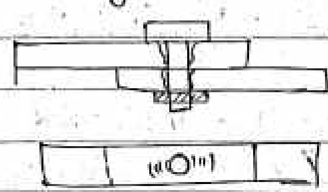
t - thickness of thinner plate.

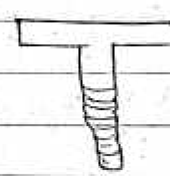
$$\left.\begin{matrix} p \ngtr 12t \\ 200mm \end{matrix}\right\} \text{compression}$$

Plate in a joint made with Bolted joint failure under 3 causes:

a) shearing failure (b) Bearing failure (c) Tension failure





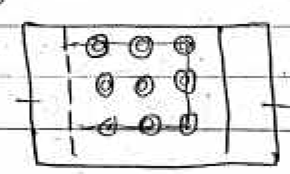


(b) crushing or tearing

Block shear failure

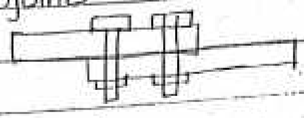
(a) yielding of plate

(c)



Type of bolted joints

a) Lapjoints



(b) Butt joints (end-to end connection)

Design parameters and design specification:

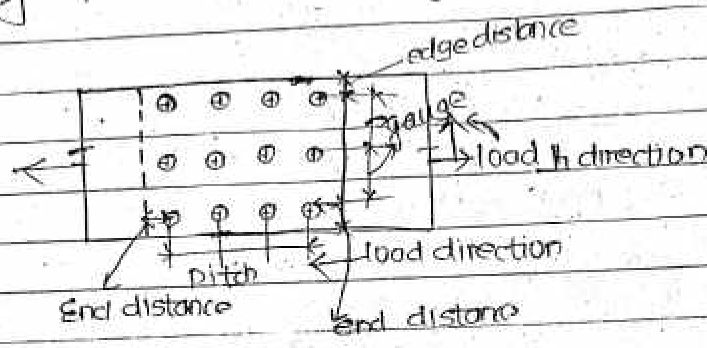
on the Basis of Type of force

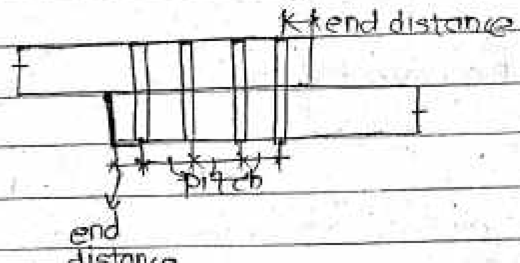
Shear – lap and butt joints

Tension → hanger connection

combined (τ+T) = When inclined member is to be connected to through column through bracket

eg: connection of bracing.





Specification: 10:2.2 clause page 73/74

1. Diameter of hole (do/dn) = Nominal diameter (d) + clearance.

(table: 19 page (73) IS-800

2. Pitch of bolt shouldn't be less than 2.5d (d-nominal diameter)

Pitch $\nless$ 2.5d or Pitch $\geq$ 2.5d

3. Pitch shouldn't be greater than 32t or 300mm whichever is greater in general

P $\ngtr$ 16t or 200 mm (T +ve)

P $\ngtr$ 12t or 200 mm (C –ve)

Fe - 410
$f_u = 410$ Mpa
$f_y = 250$ Mpa

t = thickness of thinner plate.

4. End distance shouldn't be less than $1.5d_0$, in case of machine flame cut
$1.7d_0$ in case of hand flame cut

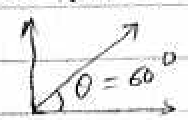
5. Max. end distance shouldn't be greater than $12 \times t\varepsilon_0$
$\varepsilon_0 = \sqrt{\frac{250}{f_y}}$

Design strength of bolt in :-

a) shear strength of bolt:

Nominal shear strength of bolt: Page: 75 Clause 10.3.3

$(V_{nsb}) = \frac{f_{ub}}{\sqrt{3}} \times (\eta_n A_{nb} + \eta_s A_{sb})$



f_{ub} = ultimate tensile strength of bolt

n_n = no. of shear planes intercepting with threaded portion of bolt

A_{sb} = are of shank or non threaded portion $(\pi d^2/4)$

n_s = no. of shear planes intercepting with non. threaded portion of bolt

A_{nb} = area of threaded portion (Generally $0.78 A_{sb}$).

b) Design shear strength of bolt:

$(V_{dsb}) = \frac{V_{nsb}}{\gamma_{mb}}$, γ_{mb} - partial safety factor for bolt:
Table: 5, Page: 30

B) Bearing capacity of bolt: Page 75, 10.3.4

a) nominal bearing capacity of bolt:

$(V_{npb}) = 2.5 k_b\, d\, t\, (f_u \text{ or } f_{ub})$

K_b is lesser of

$\frac{e}{3d_0}, \frac{P}{3d_0} - 0.25, 1, \frac{f_{ub} \leftarrow \text{bolt}}{f_u \leftarrow \text{plate}}$

p - pitch distance

e - end distance

d - nominal diameter

t - thickness of thinner plate.

d_0 = diameter of hole

f_{ub} = Ultimate tensile strength of bolt

b) Design bearing strength of bolt: (V_{dpb}) $= \frac{V_{npb}}{\gamma_{mb}}$

c) Tensile strength of bolt: (10.3.5) / Page 76

i) Nominal tensile strength of bolt: $= 0.9 f_{ub} A_{nb} \leq f_{yb} A_{sb} * \frac{\gamma_{mb}}{\gamma_{mo}}$

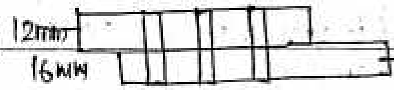
γ_{mo} = partial safety factor for yielding

Table: 5 page: 30.

ii) Design tensile strength of bolt: $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$

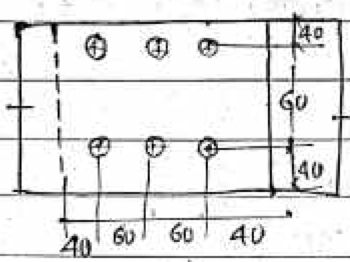
Numerical

Calculate bolt value, in following cases:

a) 

all dimension in mm

Use M20 bolt Pr. Cl. 4.6



Nominal diameter (d) = 20 mm

hole diameter (d_0) = 20 + 2 = 22 mm

page: 73, table-19

ultimate tensile strength of bolt f_{ub} = 400 Mpa

critical: threaded portion

yield " " " " f_{yb} = 240 Mpa

pitch distance p = 60 mm

Design:

end distance e = 40 mm

a) Shear strength of bolt: $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi \times d^2}{4}\right) = 45.27$ KN.

assuming: shear plane intercept with threaded portion of bolt

$n_n = 1$

$n_s = 0$

Page 30, table: 5 $\gamma_{mb} = 1.25$

b) Design Bearing capacity:

Kb is at least,

$$\frac{e}{3d_0} \quad \frac{p}{3d_0} - 0.25 \quad 1 \quad \frac{f_{ub}}{f_u}$$

$$\Rightarrow \frac{40}{3\times22}, \quad \frac{60}{3\times22} - 0.25, 1, \quad \frac{400}{410}$$

$$\boxed{0.606} \quad 0.659, \; 1, \; 0.976$$

$\therefore K_b = 0.606$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

(least among them)

$$= \frac{2.5\, K_b\, d\, t\, (f_u \text{ or } f_{ub})}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.606 \times 12 \times 400}{1.25}$$

$$= 116.352 \text{ KN}$$

c) Tensile ~~bear~~ strength of bolt:

Design tensile strength $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$

$$\therefore T_{nb} = 0.9\, f_{ub} A_{nb} \le f_{yb} A_{sb} \times \frac{\gamma_{mb}}{\gamma_{m0}}$$

$$= 0.9 \times 400 \times 0.78 \times \frac{\pi d^2}{4} \le 240 \times \frac{\pi d^2}{4} \times \frac{1.25}{1.1}$$

$\gamma_{m0} = 1.1$

$\gamma_{mb} = 1.25$

$d = 20\text{mm}$

$$= 88.216 \le 85.679 \text{ KN}$$

least among them

$$T_{db} = \frac{85.679}{1.25}$$

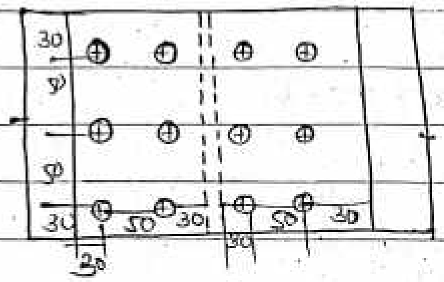
$$= 68.5 \text{ KN.}$$

Bolt value = min. of shear, bearing and Tensile capacity of bolt
= 45. 27 KN

Bending आएमा Tensile strength पनि check गर्ने!

Connection Design:

(B)



$d = 16mm$

$d_0 = 16+2 = 18mm.$

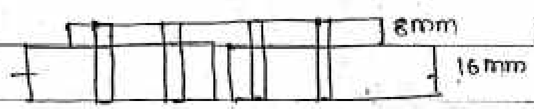
clearance 2mm

$e = 30mm$

$p = 50.mm$

$f_{ub} = 400\ N/mm^2$

$f_{yb} = 240\ N/mm^2$



Use M16 bolt of Pr. Cl 4.6

Now,

(I) Design shear capacity of bolt

$$(V_{dsb}) = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

assume, shear plane lies on thread

$$n_n = 1, n_s = 0$$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \frac{\pi}{4} \times 16^2\right)$$

$$= 28.97\ KN$$

(II) Bearing capacity

k_b is least of

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, 1, \frac{f_{ub}}{f_u}$$

$$\text{or, } \frac{30}{3 \times 18}, \frac{50}{3 \times 18} - 0.25, 1, \frac{400}{410}$$

$$\therefore 0.556, 0.676, 1, 0.976$$

so, $k_b = 0.556$

$$(V_{dpb}) = \frac{2.5 k_b d t (f_u \text{ or } f_{ub})}{\gamma_{mb}} = \frac{2.5 \times 0.556 \times 16 \times 8 \times 400}{1.25}$$

$$= 56.889\ KN$$

Tensile:

$(T_{db}) = \dfrac{T_{nb}}{\gamma_{mb}}$

Since,

$T_{nb} = 0.9 f_{ub} A_{nb} \le f_{yb} A_{sb} \dfrac{\gamma_{mb}}{\gamma_{m0}}$

$= 0.9 \times 400 \times 0.78 \times \dfrac{\pi \times 16^2}{4} \le 240 \times \dfrac{\pi \times 16^2}{4} \times \dfrac{1.25}{1.10}$

$= 56.458 \le 54.835$

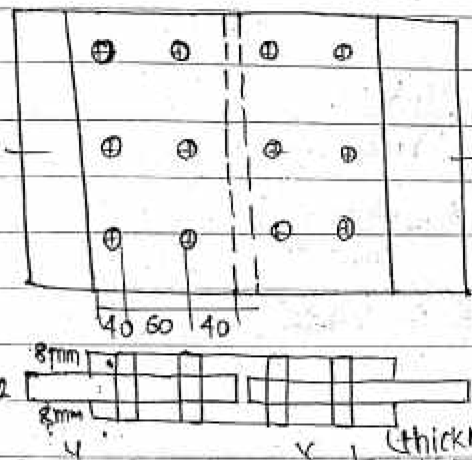
So, $T_{db} = \dfrac{54.835}{1.25}$

$= 43.868$ KN

Bolt value = least among V_{dsb}, V_{dpb}, T_{db}

$= 28.97$ KN

(c)



1 shear plane in threaded region
1 shear " " un "
$N_n = 1$, $N_s = 1$,

use M20 bolt of pr. cl => 4.6

Now:

$d = 20$ mm

Clearance $= 2$ mm

$d_0 = 22$ mm

$f_{ub} = 400$ Mpa

$f_{yb} = 240$ Mpa

(thickness)

एउटै function गर्नेको add गर्ने

$t_1 = 8 + 8 = 16$ mm

$t_2 = 12$ mm (least)

2. in case of double cover butt joint, eccentricity force doesn't exists so bending is eliminated but not in lap joints.

a) Designed shear strength $= V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \frac{\pi \times 20^2}{4} + 1 \times \frac{\pi \times 20^2}{4}\right)$

$= 103.314$ KN

b) Designed Bearing capacity (V_{dpb}) $= \frac{2.5\, k_b\, d\, t\, (f_u \text{ or } f_{ub})}{\gamma_{mb}}$

for k_b

$\frac{e}{3d_0}, \frac{P}{3d_0} - 0.25, 1, \frac{f_{ub}}{f_u}$

or, $\frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, 1, \frac{400}{410}$

or, $0.606, 0.659, 1, 0.975$

so, $(V_{dpb}) = \frac{2.5 \times 0.606 \times 16 \times 12 \times 400}{1.25}$

$= 116.352$ KN

c) Design tensile strength $(T_{db}) = \frac{T_{nb}}{\gamma_{mb}}$

$T_{nb} = 0.9 f_{ub} A_{nb} \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{m0}}$

$= 0.9 \times 400 \times 0.78 \times \frac{\pi \times 20^2}{4} \leq 240 \times \frac{\pi \times 20^2}{4} \times \frac{1.25}{1.1}$

$= 88.216 \leq 85.679$

So, $T_{db} = \frac{85.679}{1.25} = 68.544$ KN

Hence, Bolt value $= 68.544$ KN

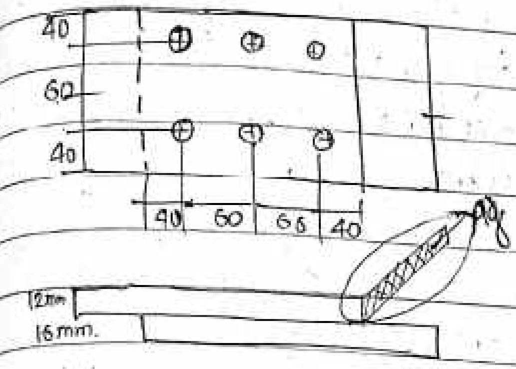
Note: for snug tight bolt use butt joints because,

1. for Double cover butt joints → Total shear force to be transmitted by member is split into 2 parts, and force acts on each half.

But for lap, there is only one shear plane on which force acts and shear-carrying of a bolt in double cover butt joint is double that of a bolt in lap joints.

Calculate bolt value, strength of connection and efficiency of connection for following cases:

for question (a).



Given, M20 bolt Prcl. 4.6

$d = 20mm$

$d_0 = 22mm$

$e = 40mm$

$P = 60mm$

from above solⁿ: Bolt value = 45.27 KN.

(i) Shear capacity of bolt $(V_{dsb}) = 45.27 KN$

(ii) Bearing capacity .. $(V_{dpb}) = 116.352 KN$.

(iii) Yielding capacity of plate: Page 32. 6.2

$$T_{dg} = \frac{f_y \cdot A_g}{\gamma_{m0}} \qquad f_y = 2L$$

A_g - gross area of plate having least thickness.

If not stated Fe 410, $t = 12mm < 20$

$\therefore f_u = 410$ Mpa

$\therefore f_y = 250$ Mpa

$$\therefore T_{dg} = \frac{250 \times (140 \times 12)}{1.10}$$

$= 381.82$ KN

(iv) Rupture strength due to rupture of critical section

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

γ_{m1} = partial safety factor for failure at ultimate stress

f_u = ultimate stress

A_n = net effective are of member

Note: Force mechanism: - Connection

Bearing type: bolt bear against hole to transfer load - slip type

friction "

$$A_n = \left[b - nd_n + \sum_i \frac{P_{si}^2}{4g_i}\right] \times t$$

net area, tear area.

n - no. of bolt in tear zone

d_n = diameter of bolt hole

t = thickness of plate

b = width

$\therefore A_n = (140 - 2 \times 22) \times 12$

$\therefore T_{dn} = \dfrac{0.9 \times (140-44) \times 12 \times 410}{1.25}$

$= 340.070 \text{ kN}$

(V) Total Shearing capacity of connection = no. of bolt x Shear strength of single bolt

$= 6 \times 45.27$

$= 271.62 \text{ kN}$

(VI) total Bearing capacity of connection $= 6 \times 116.352$

$= 698.112 \text{ kN}$

Hence, strength of connection = least of (III), (IV), (V), & (VI)

$= 271.62 \text{ kN}$

For, efficiency of connection:

strength of plate before drilling $= \dfrac{0.9 A_n f_u}{\gamma_{ms}}$

$= \dfrac{0.9 \times 140 \times 12 \times 410}{1.25}$

$= 495.936 \text{ kN}$

% efficiency = $\dfrac{\text{strength of connection after drill}}{\text{strength of plate before drill}} = \dfrac{271.62}{495.936} \times 100\%$

$= 54.769\%$

Connection Design:

(B) single cover buttjoints:

B. shear strength of bolt (V_{dsb}) = 28.97 KN

Bearing " " " (V_{dpb}) = 56.889 KN

$d = 16mm$

$d_0 = 18mm$

$e = 30 mm$

$P = 50mm$

$f_{ub} = 400 N/mm^2$

$f_{yb} = 240 N/mm^2$

Bolt value = 28.97 KN

Now, yielding capacity of plate:

$$T_{dg} = \frac{f_y \cdot A_g}{\gamma_{m0}}$$

A_g = gross area of least thickness

Fe = 410, $f_u = 410$, $f_y = 250$ Mp since, $t < 20 mm$

$$T_{dg} = \frac{250 \times (160 \times 8)}{1.10}$$

$$= 290.90 KN$$

Rupture strength due to critical section

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$A_n = [b - n d_n] \times t$$

$$= [160 - 3 \times 18) \times 8$$

$$= 848$$

So, $$T_{dn} = \frac{0.9 \times 848 \times 410}{1.25}$$

$$= 250.33 KN$$

$\rightarrow$ no of bolt given to 1 plate

Total shearing capacity $= 6 \times 28.97$
$= 173.82$ KN

Total bearing capacity $= 6 \times 56.889$ KN
$= 341.334$ KN

$\therefore$ strength of connection $= 173.82$ KN

for efficiency of connection,

Strength of plate before drilling $= \frac{0.9 A_n f_y}{\gamma_{m1}}$

$= \frac{0.9 \times 160 \times 8 \times 410}{1.25}$

$= 377.856$ KN

% efficiency $= \frac{173.82}{377.856}$

$= 46\%$

(C) Calculate η:

$V_{dsb} = 103.314$ KN

$V_{dpb} = 116.352$ "

Bolt value $= 68.544$ KN

$d = 20$ mm

$d_0 = 22$ mm

for Fe 410

$f_u = 410$ MPa

$f_y = 250$ MPa $t < 20$ mm

Then,

yield capacity $(T_{dg}) = \frac{f_y A_g}{\gamma_{m0}}$

$= \frac{250 \times 200 \times 12}{1.10}$

$= \cancel{727.27 \text{ KN}}$

$= 545.45$ KN

Design Tearing Rupture strength

$T_{dn} = \frac{0.9 A_n f_y}{\gamma_{m1}}$

$= \frac{0.9 \times (200 - 3 \times 22) \times 12 \times 410}{1.25}$

$= 474.68$ KN

Total shearing capacity $= 6 \times 103.314$
$= 619.884$ kN

Total Bearing $= 6 \times 116.352$
$= 698.112$ kN

$\therefore$ strength of connection $= 474.68$

Strength before drilling of plate $= \frac{0.9 A_n f_u}{\gamma_{m_1}}$
$= 708.480$

$\therefore \eta = \underline{67\%}$

Design bolted connection

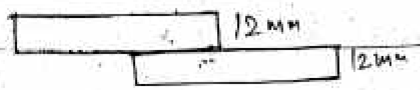
To connect two members (200 x 12) mm, carrying an axial load of 200 kN using M20 bolt of Pr. Cl. 4.6 in lap connection.

Soln:-

$d = 20\,mm$ — Ultimate / design / factor load : no modification.

$d_0 = 22\,mm$

$f_{ub} = 400\,Mpa$



$f_u = 410\,Mpa$

$f_y = 240\,Mpa$

axial load (P) = 200 kN. (working load) / service load:)

∴ Design load = Partial safety factor × working load

$= 1.5 \times 200$

$= 300\,kN$

$n_s = 0$

$n_n = 1$

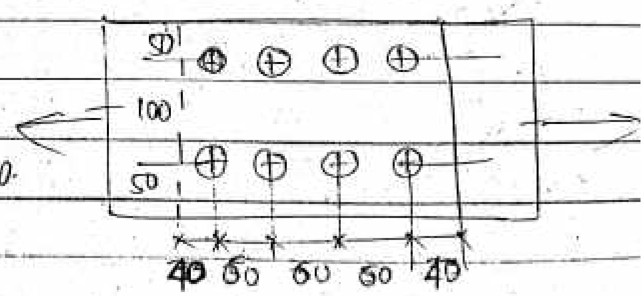
① shear strength strength of bolt $(V_{dsb}) = \frac{f_{ub}}{\sqrt{3}\,\gamma_m}(n_n A_{nb} + n_s A_{sb})$

$= \frac{400}{\sqrt{3} \times 1.25} \times \left(1 \times 0.78 \times \pi \times \frac{20^2}{4}\right)$

$= 45.27\,kN$

No. of Bolts $= \frac{\text{Design load}}{\text{Shear strength}}$

$= \frac{300}{45.27}$



$= 6.69$ near even no.

$= 8$ bolts

Now,

pitch $(p) = 2.5d = 2.5 \times 20\,mm$

$= 50\,mm$

End distance(e) $= 1.5 \times d_0$

$= 1.5 \times 22$

$= 33\,mm$

Bearing capacity :

$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, 1, \frac{f_{ub}}{f_u}$

$\frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, 1, \frac{400}{410}$

min यति चुन पर्छ | 0.606, 0.65, 1, 0.97

$\therefore k_b = 0.60$

$\therefore V_{pb} = \dfrac{2.5 \times 0.606 \times 20 \times 12 \times 400}{1.25}$

$= 115.352$ kN

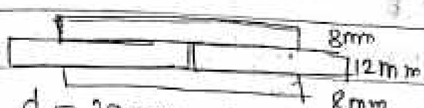
Total bearing capacity $= 8 \times 116.352$

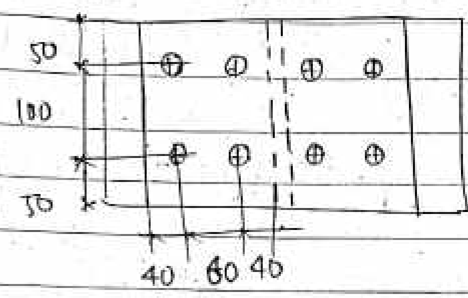
$= 930.816$ kN > 300 KN

same eqn with coveplate of thickness 8mm in

① single cover butt joint

② Double " " "





$d = 20$ mm

$d_o = 22$ mm

$f_{ub} = 400$ Mpa

$f_u = 410$ Mpa

axial load $= 200$ kN

design load $= 1.5 \times 200$

$= 300$ kN

$n_n = 1$

$n_s = 1$

① shear strength of bolt $= (V_{dsb}) = \dfrac{f_{ub}}{\sqrt{3} \times \gamma_m}\left(n_n A_{nb} + n_s A_{sb}\right)$

$= \dfrac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \dfrac{\pi d^2}{4} + 1 \times \dfrac{\pi d^2}{4}\right)$

$= \dfrac{400}{\sqrt{3} \times 1.25} \times 1.78 \times \dfrac{\pi \times 20^2}{4}$

$= 103.314$ kN

$\therefore$ No of Bolts $= \dfrac{300}{103.319}$

$= 2.90$

$= 4$

pitch $= 2.5d = 50$ mm

end distance $= 1.5 d_o = 33$ mm

for Bearing capacity k_b

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, 1, \frac{f_u}{f_{ub}}$$

$$= 0.606 \quad 0.650, 1, 0.97$$

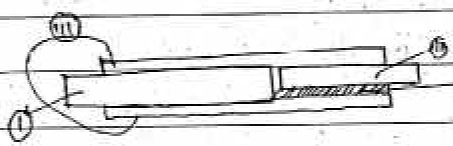
$$\therefore V_{dpb} = \frac{2.5 \times 0.606 \times 12 \times 20 \times 400}{1.25}$$

$$= 116.352 \text{ KN}$$

$$\text{Total bearing capacity} = 4 \times 116.352 \times$$

$$= 465.408 \text{ KN} > 360 \text{ KN}$$

for Buttjoints having different thickness of plate



page: 75: 10.3.3.3

packing plate (not for load)

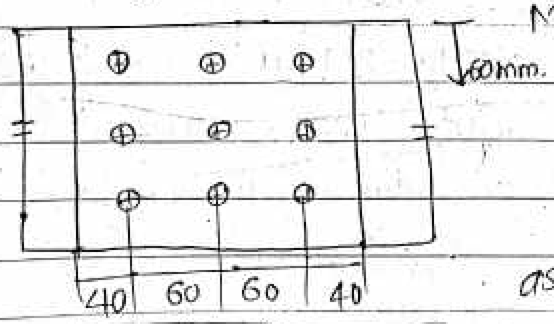
$$\beta_{pk} = 1 - 0.0125 t_{pk}$$

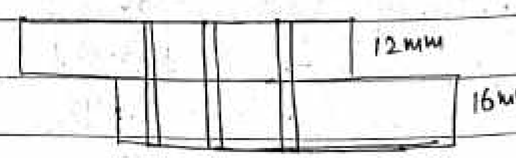
t_{pk} = thickness of packing plate

$$\text{design shear strength} = \beta_{pk} \times \frac{f_{ub}}{\sqrt{3}\,\gamma_m} (n_n A_{nb} + n_s A_{sb})$$

calculate strength of connection, efficiency of connection per pitch

M_{20} bolt of Pr.Cl. 4.6





assume. P = Gauge.

For, Tearing strength

$$T_{dg} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$A_n = (\text{Pitch shift} - n d_h) \times t$$

n – for shifted pitch how many bolt lies.

Given:-

M20 bolt

$d = 20mm$

$d_o = d_h = 22mm$

for Pr. Cl: 4.6

$f_u = 410$ Mpa

$f_{yb} = 240$ Mpa

$f_{ub} = 400$ "

(a) for design shear strength per pitch length

$$V_{dsb} = 1 \times \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times (A_{nb} n_n + A_{sb} n_s)$$

$$= \frac{400}{\sqrt{3} \times 1.25} \times 1 \times 0.78 \times \frac{\pi d^2}{4}$$

$$= 45.264 \text{ KN}$$

(b) for design bearing capacity per pitch length.

$$V_{dpb} = \frac{1 \times 2.5\, K_b d t (f_u \text{ or } f_{ub})}{1.25}$$

$$K_b = \frac{e}{3d_o} = 0.606$$

$$= \frac{1 \times 2.5 \times 0.606 \times 20 \times 12 \times 400}{1.25}$$

$$= 116.352 \text{ KN}$$

(c) net strength of plate per pitch: (Tearing)

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times (60 - 22) \times 12 \times 410}{1.25}$$

$$= 134.611 \text{ KN}$$

(f) ∴ Strength of ~~plate~~ connection per pitch = 45.264 KN

$$\text{Strength of plate befor drilling} = \frac{0.9\, p t \times f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times 60 \times 12 \times 410}{1.25}$$

$$= 212.544 \text{ KN}$$

so,

efficiency$(\eta) = \frac{45.264}{212.544} = 21.29\%$

Design bolted connection to connect two member 250 x 12 mm² carrying axil load of 200 KN, M20 bolt, Pr. Cl 4.6 ~~lapc~~

① single cover buttjoints:

Given.

M_{20} bolt

$d = 20mm$

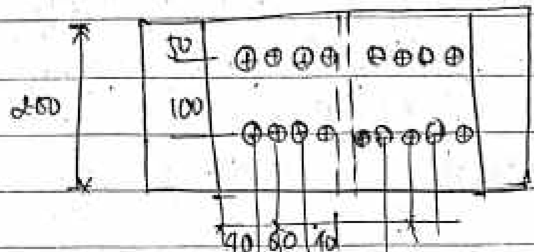
$d_o = 22 mm$

for Pr.Cl: 4.6

$f_{ub} = 400$ Mpa

$f_{yb} = 240$ "

$f_u = 410$ "



(i) Design load = P.SF x working load

$= 1.5 \times 200$

$= 300 KN$

(ii) Shear strength of bolt $(V_{dsb}) = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times (n_n A_{nb} + n_s A_{sb})$

$= \frac{1 \times 400}{\sqrt{3} \times 1.25} \times 0.78 \times \frac{\pi \times 20^2}{4}$

$= 45.27 KN$

(iii) No. of Bolts $= \frac{300}{45.27} = 6.62$ take, 8 bolts.

(iv) pitch $= 2.5d = 2.5 \times 20 = 50 mm$ take, $P = 60mm$

end.d $= 1.5 d_o = 1.5 \times 22 = 33mm$. $e = 40mm$

(v) Design bearing capacity $= V_{dpb} = \frac{2.5 k_b d t f_{ub}}{1.25} = \frac{2.5 \times 0.606 \times 20 \times 8 \times 400}{1.25}$

$k_b = \frac{e}{3d_o}$ or $\frac{p}{3d_o} - 0.25$ $= 77.568 KN$

0.606 , 0.65

$\therefore$ Total bearing capacity $= 4 \times 77.568 \times 2$

$= 2 \times 310.272\ kN > 300\ kN$

Hence,

provide 8 bolts of M20, Pr. Cl. 4·6 for single cover of $t = 8\ mm$ butt joints.

High strength friction grip bolt

10.4, Page: 76

Nominal shearing friction capacity of bolt $(V_{nsf}) = \mu_f n_e K_h F_o$

Design .. $(V_{dsf}) = \dfrac{V_{nsf}}{\gamma_{mf}}$

μ_f = coefficient of friction (0.55)

n_e = no. of friction plane (parallel to load)

K_h = 1.0 for fasteners holes (clearance), 0.70 long slotted hole

" = 0.85 for " in oversized and short slotted holes

F_o = Min. bolt tension = $A_{nb} f_o$

A_{nb} = net area of bolt at thread

f_o = proof stress $(0.70 f_{ub})$

γ_{mf} = 1.10 (If slip resistance is designed for service load)

= 1.25 (" " " " ultimate ")

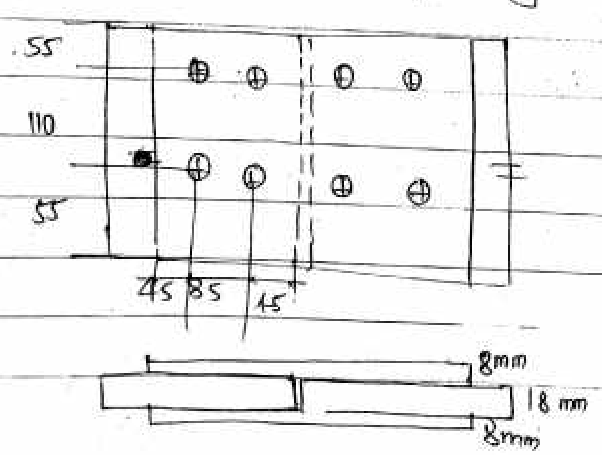
2070/Bhadra:

Two plates 18 mm thick as spliced (connect) by a cover plate of 8mm using M18 bolt of pr. cl. 10.9, calculate ultimate load carrying capacity (strength) of connection

(i) if slip is permitted at ultimate load.

(ii) if slip is not " " " [friction type]

(iii) if slip is not " at working " [slip is permitted at ultimat load] bearing.



soln: ① friction type under working load.

(i) Design shear friction capacity of bolt (V_{dsf}) = $\frac{\mu_f\, n_e\, K_h\, f_0}{\gamma_{mf}}$

$= \frac{0.55 \times 2 \times 1.0 \times 0.78 \times \pi \times \frac{18^2}{4} \times 0.70 \times 1000}{1.10}$

$= 138.94\ KN.$

② (ii) friction typer Ultimate load.

(ii) Design shear friction capacity of bolt (V_{dsf}) $= \frac{0.55 \times 2 \times 1.0 \times 0.78 \pi \times \frac{18^2}{4} \times 0.70 \times 1000}{1.25}$

$= 122.27\ KN$

(iii) design shear strength (V_{dsb}) $= \frac{1000}{\sqrt{3} \times 1.25} \times \left(1.78 \times \pi \times \frac{18^2}{4}\right)$

$= 209.20\ KN$

(iv) design bearing strength (V_{dpb}) ⇒ K_b can be

$\frac{e}{3d_0}, \frac{P}{3d_0} - 0.25, 1, \frac{f_{ub}}{f_{ub}}$

or, $\frac{45}{3 \times 20}, \frac{85}{3 \times 20} - 0.25, 1, \frac{410}{1000}\ \frac{1000}{410}$

$0.75,\ 1.16,\ 1,\ 0.\frac{1}{410}\ (2.44)$

So, $(V_{dpb}) = \frac{2.5 \times 0.75 \times 18 \times 16 \times 410}{1.25}$

$= 177.120\ KN$

(v) Yeilding strength (T_{dg}) $= \frac{f_y A_g}{\gamma_{mo}} = \frac{250 \times 220 \times 16}{1.10}$

$= 800\ KN$ — (i)

(vi) Tearing " (T_{dn}) $= \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times (220 - 2 \times 20) \times 16 \times 410}{1.25}$

at (critical section)

$= 850.176\ KN$ — (ii)

i)

iii)

$< f_{ud}$

section

11

Total shearing briction strength at working load $= 4 \times 138.94 = 555.76$ KN — (iii)

at ultimate load $= 4 \times 122.267 = 489.068$ KN — (iv)

Total shear capacity at bearing type bolt $= 836.29$ KN — (v)

Total bearing capacity $= 4 \times 177.12 = 708.48$ KN — (vi)

(i) When slip is permitted at ultimate load:

(Bearing type मा काम गर्छ, friction मा काम गर्दैन ।

ultimate load capacity $=$ min of T_{dg}, T_{dn}, (v) & (vi)

$= 708.48$ KN

(ii) When slip is not permitted at ultimate load:

i.e, friction type मा काम गर्छ।

ultimate load capacity $=$ min. of T_{dg}, T_{dn}, (iv) & (vi)

$= 489.068$ KN.

(iii) When slip is not permitted at working load.

(Bearing type' भयो)

$\therefore$ working load capacity $=$ min. of T_{dg}, T_{dn}, (iii) (v)

$= 555.76$ KN.

Design bolt connection using M16 bolt of pr.cl: 8.8 to connect two plates 200x16 mm carrying a load of 300 KN. at working condition if, (i) slip is not permitted at working load

(ii) slip is " " " ultimate load.

design double cover butt joints of 10mm thick cover plate.

Now;

Given:

M16 bolt

$d = 16mm$

$d_0 = 18mm$

$f_{ub} = 800$ Mpa

$f_{yb} = 400$ Mpa.

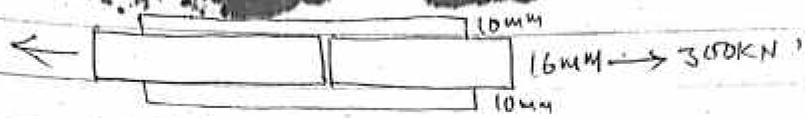
let, Fe410 Grade of steel

so, $f_u = 410$ Mpa

Working load = 300 kN

Ultimate load = $300 \times 1.5 = 450$ kN

Plate dimension = 200×16



Then,

① For working load

Design shear friction capacity $= (V_{dsf}) = \dfrac{0.55 \times 2 \times 1.0 \times f_o}{\gamma_{mf}}$

$f_o = A_{nb} \cdot f_o$

$= 0.78 \times \dfrac{\pi d^2}{4} \times 0.7 \times f_{ub}$

$= 87823.85$

$(V_{dsf}) = 87.82384$ kN

$= 87.823$

So, no. of bolts at working load

$= \dfrac{300}{87.823}$

$= 3.41$

$= 4$ bolts

② for Ultimate load. $\gamma_{mf} = 1.25$

Design shear friction capacity

$(V_{dsf}) = \dfrac{\mu_f \cdot k_h \cdot n_e \times f_o}{\gamma_{mf}}$

$= \dfrac{0.55 \times 2 \times 2 \times 87823.85}{1.25}$

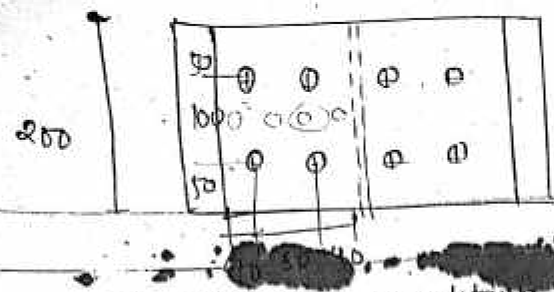
$= 77.284$ kN

So, no. of bolts at Ultimate load

$= \dfrac{300 \times 1.5}{77.284}$

$= 5.82$

$= 6$ bolts



at working condition.

$n_n = 1,\ n_s = 1$

(iii) design shear strength $(V_{dsb}) = \dfrac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

$$= \frac{800}{\sqrt{3} \times 1.25} \times 1.78 \times \pi \times \frac{16^2}{4}$$

$$= \frac{1145248.75}{8.66025}$$

$$= 132.241\,KN$$

(iv) Design bearing strength $(V_{dpb}) = \dfrac{2.5 K_b\, d t (f_u / f_{ub})}{1.25}$

for K_b: $\dfrac{e}{3d_0},\ \dfrac{p}{3d_0} - 0.25,\ 1,\ \dfrac{f_{ub}}{f_u}$

$p \not< 2.5d = 2.5 \times 16 = 40mm$ $\quad \therefore 0.740,\ 0.86,\ 1,\ 1.95$

$e \not< 1.7 d_0 = 1.7 \times 18 = 30.6mm$

take, $P = 60mm$

$e = 40mm$

so $K_b = 0.740$

and, $(V_{dpb}) = \dfrac{2.5 \times 0.74 \times 16 \times 16 \times 410}{1.25}$

$= 155.340\ KN$

(v) Yielding strength $(T_{dg}) = \dfrac{f_y A_g}{\gamma_{m0}} = \dfrac{250 \times 200 \times 16}{1.10} = 727.272\,KN$ —— (i)

$(t < 20\ mm)$

(v). Tearing strength $(T_{dn}) = \dfrac{0.9 A_n f_u}{\gamma_{m1}} = \dfrac{0.9 \times (200 - 2 \times 18) \times 16 \times 410}{1.25}$ —— (ii)

$= 774.604\ KN$

(Friction)

so, total shearing frictional ultimate load = 77.284 × 4 = 309.136 KN (III)

total " " " working load = 87.823 × 4 = 351.292 KN (IV)

(Bearing)

Total shearing capacity for bearing type bolt = 132.241 × 4 = 528.964 (V)

Total bearing capacity for bearing " " = 155.340 × 4 = 621.36 KN (VI)

(I) When slip is Not permitted at working load: (Bearing type)

so,

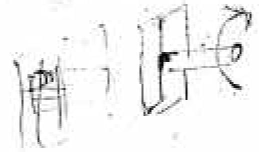
Ultimate load capacity = min of (I), (II), (IV) and (VI)

= 351.292 KN

(II) When slip is not permitted at Ultimate load. (friction type)

Ultimate load capacity = min. of (I), (II), (III) and (VI)

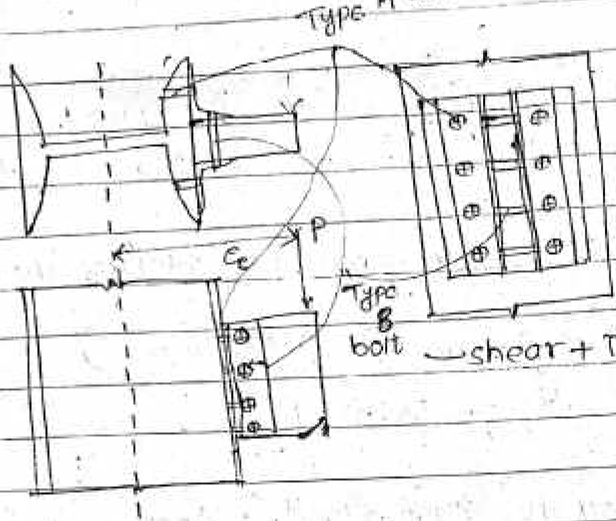
= 309.136 KN



Eccentric connection:

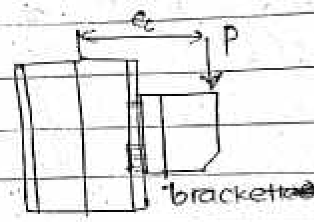
if the bolt are under the action of moment as well as shear, such connection is called eccentric connection.

Type A bolt – shear | bending force

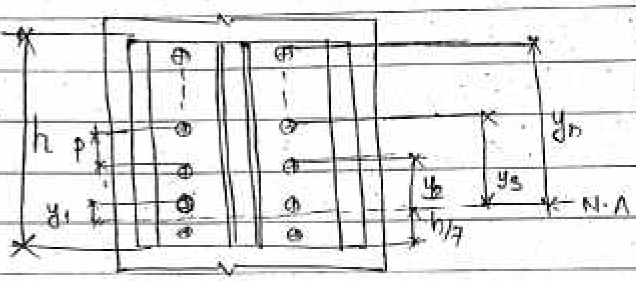


Type B bolt – shear + Torsion

Type A bolt is under bending moment & shear whereas type B bolt is under shear & Torsion.



Design of type A bolt



We assume that neutral axis lies at h/7 from bottom bracket plate,

we also know that,

$$M = P \times e_c$$

$$M = [T_1 y_1 + T_2 y_2 + \dots + T_n y_n] \times 2 \quad \text{—(1)}$$

also from bending stress relation, we have,

$$T \propto y$$

Hence, $\dfrac{T_1}{y_1} = \dfrac{T_2}{y_2} = \dfrac{T_3}{y_3} = \dots = \dfrac{T_n}{y_n}$

Or, $T_1 = \dfrac{T_n y_1}{y_n}$, $T_2 = \dfrac{T_n y_2}{y_n}$, $T_3 = \dfrac{T_n y_3}{y_n}$ ... $T_n = \dfrac{T_n \cdot y_n}{y_n}$

Then,

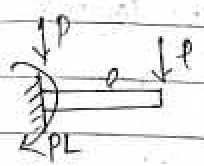
$$P \times e_c = 2\left[\frac{T_n}{y_n} \cdot y_1^2 + \frac{T_n}{y_n} y_2^2 + \frac{T_n}{y_n} \cdot y_3^2 + \cdots + \frac{T_n}{y_n} \times y_n^2\right]$$

$$= 2\frac{T_n}{y_n}\left[y_1^2 + y_2^2 + y_3^2 + \cdots + y_n^2\right]$$

$$P \cdot e_c = \frac{2T_n}{y_n} \Sigma y_i^2$$

$$\text{or. } T_n = \frac{P \times e_c \cdot y_n}{2\Sigma y_i^2}$$

$$\therefore T_{max} = \frac{P \times e_c \times y_{max}}{2\Sigma y_i^2}$$



Shear force on each bolt $(V_{max}) = \frac{P}{N}$

N - no. of total bolt

Then for safe design following combined relation should be satisfied.

$$\left(\frac{T_{max}}{T_{db}}\right)^2 + \left(\frac{V_{max}}{V_{dsb}}\right)^2 \leq 1.0 \quad \text{clause (10.3.6)}$$

Note: No. of bolt required is given by $n = \sqrt{\frac{6P \times e_c}{m \times p \times T_{db}}}$

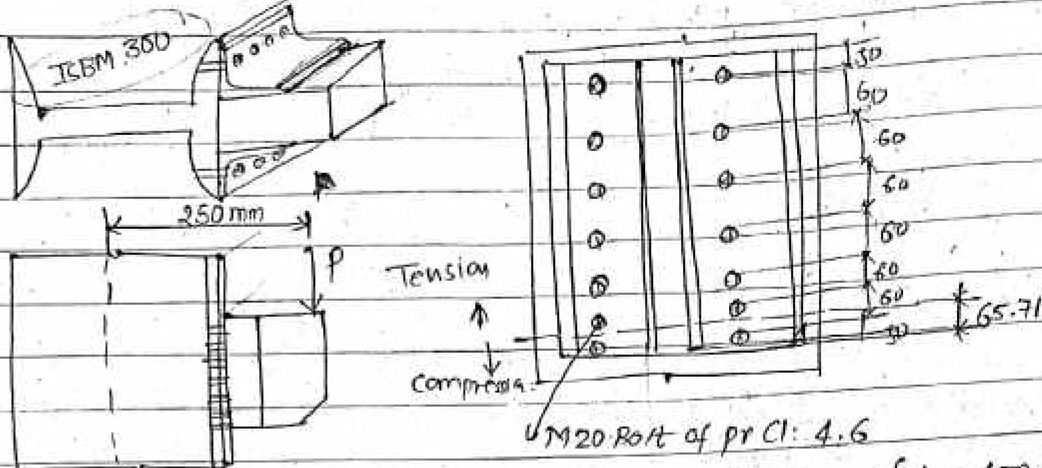
P - applied load

p - pitch

T_{db} - design tensile capacity

m = no. of line of bolt

Calculate the load that can be applied on the following connection.



$f_{ub} = 400\ N/mm^2$

$f_{yb} = 240\ N/mm^2$

Total height of bracket $(h) = 50\times2 + 60\times6$

$= 360 + 100$

$= 460mm$

Position of N.A $= \frac{460}{7} = 65.71mm$

$\Sigma y_i^2 = [44.29^2 + 104.29^2 + 164.29^2 + 224.29^2 + 284.29^2 + 344.29^2]$

$= 289491.6246$

$\therefore y_{max} = 344.29mm$

$T_{max} = \frac{P * e_c \times y_{max}}{2\Sigma y_i^2}$

$= \frac{P \times 250 \times 344.29}{2\times 289491.6246}$

(i) $T_{max} = 0.1486P$ —— (i)

(ii) $V_{max} = \frac{P}{N} = \frac{P}{14} = 0.071P$ —— (ii)

(iii) $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\times1.25} * (n_n A_{nb} + \cancel{n_s}^{0} A_{sb})$

$= \frac{400}{\sqrt{3}\times1.25} \times 0.78\times\pi\times\frac{20^2}{4} = 45.272\ Mpa$ ✓

(iv) $T_{db} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times A_n \times 410}{1.25}$

$A_n = (b - n d_1) \times t$

$=$

$T_{dg} = \frac{f_y \cdot A_g}{\gamma_{m0}} = 250 \times 460 \times$

$= 68.54 \text{ kN}$

Tensile strength $T_{(db)}$

$0.9 f_{ub} A_{nb} \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{m0}}$

$= \frac{0.9 \times 400 \times 0.78 \times \pi \times 20^2}{4} \leq 240 \times \frac{\pi \times 20^2}{4} \times \frac{1.25}{1.10}$

$= 88.215 \leq 85.679 \text{ kN}$

So, $T_{db} = \frac{85.679}{1.25}$

$= 68.543 \text{ kN}$

Then,

we have,

$$\left(\frac{0.1486P}{68.54}\right)^2 + \left(\frac{0.071P}{45.272}\right)^2 \leq 1.00$$

Solving we get

$P = 373.714 \text{ kN}$

So, service load $= \frac{\text{design load}}{\text{FOS}}$

$= \frac{373.714}{1.5}$

$= 249.142 \text{ kN}$

⊕ Design eccentric connection of an axial load of 150 kN at an eccentricity of 300 mm from center of column, using M16 bolt of pr. cl. 4.6

Solⁿ:-

Now

Given: $P = 150 kN$ (service load)

Design load $P_d = 150 \times 1.5$
$= 225 kN$

no. of bolt $N = \sqrt{\dfrac{6 P \times e_c}{m p \times T_{db}}}$

for design shear strength of bolt

$V_{dsb} = \dfrac{f_{ub}}{\sqrt{3} \times 1.25} \times \left(0.787 \times \dfrac{16^2}{4}\right)$

$= 28.917 kN$

$V_{max} = \dfrac{P}{N} = \dfrac{225}{10}$
$(V_{sb}) = 22.5 kN$

Tdb can be calculated as

$0.9 f_{ub} \times A_{nb} \leq f_{yb} \times A_{sb} \times \dfrac{\gamma_{mb}}{\gamma_{m1}}$

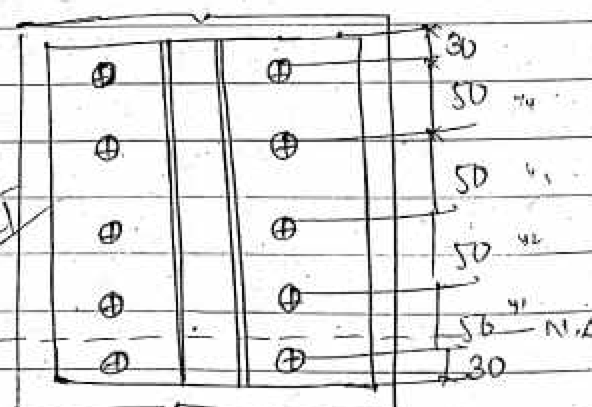
$0.9 \times 400 \times 0.787 \times \dfrac{16^2}{4} \leq 240 \times \pi \times \dfrac{16^2}{4} \times \dfrac{1.25}{1.10}$

$56.46 \leq 548367$

So, $T_{db} = \dfrac{54.83}{1.25} = 43.86 kN$

pitch (p) $\geq 2.5 d$
$\geq 2.5 \times 16$
$\geq 40 mm$
take 50 mm

end distance $\geq 1.5 \times d_0 = 1.5 \times 18 = 27$
$\simeq 30 mm$



So,

$n = \sqrt{\dfrac{6 \times 225 \times 300}{2 \times 50 \times 43.86}}$

$= 9.6$

≈ 10

Total height = 260 mm

$N.A = \dfrac{260}{7} = 37.142$

150+40

Then.

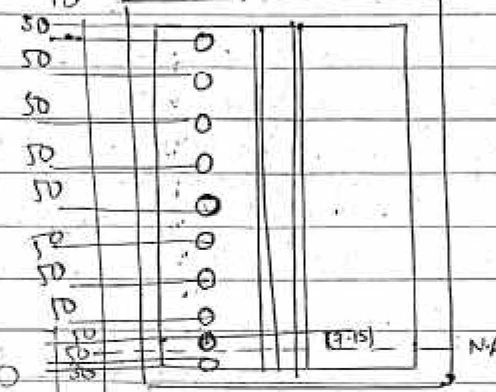
$$\Sigma y_1^2 = 42.858^2 + 92.858^2 + 142.858^2 + 192.858^2 + \cancel{222.858^2}$$
$$= 117725.435 - 222.858^2$$
$$= 68059.74$$

$y_{max} = \cancel{222.858}\ 192.858$

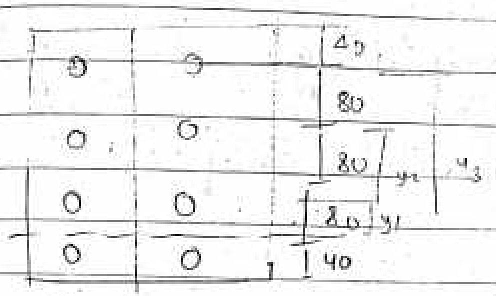
$$T_{max} = \frac{225 \times 300 \times \cancel{222.858}\ 192.858}{2 \times \cancel{117725.435}\ 68059.74}$$
$$= \cancel{63.889}\ 95.63 KN$$

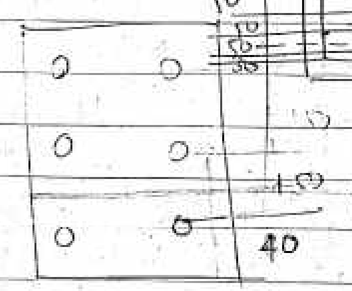
$$V_{max} = \frac{P}{N} = \frac{225}{10} = 22.5$$

$$\left(\frac{63.889}{43.86}\right)^2 + \left(\frac{22.5}{28.917}\right)^2 > 1 \quad \text{no ok}$$



p = 80 mm. n = 8
e = 40 mm





p = 50 mm
e = 30 mm

$\therefore h = 510$

$N.A = h/7 = \cancel{7}2.85$

h = 320

N.A = 45.914

$\therefore \Sigma y_i^2 = 74.286^2 + 154.286^2 + 234.286^2 = 84212.50$

$y_{max} = 234.286$

So, $T_{max} = 33.$

$\therefore y_{max} = 407.15$ mm

$\Sigma y_i^2 = 602326.225$

$\therefore T_{max} = 22.81$ KN

So, $\left(\frac{22.81}{43.86}\right)^2 + \left(\frac{22.5}{28.917}\right)^2 = 0.876 < 1$

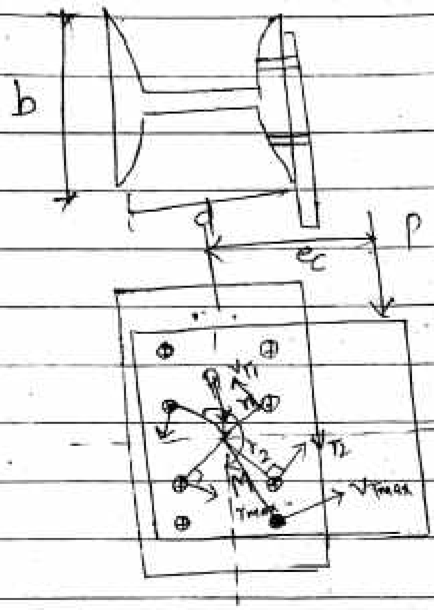
OK!

for Bearing:

$$V_{dpb} = \frac{2.5 \times K_b \times d\,t\,f_{ub}}{1.25}$$
$$=$$

$K_b = \frac{e}{3d_0} = 0.55$

Eccentric connection (Type-B)
(parallel)



Type B bolt under shear & Torsion

Then:

$$M = P \times e_c$$

It is resisted by shear developed on each bolt at ⊥r to radius vectors.

Then,

$$M = r_1 \times V_{T1} + r_2 \times V_{T2} + r_3 \times V_{T3} + ---- + r_n V_{Tn}$$

also, $V_T \propto r$

$$\text{so, } \frac{V_{T1}}{r_1} = \frac{V_{T2}}{r_2} = ---- = \frac{V_{Tn}}{r_n}$$

$$\text{so, } V_{T1} = \frac{V_{Tn} r_1}{r_n} \quad V_{T2} = \frac{V_{Tn}}{r_n} r_2$$

$\sigma \propto y$

$\boxed{V_{Tn} \propto r}$

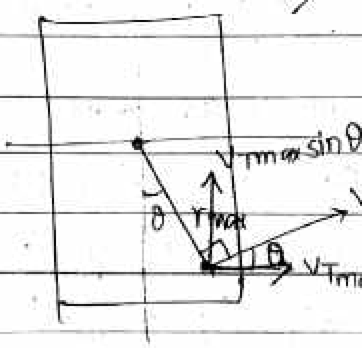
(Bending stress) behave s.t.c

direction shear of along HT (bolt)

$$\therefore M = \frac{V_{Tn}}{r_n} r_1^2 + \frac{V_{Tn}}{r_n} r_2^2 + \frac{V_{Tn}}{r_n} r_3^2 + --- + \frac{V_{Tn}}{r_n} \times r_n^2$$

$$= \frac{V_{Tn}}{r_n} \left[r_1^2 + r_2^2 + ---- + r_n^2 \right]$$

$$M = \frac{V_{Tn}}{r_n} \times \Sigma r_i^2$$



$$\therefore \boxed{V_{Tn} = \frac{M * r_n}{\Sigma r_i^2}}$$

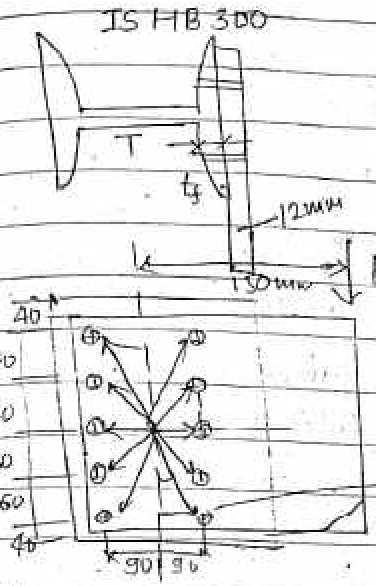
$$V_{Tmax} = \frac{M * r_{max}}{\Sigma r_i^2}$$

Vertical Shear force on each bolt $(V) = \frac{P}{N}$

Resultant $(R) = \sqrt{V_x^2 + V_y^2}$

$V_y = V + V_{Tmax} \sin\theta$

calculate max. value of load p, that can be applied in the following connection



Now,

$d = 16\,mm$

$d_o = 18\,mm$

$f_{ub} = 400\,MPa$

$f_{yb} = 240\,Mpa$

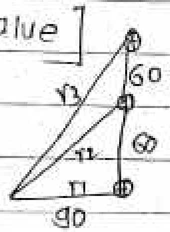
(a) Design shear capacity of bolt

$$(V_{dsb}) = \frac{f_{ub} \times 0.78 \pi d^2}{\sqrt{3} \times 1.25 \times 4}$$

$$= \frac{400 \times 0.78 \times \pi \times 16^2}{\sqrt{3} \times 1.25 \times 4}$$

$$= 28.974\,kN \quad \text{---} (I)$$

$[R \leq \text{bolt value}]$



(b) Bearing capacity, k_b can be found as

$$\frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, 1, \frac{f_u}{f_{ub}}$$

$$\frac{40}{3 \times 18}, \frac{60}{3 \times 18} - 0.25, 1, \frac{410}{400}$$

$$0.740, 0.86, 1, 0.970$$

$k_b = 0.740$

from 5 A

IS HB 300	58.8	$r_1 = 90$
300	63.0	$r_2 = 108.16$
		$r_3 = 150$

X-section - T

flange Thickness = T_f

for ISHB 300 63 kg, thickness of flange = 10.6 mm

$$\therefore V_{dpb} = \frac{2.5 \times 0.740 \times 16 \times 10.6 \times 400}{1.25}$$

$$= 100.403\,kN \quad \text{---} (II)$$

So, Bolt value = min. of (I) & (II)

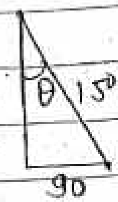
= 28.974 kN

Then, $\Sigma r_i^2 = 2 \times 90^2 + 4 \times 108.16^2 + 4 \times 150^2 = 152994.34$

$r_{max} = \sqrt{\ }$ 150mm

$\therefore V_{Tmax} = \frac{Pe_c \times r_{max}}{\Sigma r_i^2}$

$V_{Tmax} = \frac{P \times 150 \times 150}{152994.34} = 0.147P$



$V_{Tmax} \sin\theta = 0.147P\sin\theta$

$= 0.08818P$

$V = \frac{P}{N} = \frac{P}{10} = 0.1P$

$\therefore \theta = \sin^{-1}\left(\frac{90}{150}\right)$

$= 36.87^\circ$

$V_y = V + V_{Tmax}\sin\theta = 0.18818P$

$V_x = V_{Tmax}\cos\theta$

$= 0.1176P$

so, $R = \sqrt{V_x^2 + V_y^2}$

$= \sqrt{(0.1176P)^2 + (0.18818P)^2}$

$= P \times 0.222$

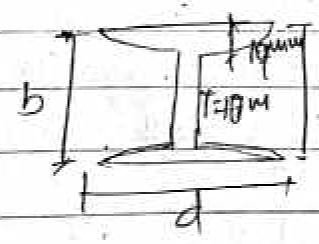
$\boxed{\text{since, } R \leq 28.97}$

$0.222P \leq 28.97$

$\therefore P \leq 131.68$ kN

Design bolted connection to carry an axial ~~w~~load of 180 kN at a eccentric distance of 250mm applied parallel to flange. Use M20 bolt of pr.cl 4.6. The column SC 250

The thickness of brackett is 16mm



$b \simeq d$

$n = \sqrt{\frac{6Pe}{m.p V_{dsb}}}$

$t_f = 19$mm

Soln:-

Given:

Axial load (W.L) = 180 KN

Design load = 270 KN

Pr.Cl: 4.6

f_{ub} = 400 Mpa

f_{yb} = 240 Mpa

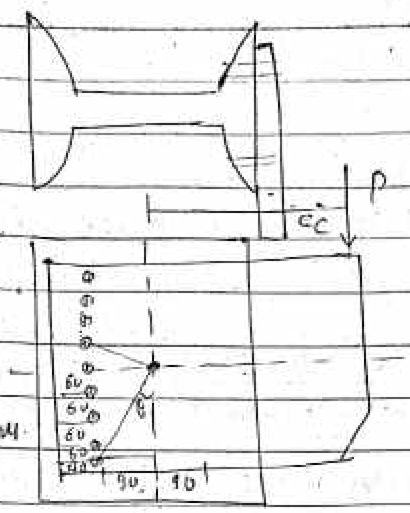
f_u = 410 Mpa for Fe 410

Eccentric distance 250 mm

column is SC 250

t_f = 19 mm

bracket thickness = 16 mm



d = 20 mm, d_o = 22 mm

Desig shear strength of bolt (V_{dsb}) $= \frac{400}{\sqrt{3} \times 1.25} \times \left(0.78 \times \pi \times \frac{20^2}{4} + 0\right)$

$= 45.272$ KN

no. of bolts $n = \sqrt{\frac{6 P e_c}{m p V_{dsb}}}$

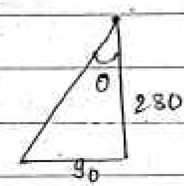
Pitch (P) $\geq 2.5d = 50$ mm

end distance $e \geq 1.5 d_o = 1.5 \times 22 = 33$ mm

take pitch = 60 mm

end d = 40 mm

so, $n = \sqrt{\frac{6 \times 270 \times 250}{2 \times 60 \times 45.272}} = 8.63 \approx 9$



$\tan\theta = \frac{90}{240}$

$\therefore \theta = 20.55°$

(ii) Bearing capacity can be found as, for K_b

$\frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, 1, \frac{f_{ub}}{f_{ub}}$

$\Rightarrow \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, 1, \frac{400}{410}$

0.606, 0.659, 1, 0.9756

$V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 16 \times 400}{1.25}$

$= 155.136$ KN

Bolt value = 45.212 kN

For, $V_{Tmax} = \frac{P e_c r_{max}}{\Sigma r_i^2}$

$r_{max} = \cancel{294.108}\ 256.32$

$\Sigma r_i^2 = 2\times 90^2 + 4\times 108.17^2 + 4\times 150^2 + 201.246^2\times 4 + 4\times 256.32^2$

$= 577802.57$

(i) $\therefore V_{Tmax} = \frac{270\times 250\times 256.32}{577802.57} = 29.94\text{ kN}$

(ii) $V_y = V + V_{Tmax}\sin\theta = \frac{P}{N} + 29.94\sin\theta = 45.50\text{ kN}$

(iii) $V_x = V_{Tmax}\cos\theta = 28.034\text{ kN}$

(iv) $R = \sqrt{V_x^2 + V_y^2} = 48.1236\text{ kN}$

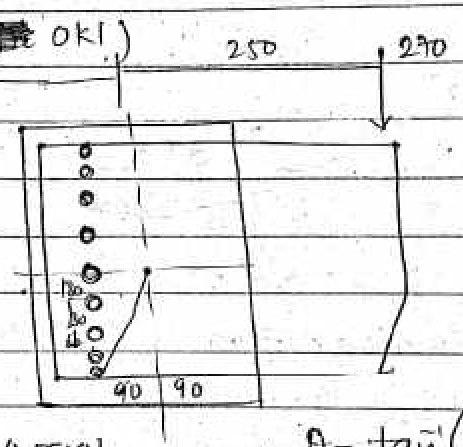
since,

$R \leq$ Bolt value (OK!)

No-need Redesigning.

for pitch $p = 80$ mm

$e = 40$ mm.



$r_{max} = 332.41$ mm

$\Sigma r_i^2 = 913860$

$\therefore V_{Tmax} = \frac{270\times 250\times 332.41}{913860} = 24.55\text{ kN}$

$\theta = \tan^{-1}\left(\frac{90}{320}\right) = 15.70^\circ$

$V = \frac{P}{N} = \frac{270}{9} = 30\text{ kN}$

$V_y = V + V_{Tmax}\sin\theta = 36.64\text{ kN}$

$V_x = V_{Tmax}\cos\theta = 23.64\text{ kN}$

$R = \sqrt{V_x^2 + V_y^2} \doteq 43.60\text{ kN}$

so, $R \leq$ Bolt value is satisfied;

Hence, provide 18, M_{20} Bolt of Pr cl: 4.6 with pitch = 80 mm, end distance 40mm
with bolt value = 45.272 kN.

2/15 [illegible]

CH-4 Connection Design:

Welded connection

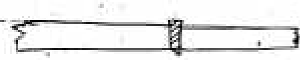
- connects two metal plates

advantage and disadvantage

advantage	disadvantage
i) rigid connection	skilled manpower
ii) can be jointed any shape	brittle joint
iii) high efficiency ≈ 100%	Electricity
iv) No drilling	no better performance in fatigue
v) Economic	Maintenance cost is high.
vi) light in weight	
vii)	

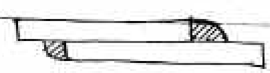
Types of welded connection:

1) Butt weld (groove weld)

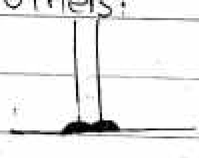


Rectangular Butt joints

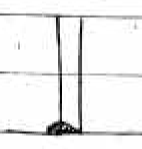
2. fillet weld.



Others:



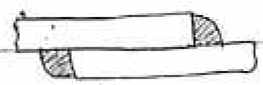
Double-Tee weld

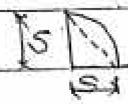


single Tee-weld

fillet weld:

⇒ Size of fillet weld: (S)





(I) max weld size depends upon thickness of thinner plate

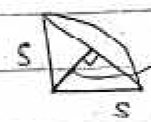
for rectangular $= t - 1.5$

for rounded $= \frac{3}{4} * t$

t = thickness of thinner plate.

(II) Min. size of weld depends upon the thickness of thicker plate given by table. 21 page: 78

Throat thickness (t_e): It is the effective size of weld.



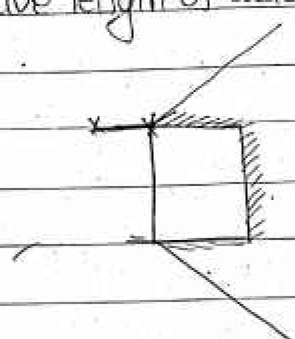
$t_e = \sqrt{s^2+s^2}$

$= 0.7S$

$t_e = K * S$ ~~constant~~

K - constant given by Table 22, for give angle betn fusion face

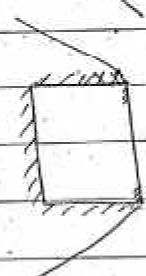
effective length of weld: It is the length of weld of full size / Area of weld.



$l_e = l_w - 2S$

l_w = overall length of weld

End return:



Extra length of weld provided to opposite side of welding

effective area = $A_e = l_e \times t_e$

gusset plate - it can't take strength only for assist connection

tack bolt
Extra bolt to meet design specification

Page (78 - 79)

overlap: common length of two plates $\geq 40mm$

$\geq 4t$ → t-thickness of thinner plate

Calculation of strength

(a) Design strength of weld $(f_{wd}) = \frac{f_{wn}}{\gamma_{mw}}$

$$= \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

γ_{mw} ⇒ Table: 5
1.25 shop
1.50 field

Design welded connection to connect the plates 120x12 mm to a gusset plate of thickness 20 mm to carry an axial load of 600 kN by fillet welds:

assume field fabrication ($\gamma_{mw} = 1.50$)

$f_u = 410$ Mpa

A) Size of weld:

1) Min size of weld (S_{min}) = 5 mm for t = 10-20 mm of thick plate.

2) Max size of weld $(S_{max}) = t - 1.5 = 12 - 1.5 = 10.5$ mm

adopt, S = 8 mm

B) Effective throat thickness $(t_e) = 0.7S = 0.70 \times 8$
$= 5.6$ mm

c) effective length of ~~throat~~ weld

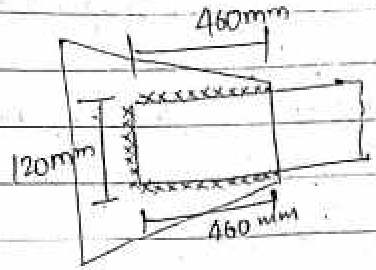
d) Design strength of weld $(f_{wd}) = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.50} = 157.81$ MPa

Now, Pu (ultimate load) = $A_e \times f_{wd}$

$600 \times 10^3 \times 1.5 = l_e \times t_e \times 157.81$

∴ $l_e = 1018.403$ mm

overlap ≥ 40 mm
$\geq 4 \times 12 = 48$ mm



$l_w = l_e + 2S$

$= 1018.403 + 2\times 8$

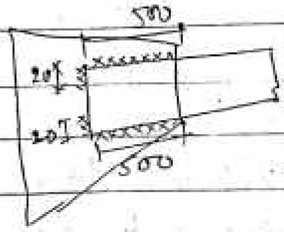
$= \underline{1034.403}\,mm$

$= \dfrac{1034.403 - 120}{2}$

$= 457\,mm$

अथवा.

take $\cong \underline{460}\,mm > 480\,mm.$



Design welded connection to connect an angle ISA 100 x 100 x 10 mm to a gusset plate of thickness 16 mm, carrying an axial load of 100 kN if not state (such that the load transfer through the CG of angle)

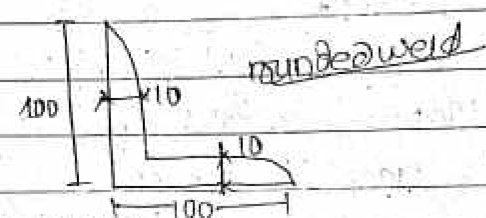
Soln:

for angle ISA 100 x 100 x 10

$C_x = C_y = 28.4\,cm$

axial load = $P = 100\,kN$

Ultimate load $P_u = 100 \times 1.5 = 150\,kN$



Now, ~~Shape~~ size of weld → $S_{max} = 3/4 \times t = \frac{3}{4} \times 10 = 7.5\,mm$

$S_{min} = 5\,mm$

take; $s = 7\,mm$

Throat thickness $(t_e) = 0.7S = 0.7 \times 7$

$= 4.9\,mm$

Design stress of weld $f_{wd} = \dfrac{f_u}{\sqrt{3}\times 1.50} = \dfrac{410}{\sqrt{3}\times 1.50} = 157.81\,Mpa$

Ultimate load $= A_e f_{wd} = l_e t_e f_{wd}$

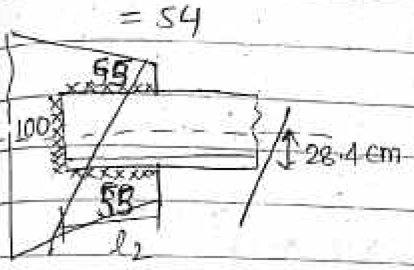
$\therefore l_e = \dfrac{1.50 \times 10^3}{4.9 \times 157.81} = 193.99\,mm$

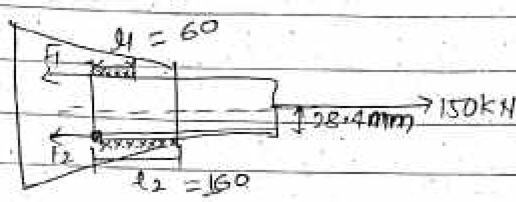
$$l_w = l_e + 2s$$
$$= 193.99 + 2 \times 7$$
$$= 193.99 + 14$$
$$= 194 + 14$$
$$= 208\,mm$$

Overlap $\Rightarrow \geq 40\,mm$

$\geq 4 \times 10 = 40\,mm$.

$$= \frac{208 - 100}{2}$$
$$= \frac{108}{2}$$
$$= 54$$





$$l_1 + l_2 = l_w$$

$$F_1 = l_1 \times t_e \times f_{wd}$$

$$F_2 = l_2 \times t_e \times f_{wd}$$

taking moment about bottom of angle

$$150 \times 28.4 = l_1 \times 4.9 \times 157.81 \times 100$$

$$\therefore l_1 = 55\ mm$$

take $l_1 = 60$ mm

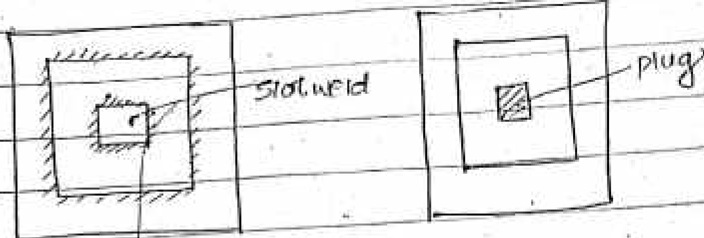
$$l_2 = 208 - 55$$
$$= 153$$

take, $l_2 = 160\ mm$

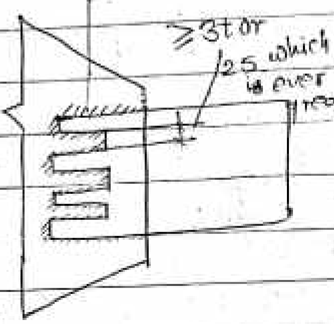
Tewsim

Welded connection

Slot and plug connection.



or,



if the length of weld required is not avilable special weld are done as shown named plug or slot weld to meet the required length of weld.

शुद्धता (theory rey)

Design fillet weld to connect a tie member 120X12 mm to a gusset plate of 16 mm thick carrying factored axilload of 900kN if the overlap is restricted to 200 mm. (shop)

Given:-

Pu(ultimate load) $= 900kN$

Size of weld $\Rightarrow$

$S_{max} = \cancel{3/4 \times t' = \frac{3}{4} \times 12 = 9mm}$ $t - 1.5$

$= 12 - 1.5 = 10.5mm$

$S_{min} = 5mm$

take $S = 7mm$

so,

throat thickness $= t_e = 0.7S$

$= 0.7 \times 7$

$= 4.9 mm$

Design stress of weld $= \frac{f_u}{\sqrt{3} \times 1.250} = \frac{410}{\sqrt{3} \times 1.25} = 189.37$

ultimate load = $A_e f_{wd}$

$= l_e t_e f_{wd}$

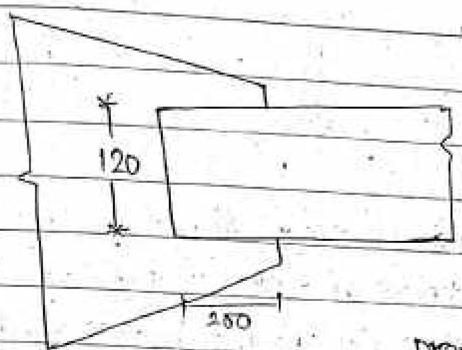
$\therefore l_e = \dfrac{900\times10^3}{4.9\times189.37}$

$= 969.91$ mm

$\therefore l_w = l_e + 2s = 969.91 + 2\times4.9$

overlap $\ge$ ~~40mm~~ $= 983.91$ mm

$\ge 4\times12 = 48$ mm



Since, the length of weld required is > ~~not~~ length available in fillet weld.

So,

slot weld $= 983.91 - (120 + 200\times2)$

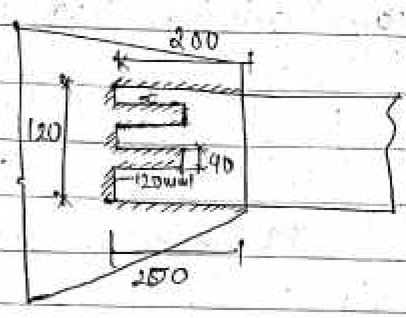
$= 463.91$ mm

provide, no. of slot required = 2 slot of 120×40 mm

(width of slot) $W_t \ge 3t = 3\times12 = 36$ mm

$\ge 25 = 25$ mm

$\therefore$ provided slot weld is

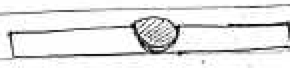


Butt weld or groove weld:

It is a end to end connection

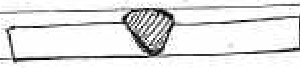
Types:

(i) single U - weld



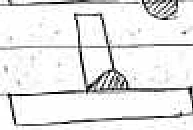
(ii) Double U weld

(iii) Single V weld

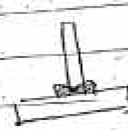


(iv) Double - V weld

(v) Tee weld.



6) Double Tee



→ all the design criteria are similar to the fillet weld except throat thickness

→ Throat thickness is taken as equal to thickness of thinner plate in case of complete penetration. (all double weld) and $(7/8)$ of the ~~thickness~~ thickness of thinner plate, in incomplete penetration.

But throat thickness is taken as $(5/8)$ of thickness of thinner plate for design consideration in Incomplete penetration

Design connection betn 2 plates 200 x 10 mm carrying factored axial load of 120 kN in,

(i) Single U weld

(ii) double U weld

(a) Given:

for 5, Smax =

$P_u = 120\,kN$

$t = 10\,mm$

Now, in Sing U weld.

$$\text{throat thickness} = \frac{5}{8} \times t = \frac{5}{8} \times 10 = 6.25\,mm$$

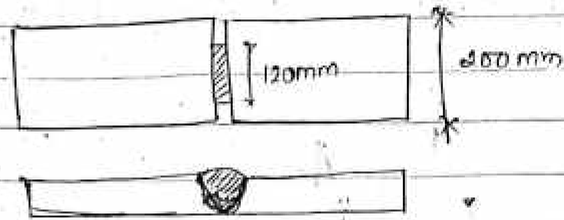
$$f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25} = \frac{410}{\sqrt{3} \times 1.25} = 189.37\,kN$$

Then:

$P_u = l_e t_e f_{wd}$

$\therefore l_e = \dfrac{120 \times 1000}{6.25 \times 189.37}$

$= 101.38$ mm



$l_w = l_e + \cancel{2s}$

$= 101.38 + \cancel{2\times0}$

$= 101.38$ mm

Take. $l_e = 120$mm

(ii) for double V weld.

Given:

$P_u = 120$ kN

$t = 10$ mm

$f_{wd} = \dfrac{f_u}{\sqrt{3} \times 1.25} = \dfrac{410}{\sqrt{3} \times 1.25} = 189.37$ kN

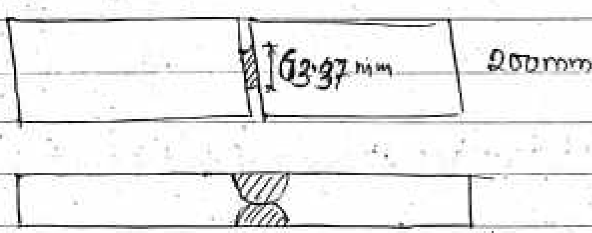
throat thickness $= \cancel{[illegible]} \times 10 = 10.00$mm

$\therefore P_u = l_e t_e f_{wd}$

$l_e = \dfrac{120 \times 1000}{10.00 \times 189.37}$

$= \cancel{63.92 \text{ mm}}$

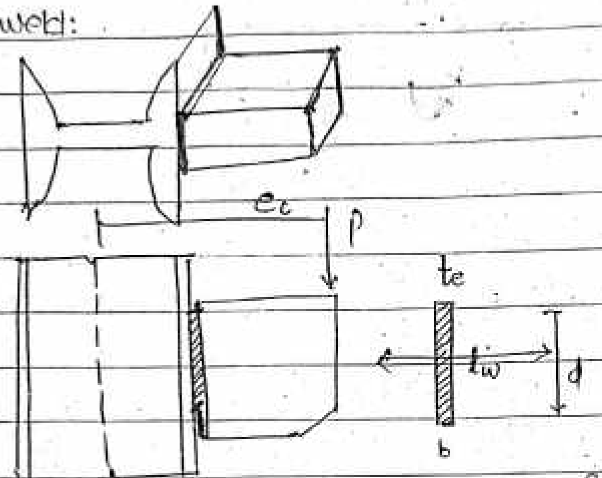
$= 63.37$ mm



Eccentric connection:

* Perpendicular to flange [weld under bending & shear)

(a) Butt weld:



$Z = \frac{bd^2}{6}$

$= \frac{t_e \times l_w^2}{6}$

design stress of weld $f_{wd} = \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

weld is under bending stress and shear stress

(a) shear stress in weld $(q) = \frac{P}{l_w \times t_e}$

(b) Bending stress in weld $(f_b) = \frac{M}{Z}$

$= \frac{M}{\frac{t_e l_w^2}{6}}$

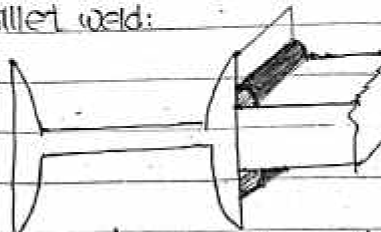
$\therefore f_b = \frac{6M}{l_e t_w^2} = \frac{6Pe_c}{l_e t_w^2}$

$\therefore$ Resultant stress $= \sqrt{f_b^2 + 3q^2} \leq f_{wd}$

(ii) for design:

length of weld required $(l_w) = \sqrt{\frac{6P \times e_c}{t_e \times (f_y/\gamma_{m0})}}$

(b) fillet weld:

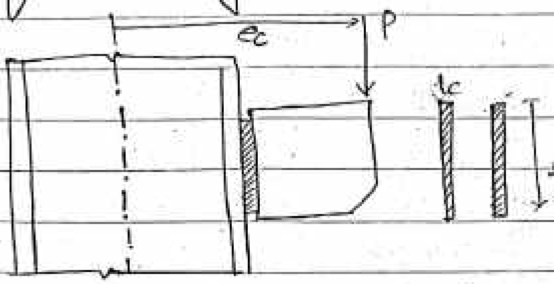


a) design shear stress of weld (f_{wd})

$$= \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

weld is under bending and shear

b) shear stress in weld $= \frac{P}{2 l_w t_e} = q$



c) Bending stress (f_b) $= \frac{M}{2Z}$

$$= \frac{6Pe_c}{2 t_e l_w^2}$$

Now;

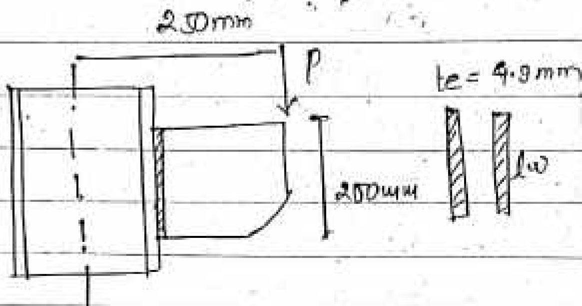
Resultant stress $= \sqrt{f_b^2 + 3q^2} \leq f_{wd}$

for design

length of weld (l_w) $= \sqrt{\frac{6P \times e_c}{2 t_e \times (f_y/\gamma_{mo})}}$

Calculate max. value of load P that can be applied in following connection

Given;



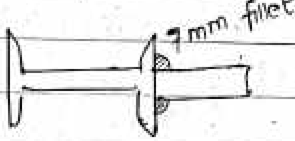
Given

$e_c = 250mm$

$l_w = 200mm$

$t_e = 4.9mm = 0.7S$

size of weld = 7mm



shear stress in weld (q) $= \frac{P}{2 l_w t_e}$

$$= \frac{P}{2 \times 200 \times 4.9}$$

$$= 5.10 \times 10^{-4} P$$

Bending stress $(f_b) = \frac{M}{Z}$

$$= \frac{6\,Pe_c}{2 \times t_e l_w^2} = \frac{6 \times P \times 250}{2 \times 4.9 \times 200^2}$$

$$= 3.82 \times 10^{-3} P$$

Resultant stress $(R) = \sqrt{3q^2 + f_b^2}$

$$= P \times \sqrt{3 \times (5.1 \times 10^{-4})^2 + (3.82 \times 10^{-3})^2}$$

$$= 3.92 \times 10^{-3} P$$

design stress of weld $(f_{wd}) = \frac{410}{\sqrt{3} \times 1.25} = 189.37$ KN

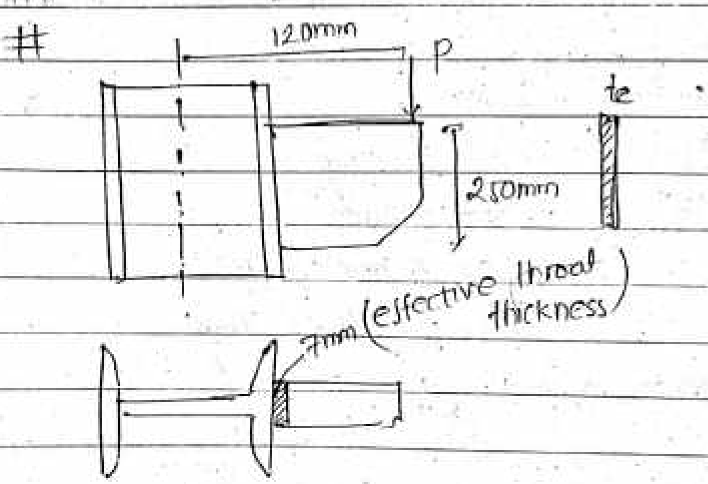
for safe design

$$R \leq f_{wd}$$

$$P \times 3.92 \times 10^{-3} \leq 189.37$$

$$\therefore P_u \leq 48.30 \text{ KN}$$

Working load (P) = 32.21 KN



Sol^{n}:

Given:

load = P

$e_c = 120$mm

$t_e = 7$mm

$l_w = 250$ mm

design shear stress in weld,

$$f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25} = \frac{410}{\sqrt{3} \times 1.25}$$

$$= 189.37 \text{ KN}$$

2-1007

Bending stress (f_b) $= \frac{M}{Z}$

$= \frac{6Pe_c}{t_e l_w^2}$

$= \frac{6 \times P \times 120}{7 \times 250^2}$

$= 1.645 \times 10^{-3} P$

Shear stress (q) $= \frac{P}{l_w t_e}$

$= \frac{P}{250 \times 7}$

$= 5.714 \times 10^{-4} P$

$R = \sqrt{f_b^2 + 3q^2}$

$= P \times 4.75 \times 10^{-3}$

For safe design,

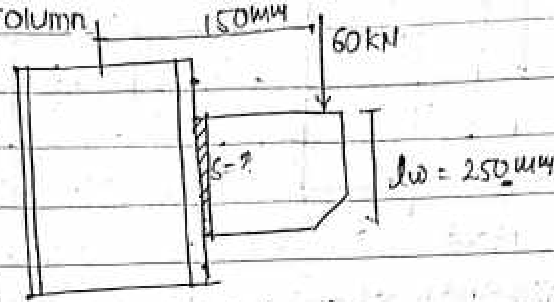
$R \leq f_{wd}$

$P \times 4.75 \times 10^{-3} \leq 189.37$

or, $P \leq \frac{189.37 \times 10^3}{4.75}$

$\therefore \; P \leq 39.873 \text{KN}$

\# Determine the size of fillet weld required to join a bracket plate with a flange of a column.



Now,

Given

$P = 60 \text{KN}$

$e_c = 150 \text{mm}$

$t_e = 0.7S$

$l_w = 250 \text{mm}$

(a) design shear stress of weld $f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25}$

$= 189.37 \text{KN}$

b) shear stress (q) $= \frac{P}{2 t_e l_w} = \frac{60}{2 \times t_e \times 250}$

$= \frac{0.12}{t_e}$

c) Bending stress (f_b) $= \frac{6Pe_c}{2 t_e l_w^2} = \frac{0.432}{t_e}$

d) $R = \sqrt{f_b^2 + 3q^2} = \frac{1}{t_e} \times \sqrt{3 \times 0.12^2 + 0.432^2}$

$= \frac{0.479}{t_e}$

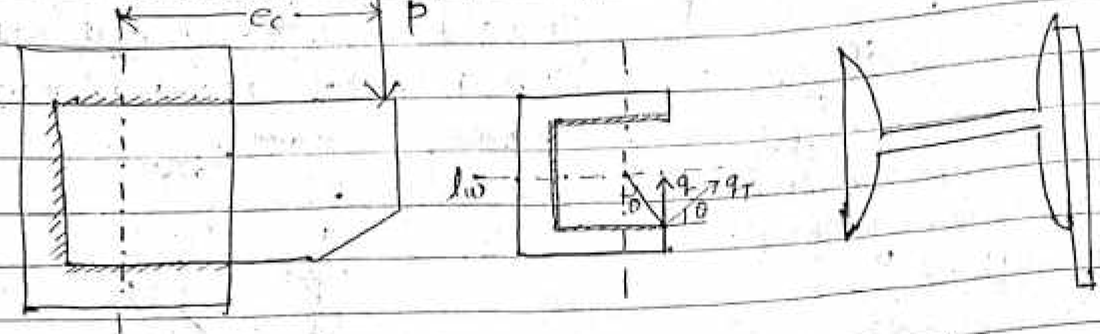
e) So, $R \leq f_{wd}$

or, $\frac{0.479}{t_e} \leq 189.37$

$\therefore \; t_e \geq 2.53 \text{ mm}$

& $S = \frac{t_e}{0.7} = 3.61 \text{mm}$

Welded connection. When load applied parallel to flange



Here,

(a) Vertical shear stress in weld $(q) = \dfrac{P}{t_e l_w}$ $\left(\dfrac{T}{I_p} = \dfrac{\tau}{R}\right)$

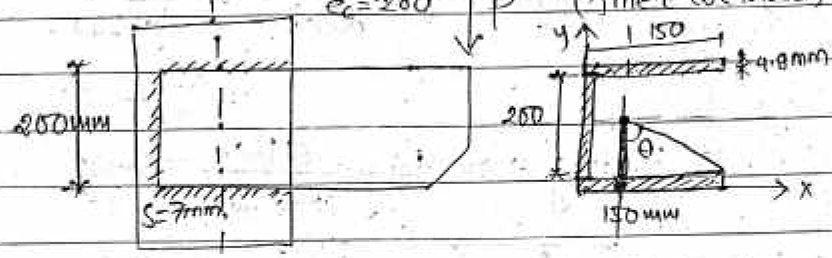
(b) Torsional stress in weld $(q_T) = \dfrac{P \times e \times r_{max}}{I_p}$

(c) shear stress along x-axis $(\sigma_x) = q_T \cos\theta$

$\quad$ Y " $\sigma_y = q_T \sin\theta + q$

(d) Resultant stress $(f) = \sqrt{\sigma_x^2 + \sigma_y^2} \le f_{wd}$

calculate the max value of load p in the following eccentric connection: (fillet welded) of size 7mm



$S = 7mm$

$t_e = 0.7S = 0.7 \times 7$

$= 4.9mm$

$$\bar{X} = \frac{\Sigma A_i x_i}{\Sigma A_i}$$

$$= \frac{200\times4.9\times\frac{4.9}{2} + 2\times150\times4.9\times\frac{150}{2}}{(200\times4.9 + 2\times150\times4.9)}$$

$= 45.98$ mm

Due to symmetry, $\bar{Y} = 104.9$ mm

$$I_{xx} = \frac{4.9\times200^3}{12} + \frac{2\times150\times4.9^3}{12} + 200\times4.9\times(104.9-104.9)^2 + 2\times150\times4.9\times\left(\frac{4.9}{2} - 104.9\right)^2$$

$= 18698731.57$ mm^4

$$I_{yy} = \frac{200\times4.9^3}{12} + \frac{2\times150^3\times4.9}{12} + 200\times4.9\left(\frac{4.9}{2} - 45.98\right)^2 + 2\times150\times4.9\left(\frac{150}{2} - 45.98\right)^2$$

$= 5853150.28$ mm^4

$J_p = I_{xx} + I_{yy}$

$= 24551881.86 \ mm^4$

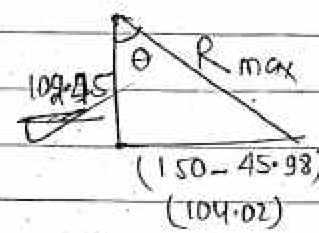
V. Shear stress in weld $(q) = \frac{P_u}{t_e l_w}$

$= \frac{P_u}{4.9 \times 500}$

$= 4.0816 \times 10^{-4} P$

Torsional stress in weld $(q_T) = \frac{P_u \times 200 \times r_{max}}{24551881.86}$

$= 1.189 \times 10^{-3} P_u$



$\therefore R_{max} = \sqrt{109.45^2 + 104.02^2}$

$= 146.00$

$\theta = 45.44°$

So, σ (Resultant) can be calulated as,

$\sigma_x = q_T \cos\theta = 8.343 \times 10^{-4} P_u$

$\sigma_y = 4.0816 \times 10^{-4} P_u + 8.472 \times 10^{-4} P_u$

$= 1.255 \times 10^{-3} P_u$

So. $\sigma = P_u * 1.507 \times 10^{-3}$

Then, design shear stress $(f_{wd}) = \frac{f_u}{\sqrt{3} \times 1.25}$

$= \frac{410}{\sqrt{3} \times 1.25}$

fp.

$= 189.37$ KN

for safe design

So, $\sigma \leq f_{wd}$

$1.507 \times 10^{-3} P_u \leq 189.37$

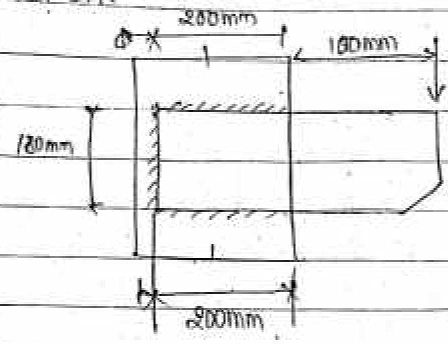
$\therefore P_u \leq 125.6$ KN

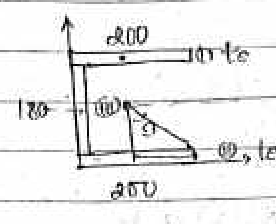
$\therefore$ working load $= \frac{125.6}{1.5}$

$\therefore P = 83.757$ KN

Calculate the size of weld required in following connection:

Given:





Section	Area	$\bar{x}$	$\bar{y}$
(i)	200te	100	$(180+\frac{te}{2})$
(ii)	200te	100	$\frac{te}{2}$
(iii)	180te	$\frac{te}{2}$	(te+90)
	580te		

So, $\bar{X} = \dfrac{100\times 200te + 100\times 200te + 180te\times te/2}{580te}$

$= \dfrac{(40000 + 180te/2)}{580}$

$= (68.96 + \frac{0.31}{2}te) = 68.96 + 0.16te$

$\simeq 68.96\,mm$

$\bar{Y} = (te + 90) \approx 90$

$I_{xx} = \cancel{2\times\frac{200\times te^3}{12}}^{\,0} + 200te\times[\frac{te}{2} - te - 90]^2 + 200\times te\times(180 + \cancel{te} + \frac{te}{2} - \cancel{te} - 90)^2$

$+ \frac{te\times 180^3}{12} + (te + 90 - te - 90)^2\times 180te$

$= 200\times te\times 90^2 + 200\times te\times 90^2 + \frac{te\times 180^3}{12} + 0$

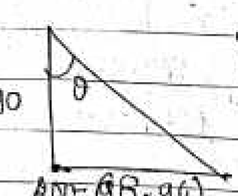
$= 3240000te + 486000te$

$= 3726000te$

$I_{yy} = \frac{180te^3}{12} + 180\times te\times 68.97^2 + 2\times[\frac{te\times 200^3}{12} + 200\times te\times(68.97 - 100)^2]$

$= 2574464.4te$

$I_p = I_{xx} + I_{yy} = 6300464.4te$



$\theta = \tan^{-1}\left(\frac{200-68.96}{90}\right)$

$= 55.52^\circ$

$r_{max} = \sqrt{90^2 + (200-68.96)^2}$

$= 158.970$

$e_c = (100 + 200 - 68.96)$
$= 231.04$

$P = 300\,kN$

$\theta = 55.52^\circ$

Then,

Vertical shear stress $(q) = \frac{P}{t_e l_w}$

$= \frac{300}{t_e \times 580}$

$= \frac{0.5172}{t_e}$

Torsoinal stress $(q_T) = \frac{P e r_{max}}{I_p}$

$= \frac{300 \times 231.04 \times 158.97}{6300464.4\, t_e}$

$= \frac{1.74}{t_e}$

$\sigma_x = q_T \cos\theta$

$= \frac{1.74}{t_e} * \cos 55.52$

$= \frac{0.9907}{t_e}$

$\sigma_y = q_T \sin\theta + q$

$= \frac{1.434}{t_e} + \frac{0.5172}{t_e}$

$= \frac{1.952}{t_e}$

$\therefore R = \sqrt{\sigma_x^2 + \sigma_y^2} = \frac{2.188}{t_e}$

for safe design

$R \leq f_{wd}$

$\frac{2.188}{t_e} \leq 189.37$

$\therefore t_e \leq 11.55\,mm$

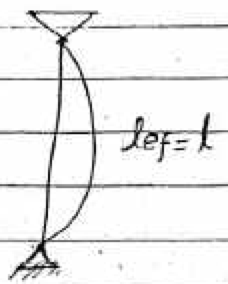
so, $t_e = 0.7S$

$\therefore S = 16.51\,mm.$

CH:6 Compression member

if a structural member is under axial compression as a major action sometime as companied by moment is called compression member.
eg. column, post, vertical member of truss are under axial loading.

Effective length:
It is length betn two points of zero rotation
It depends upon boundary condition



(i) effective length is given by table
IS 800.

$l_{eff} = k \cdot L$

$l_{eff} = 0.65L$

Generally

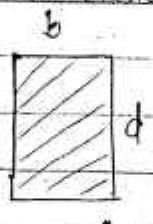
$l_{eff} = 0.8L$

(ii) Radius of gyration: It is a physical quantity whose square multiplied by area gives MOI
ie,

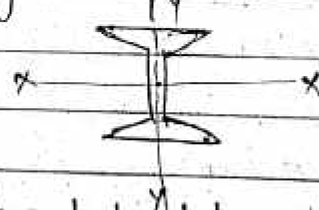
$$r^2 \times A = I$$

$$\therefore r = \sqrt{\frac{I}{A}}$$

Slenderness ratio(λ): It is the ratio of effective length to least lateral dimension. In case of steel structure slenderness ratio is taken as effective length to min. radius of gyration.



$$\lambda = \frac{l_{eff}}{r_{min}}$$



$l_e = \frac{l_{eff}}{b}$

Buckling :- It is the lateral bending observed in compression member. Most of compression membes are failed by buckling

Types of compression members:
1) column
2) Post
3). Strut
4. ~~strqnctions~~ stanchions

Types of buckling

a) flexural
b) Torsional
c) flexural - Torsional

Design of compression member:-

a) Design compressive strength :
load carrying capacity $(P_d) = f_{cd} \times A$
f_{cd} = Design compressive strength
calculation of f_{cd}
Method: ① 7.1.2.1

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

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

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

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad , E = 2 \times 10^{+5}\ N/mm^2$$

$\gamma_{m0} = 1.1$ for buckling

α - Imperfection factor
Table: 7 Clause 7.1.1

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

Buckling class: table 10

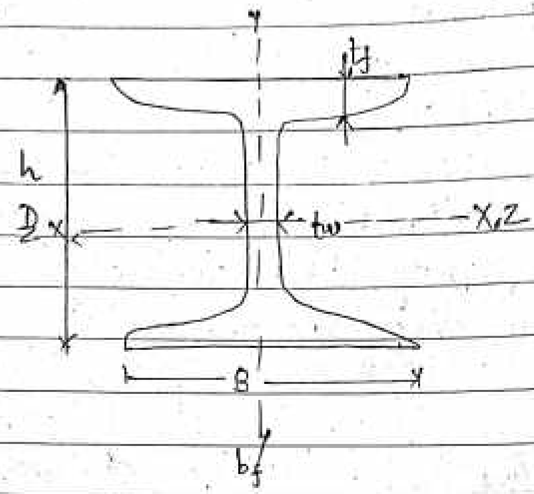
Method ⓘⓘ f_{cd} is calculated from: table: 9 as per buckling class and Slenderness ratio , $f_y \Rightarrow$ generally 250 Mpa
a, b, c, d → class $\frac{KL}{r} \Rightarrow$

Calculate load carrying capacity of SC-250 having unsupported length of 5m, one end of column is restrained again both rotation & translation while other is free on rotation but restrained again translation.

Use: Grade of steel: Fe-410.

from IS: 808-1989

For SC-250



$A = 109\,cm^2$

$D = 250mm$

$B = 250mm$

$T = t_f = 17mm$

$t = t_w = 10mm$

$r_x = 10.7cm$

$r_y = 5.46cm$

Design compressive strength

for x or z axis

$\frac{h}{b_f} = \frac{D}{B} = 1 < 1.2$

$t_f = 17 < 100mm.$

axis	buckling class
so, Z-Z axis	b
Y-Y axis	c

Buckling class

b X-X axis

c Y-Y axis

along X-X axis:

$K = 0.8$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{\frac{\pi^2 \times 2\times10^5}{\left(\frac{0.8\times5000}{107}\right)^2}}} = 0.42$$

$\alpha_b = 0.34$

$$\phi = 0.5[1 + \alpha(0.42 - 0.2) + 0.42^2]$$

$$= 0.5[1 + 0.34\times0.22 + 0.42^2] = 0.626$$

Then,

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

$$= \frac{250}{1.10[0.626 + (0.626^2 - 0.42^2)^{0.5}]}$$

$$= 208.58 \ N/mm^2 \le f_y/\gamma_{m0}$$

$$= 208.58 \le 227.27 \ N/mm^2 \quad (OK!)$$

along Z-Z axis:

$K = 0.8$

$$\lambda = \sqrt{\frac{250}{\frac{\cancel{0.8 \times 7}\pi^2 \times 2 \times 10^5}{\left(\frac{0.8 \times 5000}{54.60}\right)^2}}} = \cancel{1.46}\ 0.824$$

$\alpha_c = 0.49$

$$\phi = 0.5[1 + 0.49(\cancel{1.46}\ 0.824 - 0.20) + \cancel{1.46^2}\ 0.824^2]$$

$$= \cancel{1.8745}\ 0.9923$$

$$\therefore f_{cd} = \cancel{\frac{250}{1.10 \times [1.8745 + (1.8745^2 - 1.46^2)^{0.5}]}}$$

$$= \frac{250}{1.10[0.9923 + (0.9923^2 - 0.824^2)^{0.5}]}$$

$$= 147.066 \le 227.27 \ KN/m^2$$

Method (ii) along x-x axis, class: b

Slenderness $(\lambda_x) = \dfrac{l_{eff}}{r_x} = \dfrac{0.8 \times 5000}{107} = 37.38$

$(\lambda_y) = \dfrac{l_{eff}}{r_y} = \dfrac{0.8 \times 5000}{54.6} = 73.26$

from table:

λ	f_{cd}
30	216
37.38	??
40	206

$\dfrac{y_2 - y_1}{x_2 - x_1} = \dfrac{y - y_1}{x - x_1}$

$\dfrac{206 - 216}{40 - 30} = \dfrac{y - 216}{37.38 - 30}$

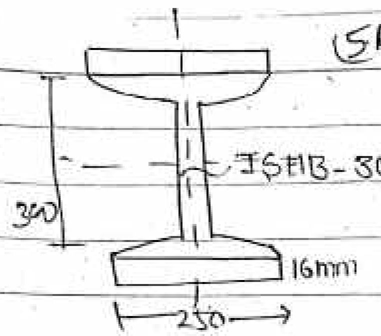
$\therefore y = 208.62 \ N/mm^2$

along y-y axis, class: c

λ	f_{cd}
70	152
80	136
73.26	146.784

$\therefore f_{cd} = 146.784 \ N/mm^2$

Calculate ultimate load carrying capacity of section (following builtup) of length 7m if both ends are fixed.



For ISHB 300 @ 63 Kg/m

D = 300mm

B = 250mm

$t_f = T = 10.6$mm

$t_w = t = 9.4$ mm

$I_{xx} = 13000\ cm^4$

$I_{yy} = 2250\ cm^4$

$r_x = 12.7$ cm

$r_y = 5.29$ cm

$A = 80.2\ cm^2$

$I_{xx}' = [I_{xx} + Ah^3]_{HB} + [I_{xx} + Ah^2]_{plate} \times 2$

$= 13000 \times 10^4 + \left[\frac{250 \times 16^3}{12} + 250 \times 16 \times (150+8)^2\right] \times 2$

$= 32988.27 \times 10^4\ mm^4$

$I_{yy}' = 2250 \times 10^4 + 0 + 2\left[\frac{16 \times 250^3}{12}\right] \cancel{\times 2}$

$= 6416.67 \times 10^4\ mm^4$

so, $I_{yy}' < I_{x'x'}$

$A' = 80.2 \times 10^2 + 250 \times 16 \times 2$

$= 160.2 \times 10^2\ mm^2$

$r_y' = \sqrt{\frac{I_{y'y'}}{A'}} =$

$= \sqrt{\frac{6416.67 \times 10^4}{160.2 \times 10^2}}$

$= 63.29$ mm

for Builtup class : Buckling class is c   Page: 44

so,

for Buckling class: c , $\alpha = 0.49$

for both end fixed, effective length = 0.65L

$$\lambda = \sqrt{\frac{fy}{fcc}} = \sqrt{\frac{250}{fcc}}$$

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

$$= \frac{\pi^2 \times 2\times10^5}{\left(\frac{0.65\times7000}{63.29}\right)^2} =$$

$\therefore fcc =$

$$\therefore \lambda = \sqrt{\frac{250}{381.562}} = 0.809$$

$\alpha = 0.49$

$$\phi = 0.5\left[1 + 0.49(0.809 - 0.2) + 0.809^2\right]$$

$$= 0.976$$

$$\therefore fcd = \frac{fy/\gamma_{mo}}{\left[\phi + (\phi^2 - \lambda^2)^{0.5}\right]} = \frac{250}{1.10\left[0.976 + (0.976^2 - 0.809^2)^{0.5}\right]}$$

$$= 149.327 \text{ Mpa}$$

Method (II)

along YY axis: class: C

$$\text{Slenderness } (\lambda_y) = \frac{l_{eff}}{r_y} = \frac{0.65\times7000}{63.29}$$

$$= 71.891$$

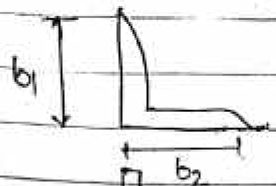
from table.

λ	~~fcd~~ χ	fcd
70	0.670	152 Mpa
71.891	?	148.974 Mpa
80	0.600	136 Mpa

$\therefore$ ~~fcd~~ χ at 71.891 = 0.657

fcd = 148.874 Mpa

Angle section : pag: 49 7.5.1.2 [one leg loaded]
[Use Method I]



$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

for loaded through one leg:

k_1, k_2, k_3 from table: 12

$$\lambda_{vv} = \frac{\left(\frac{L}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

90 x 90 x 12
$b_1 \times b_2 \times t$

$r_{vv} \rightarrow$ radius of gyration (min) about minor axis.

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

aru process same:
(ISA)

Calculate load carrying of 110 x 110 x 10 mm of length $l = 3m$ between support center is provided with 3 nos. bolt at each end and it is hinged supported are both end.

Soln:

For ISA 110 x 110 x 10 table (5.1)

$A = \cancel{19\ cm^2}\ 2110\ mm^2$

$r_{vv} = \cancel{\ \ }\ 21.6mm$

From table 12,

$k_1 = 0.70$

$k_2 = 0.60$

$k_3 = 5.00$

so, $\cancel{\lambda_{vv}} \Rightarrow \frac{L}{r_{vv}} = \frac{3000}{21.6} = 138.889$

$$\lambda_{vv} = \frac{138.889}{\left(\frac{250}{250}\right)^{0.5} \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$= 1.563$

$$\lambda_\phi = \frac{\cancel{(100-110)} \times (110+110)}{2 \times 10 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}} \times 1}$$

$= 0.124$

$$So, \lambda_e = \sqrt{0.70 + 0.60 \times 1.563^2 + 5 \times 0.124^2}$$

$$= 1.498$$

for angle: Buckling class: C

$$So, \alpha = 0.49$$

Then using Method (I)

$$\phi = 0.5[1 + 0.49(1.498 - 0.2) + 1.498^2]$$
$$= 1.940$$

$$fcd = \frac{250}{1.10 \times [1.940 + [1.940^2 - 1.497^2]^{0.5}]}$$
$$= \cancel{73.590} \text{ Mpa}$$
$$= 71.637 \text{ Mpa} \qquad \therefore P = fcd \times A = 151.475 \text{ KN}$$

Design: compression member using single unequal angle to carry factored compression of 120 kN if C-C length of column is 3.5m
if not state: let fixed end.
no of bolt ≥ 2

Soln:-

$$P = 120 \text{ kN}$$

$$\text{area required} = \frac{P}{fcd} = \frac{P}{(0.4 \text{ to } 0.6) fy}$$

$$= \frac{120 \times 1000}{0.4 \times 250}$$

$$= 12 \text{ cm}^2$$

from Table: taking, section of unequal angle
6.1 ISA: 90 × 60 × 10

load carrying capacity $\geq$ 120 kN

Then,

$A = 14\ cm^2$

$r_{w} = 1.27\ cm$

for both end fixed:

$k_1 = 0.20$

$k_2 = 0.35$

$k_3 = 20$

$$\lambda_{w} = \frac{\frac{3500}{12.7}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$= 0.0031 \times 1000 = \cancel{3100}$

$$\lambda_{\phi} = \frac{(90+60)}{2 \times 10 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$= 0.084$

so, $\lambda_e = \sqrt{0.20 + 0.35 \times 3.101^2 + 20 \times 0.084^2}$

$= 1.925$

~~so~~, buckling class = C, $\alpha = 0.49$

$\therefore \phi = 0.5[1 + 0.49(1.925 - 0.2) + 1.925^2]$

$= 2.776$

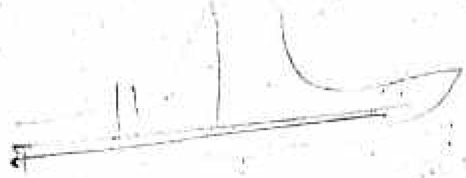
so, $$f_{cd} = \frac{250}{1.10 \times [2.776 + (2.776^2 - 1.925^2)^{0.5}]}$$

$= 47.585$ kN/mm $\cancel{< 120 kN}$ ~~(not ok)~~

$P = f_{cd} \times A = 66.619$ kN < 120 kN not ok

again taking IS: angle:

IS: 125 X 95 X 10



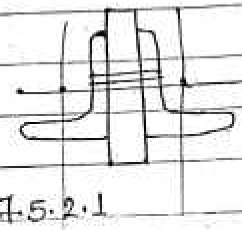
Angle section

Double angle section:

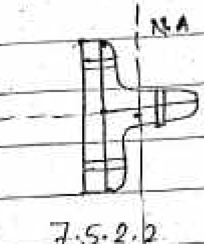
(i) Angle section connected to opposite side of gusset plate.

(ii) " " " on same " " " "

(i)



7.5.2.1



7.5.2.2

Calculate load carrying capacity of two ISA 100X100X12 of unsupported length 4m, if angles are connected –

(i) on same sides of gusset plate of 16 mm thick.

(ii) on opposite " " " " " " "

Soln: for ISA 100 X 100 X 12

Area = 22.6 cm^2

$C_x = C_y = 2.92$ cm

$r_x = 3.03$ "

$r_y = 3.03$ "

$r_{min} = 1.94$ cm

$I_{xx} = I_{yy} = 207$ cm^4

Now,

$$\lambda_{vv} = \frac{\frac{l}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{4000}{19.4}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$$= 0.0023 \times 1000 = 2.3$$

$$\lambda_\phi = \frac{(b_1 + b_2)}{2 t \times \varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{200}{2 \times 12 \times 1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$$= 0.0937$$

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2} = \cancel{0.615}\ 1.492$$

$$\left.\begin{aligned} k_1 &= 0.20 \\ k_2 &= 0.35 \\ k_3 &= 20 \end{aligned}\right\} \text{for fixed end}$$

for built up member, buckling class = c

so $\alpha = 0.49$

so

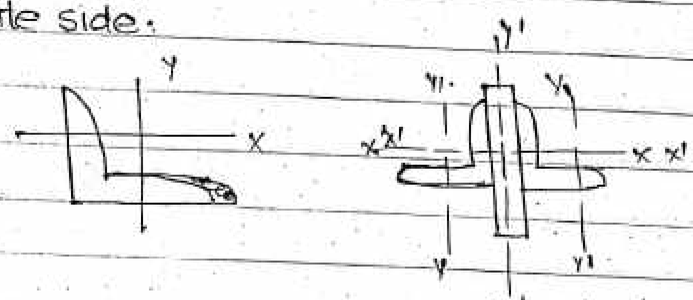
$$\phi = 0.5\left[1 + 0.49(1.492 - 0.2) + 1.492^2\right]$$
$$= 1.93$$

so
$$f_{cd} = \frac{250}{1.10 \times \left[1.93 + (1.93^2 - 1.492^2)^{0.5}\right]}$$
$$= 72.05$$

so, load carrying capacity $= \cancel{f_{cd} \times A} = 72.05 \times A$
$= 162.8\ kN$

Total load carrying capacity $= 2 \times 162.8$
$= 325.6\ kN$

opposite side.



$$I_{x'x'} = 2 \times 207 = 414\ cm^4$$
$$I_{y'y'} = 2\left[207 + 22.6 \times \left(2.92 + \frac{1.6}{2}\right)^2\right]$$
$$= 2\left[I_{yy} + A \times \left(C_x + \frac{t}{2}\right)^2\right]$$
$$= 1039.41\ cm^4$$
$$r_{x'} = \sqrt{\frac{414}{2 \times 22.6}} = 3.03\ cm$$
$$r_{y'} = \sqrt{\frac{1039.41}{2 \times 22.6}} = 4.796\ cm$$

Y axis वाट measure गरेको

$L =$ ⊥ to gusset plate $= Kl_y$

$L = \parallel$ to " " $= Kl_x$

Now,

$Kl_y = 1 \times 4 = 4$ m

$Kl_x = 0.85 \times 4 = 3.4$ m

Then,

$\lambda_x = \frac{Kl_x}{r_x} = \frac{3.4 \times 100}{3.03} \doteq 112.21$

$\lambda_y = \frac{400}{4.796} = 83.403$

Now, buckling class is C

$\alpha = 0.49$

from table 9C

along x-axis		along y axis
110	94.6	
120	83.7	

So, at $\lambda_x = 112.21$

$\therefore f_{cd} = 92.19$

$\therefore$ load carrying capacity $= f_{cd} \times A$

$= 92.19 \times 22.6 \times 10^2$

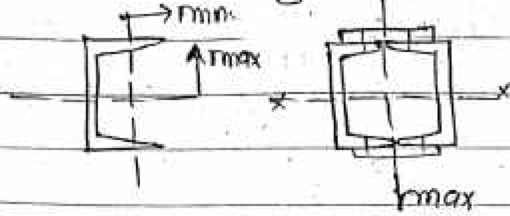
$= 208.35$ kN

So, Total load carrying capacity $= 2 \times 208.35$

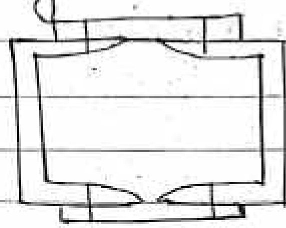
$= 416.703$ kN

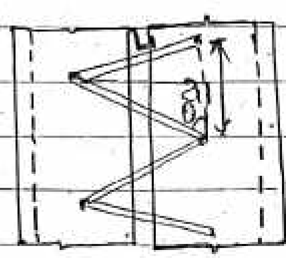
Built up column:

Two or more rolled section are connected together such that weaker axis becomes the stronger one & vice versa.

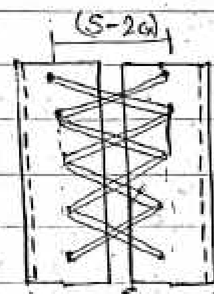


Lacing system: It is used to connect two section of builtup column and carries shear developed due to lateral displacement caused by axial loading

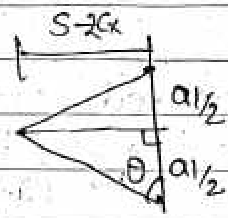




single lacing system



Double lacing system.



$\sin\theta = \frac{S-2C_x}{L}$

$\therefore L = (S-2C_x)\operatorname{cosec}\theta$

$\tan\theta = \frac{S-2C_x}{\left(\frac{a_1}{2}\right)}$

$\therefore a_1 = 2(S-2C_x)\cot\theta$

Design specification:

Lacing should be mirror image of one another on both sides of column.

effective slenderness ratio is taken as 1.05 times most critical slenderness ration of column to consider shear deformation

width of lacing bar is taken as greater than 3 times nominal diameter of bolt used.

Thickness of lacing bar shouldn't be less than $\left(\frac{1}{40}\right)^{th}$ the length of lacing bar incase of single lacing & $\left(\frac{1}{60}\right)^{th}$ in case of double lacing.

* Angle of inclination is taken betn 40 to 70°.
  (Transition of load is smooth)
* Spacing [a_1] of lacing bar should be such that $\frac{a_1}{r_1} \leq 0.7 \times$ critical slenderness ratio. ≤ 50

  r_1 – individual radius of gyration

* lacing should be able to carry 2.5% of axial load.
  [shear force = 2.5% of axial load] during buckling

* Slenderness ratio of lacing shouldn't be greater than 145.

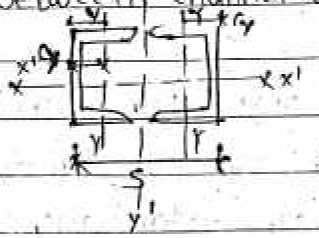
Design steps laced column:

1: Calculate required sectional area of column by,

$$A_{rea} = \frac{P}{(0.4 \text{ to } 0.6) f_y}$$

2. Select channel section such that $A \geq \frac{Area}{2}$

3. Calculating spacing between channel section such that: $I_{y'y'} \geq I_{x'x'}$



$$2\left[I_{xx} + A \times \left(\frac{S}{2} - C_y\right)^2\right] \geq 2 I_{xx}$$

4. Check load carrying capacity of column.

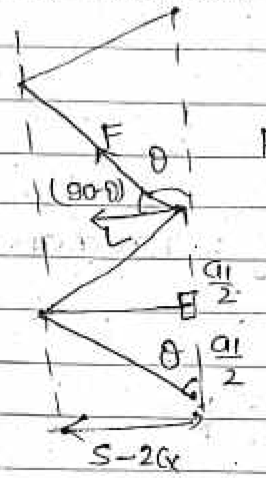
5. adopt angle of inclination

6. Calculate length of lacing & spacing of lacing & also check for spacing of lacing, $\left[\frac{a_1}{r_1}\right]$ criteria

7. Calculate width and thickness of lacing bar.
8. Calculate load carrying by lacing bar.
9. Check for compressive strength of "
10. Check for Tensile strength of "
11. connection design.
12. Design of tie plate.

Calculation of shear force:

$V = 2.5\%$ of axial load



$F\cos(90-\theta) = \frac{V}{2}$

$\therefore F = \frac{V}{2\sin\theta}$

for double lacing

$F = \frac{V}{4\sin\theta}$

Design builtup column of 7m unsupported length carrying factored axial load of 1500 kN also design lacing system to connect two channels back to back using bolted connection.

Soln:

Step 1:

$$Area = \frac{P}{0.6f_y} = \frac{1500\times10^3}{0.6\times250}$$

$$= 10{,}000\ mm^2$$

step: 2 Select channel section such that

$$A \geq \frac{A_{req}}{2} = 5000\ mm^2$$

individual channel

from SA, taking,

MC 300 @ 46.2 kg/m (Sloping flange channel)

so, $A = 58.8\ cm^2$

step: 2 contd: $D = h = 300$ mm

$B = b_f = 94$ mm

$t_f = T = 12.6$ mm

$t = t_w = 12$ mm

$r_x = 11.2$ cm

$r_y =$ ~~2.68 cm~~ 2.52 cm

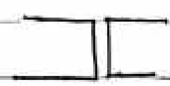
$I_{xx} = 7350\ cm^4$

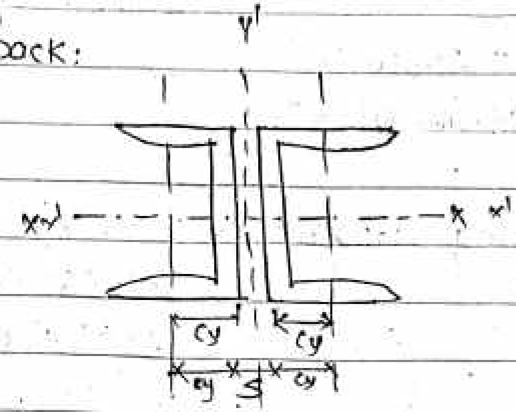
$I_{yy} =$ ~~424~~ 375 cm^4

$A = 58.8\ cm^2$

$C_y =$ ~~2.29~~ cm = 2.22 cm

Step: 3, finding the spacing :
for back to back:





$I_{y'y'} \geq I_{x'x'}$

$h_y = (S + 2c_y)$

$$\cancel{2}\left[I_{yy} + A \times h_y^2\right] > \cancel{2}\left[I_{xx} + \cancel{A h_x^2}^{\,0}\right]$$

or, $375 \times 10^4 + 5850 \times \left(\frac{S}{2} + c_y\right)^2 > 7350 \times 10^4$

or $\left(\frac{S}{2} + c_y\right)^2 \geq \cancel{0.335}\ \frac{6975 \times 10^4}{5850}$

or, $S > 173.243$ mm

$\therefore S = 180$ mm

Step: 4:

$$\lambda = \frac{KL}{r_{min}} = \frac{0.65 \times 7000}{r_x}$$

$\lambda_{actual} = 40.625$

$\lambda_{design} = 1.05 \times 40.625$
$= 42.656$

For built up section, buckling class is C
$\therefore \alpha = 0.49$

Step 5: adopt angle of inclination: $\theta = 60°$

~~Ste~~ from table 9(c)

fcd for $\lambda = 42.656$

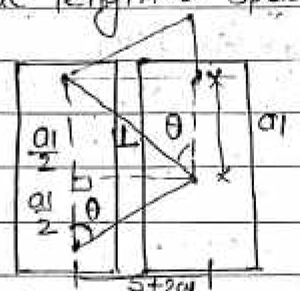
$\therefore f_{cd} = 194.016\ N/mm^2$

load carrying capacity $= f_{cd} \times A$

$= 194.016 \times 58.8 \times 10^2$

$= 2281.63\ kN > 1500\ kN$ OK!

Step: 6 calculate length & spacing of lacing:



Now, $\tan\theta = \left(\frac{S+2C_y}{a_1/2}\right)$

$\therefore a_1 = \frac{2(S+2C_y)}{\tan\theta}$

$= 2??.97\ mm$

So, $\sin\theta = \frac{S+2C_y}{l}$

$l = \cancel{242.97\ cm}\ 259.11\ mm$

also, $\frac{a_1}{r_1} = \frac{259.11}{25.2} = 10.284 \le 50$

$\le 0.7 \times 42.65 = 29.855$

7: Calculate width & thickness of lacing bar:

adopt M20 bolts of Pr. cl. 4.6

so, $f_{ub} = 400$ Mpa

$f_{yb} = 240$ Mpa

Width of lacing bar $\geq 3d = 3 \times 20$

$= 60$mm

$\simeq 80$ mm

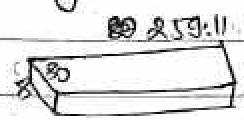
$$\text{thickness} = \frac{1}{40} \times l$$

$$= \frac{259.11}{40}$$

$= 6.5$ mm

$\simeq 8$ mm

adopt lacing bar of 80 x 8 mm.



for single lacing $l_{eff} = l = 80$ mm

$$I_{min} = \frac{bd^3}{12} = \frac{80 \times 8^3}{12} = 3413.33$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{bt^3}{12\,bt}} = \frac{t}{\sqrt{12}}$$

$$= \frac{8}{\sqrt{12}} = 2.31 \text{mm}$$

$$\lambda = \frac{KL}{r_{min}} = \frac{1 \times 259.11}{2.31}$$

$= 112.19 < 145$ ok!

Step: 8:

$V = 2.5\%$ of axial load

$= \frac{2.5}{100} \times 1500$

$= 37.5$ kN

$F = \frac{V}{2\sin\theta} = \frac{37.5}{2\times\frac{\sqrt{3}}{2}}$

$= 21.65$ kN

Step: 9 from table 9C [Page 44 a12 Table 10]

2nd last, solid bar

buckling class C

$\alpha = 0.49$

$\lambda = 112.2$

$\therefore f_{cd} = 92.202$ N/mm²

$\therefore$ load carrying capacity $= 92.202 \times 80 \times 8$

$= 59.00$ kN > 21.65 kN (OK!)

Step: 10: check tensile strength of bar.

yielding $T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{80\times8\times250}{1.10}$

$= 145.45$ kN > 26.15 kN. OK!

Step 11:- for M20 bolt of pr. Cl 4.6

$n_n = 1$

$V_{dsb} = \frac{400}{\sqrt{3}} \times \frac{0.78}{1.25} \times \frac{\pi \times 20^2}{4}$

$= 45.27$ kN

शति मात्र भरेनी कुकुल

Bearing capacity :- $V_{dpb} = \frac{2.5 K_b d t (f_u/f_{ub})}{1.25}$

$= \frac{2.5 \times 0.5 \times 20 \times 8 \times 400}{1.25}$ assume $K_b = 0.5$

$= 64 kN$

So, Bolt value $= 45.27 kN$

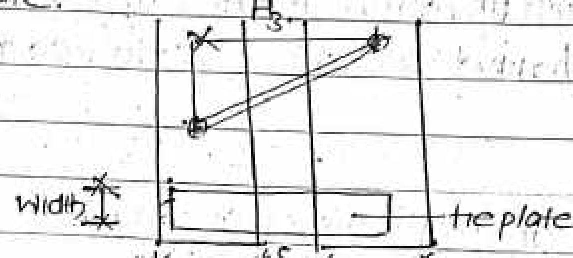
So no of bolt $= \frac{load}{Bolt\ value}$

$= \frac{21.65}{45.27}$

$= 0.478 \approx 1$

Provide 1 M20 bolt of gr.cl. 4.6 for lacing bar.

Step:12 Design of tie plate:



width of tie plate $\geq (S + 2C_y)$

≥ 224.8

≈ 230 mm

total length $= S + 2 \times B$

$= 180 + 2 \times 94$

$= 368$ mm छेउमा gap छोडेर 4/4 mm दुईतिर।

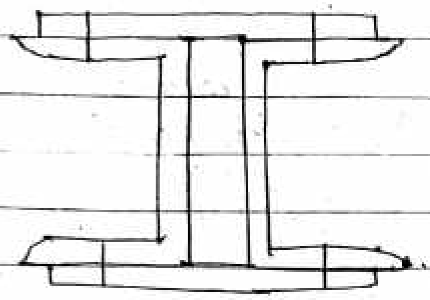
thickness $\geq \frac{1}{50} \times (S + 2C_y)$

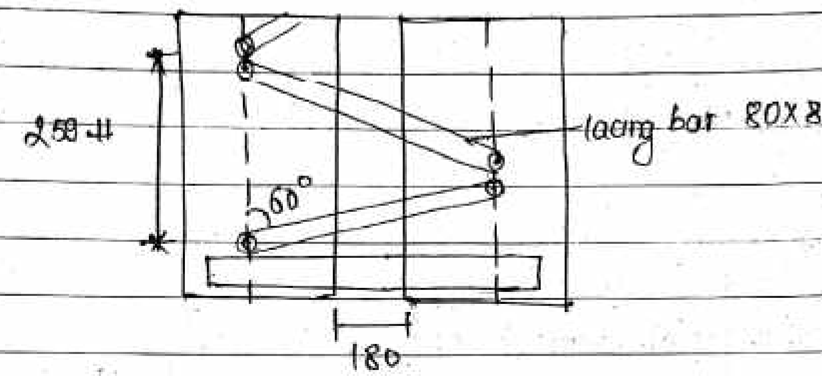
$\geq \frac{224.8}{50}$

≥ 4.5 mm

≥ 8 mm

provide tie plate of length $360 \times 230 \times 8$ mm.





Design a built up column of 10m carrying factored load of 1000kN. Using two channel section face to face or toe to toe.

Soln:-

Step 1 Calculate area required of column,

$$\therefore Area = \frac{P}{0.6 f_y} = \frac{1000 \times 10^3}{0.6 \times 250}$$

$$= 6666.67 mm^2$$

step: 2 Select channel section such that,

$A_c \geq \frac{Area}{2}$ of individual channel

$$\geq \frac{6666.67}{2}$$

$$\geq 3333.33 mm^2$$

$$A \geq 33.33 cm^2$$

from table 6A, taking sloping flange channel
MC 225 @ 30.7 kg/m

$A = 39 cm^2$ $\qquad$ $I_{yy} = 219 cm^4$

$D = 225 mm$ $\qquad$ $r_x = 8.71 cm$

$B = 82$ " $\qquad$ $r_y = 2.37 cm$

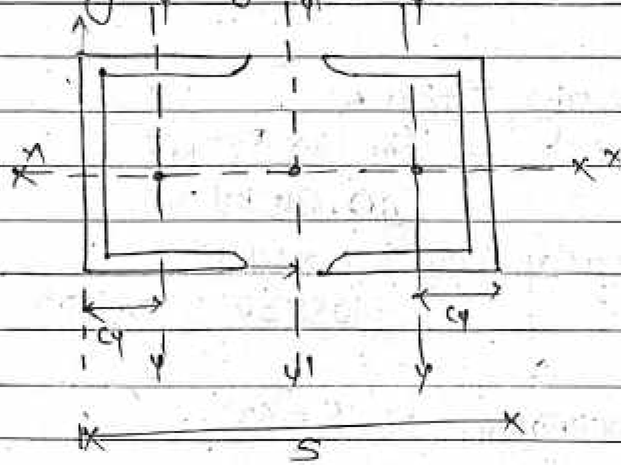
$t_w = 9$ "

$T_f = 12.4 mm$

$C_y = 2.22 cm$

$I_{xx} = 2960 cm^4$

Step:3 calculating spacing for toe to toe connection



$I_{y'y'} \geq I_{x'x'}$

$2\left[I_{yy} + A\times(S/2 - C_y)^2\right] \geq 2\left[I_{xx} + A\times \cancel{r^2}^{\,0}\right]$

or, $\cancel{2}\left[21900\times10^4 + 3900(S/2 - 2.22)^2\right] \geq \cancel{2}\left[2960\times10^4\right]$

or, $3900(S/2 - 2.22)^2 \geq 2741\times10^4$

or, $(S/2 - 2.22)^2 \geq 7028.20$

or, $S/2 \geq (83.83 + 22.20)$

$\therefore$ $S \geq 212.06 mm$

take $S = \cancel{180 mm}$ $212.06 mm$

$= 220 mm$

Step 4 calculate slenderness ratio & load carrying capacity

$$\lambda_{actual} = \frac{KL}{r_{min.}} = \frac{0.65 \times 10000}{87.1}$$

$$= 74.62$$

$$\lambda_{design} = 1.05 \times \lambda_{actual}$$

$$= 78.358$$

for builtup section buckling class = c

$$\alpha = 0.49$$

from table 9c

70	152
80	136

for $\lambda = 78.358$

$$f_{cd} = 138.627 \ N/mm^2$$

so, load carrying capacity $= f_{cd} \times A$

$$= 138.627 \times 39 \times 10^2$$

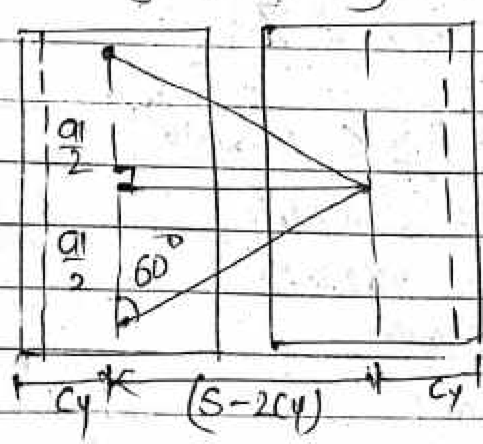
$$= 540.645 \ kN$$

Total load carrying capacity $= 2 \times 540.645$

$$= 1081.30 \ kN > 1000 \ kN \ (OK!)$$

Step:5: let, angle of inclination be $\theta = 60°$

Step:6 Calculate length & spacing of lacing



$$a_1 = \frac{2(S-2C_y)}{\tan\theta} = 202.72 \ mm$$

$$l = \frac{(S-2C_y)}{\sin\theta} = 202.72 \ mm$$

Check. $\frac{a_1}{r} = \frac{202.72}{23.7}$

$$= 8.55 \le 50$$

$$\le 0.7 \times \lambda_d$$

Step:7 width & thickness of lacing bar:

adopt M20 bolt of pr cl 4.6 — or, we can adopt any width of lacing bar

width of lacing bar $\geq 3d = 3\times 20$
$= 60$ mm
≈ 80 mm

thickness $(t) = \frac{1}{40}\times l = \frac{202.72}{40}$
$= 5.068$
≈ 8 mm

for double lacing
$t = \left(\frac{l}{60}\right)$
lacing length $= l_{eff} = 0.7\, l$
$F = \frac{V}{4\sin\theta}$

adopt lacing bar of 80x8 mm

for single lacing $l = l_{eff} =$ ~~80mm~~ 202.72 mm

$r_{min} = \frac{t}{\sqrt{12}} = \frac{8}{\sqrt{12}} = 2.31$ mm

$\lambda = \frac{KL}{r_{min}} = \frac{1\times 202.72}{2.31}$

Step: 8

$= 87.75 < 145$ OK!

so, fcd for $\lambda = 87.75$ can be calculated such that for solid section buckling class C

$\therefore$ table 9C gives

$f_{cd} = 124.375\ N/mm^2$

136
121

Step: 9

$V = 2.5\%$ of axial load $= \frac{2.5}{100}\times 1000$
$= 25$ kN

$F = \frac{V}{2\sin\theta} = 14.43$ kN

load carrying capacity of lacing bar $= 124.375 \times 80 \times 8 = 79.6$ kN
> 14.43 kN

Front to front - connection → for stress $dist^n$

Step 10: Tensile strength:

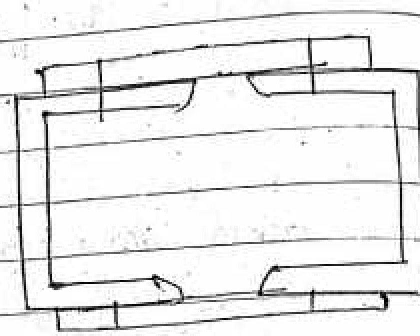
Yielding strength $T_{dg} = \frac{A_g f_y}{1.10} = \frac{80 \times 8 \times 250}{1.10}$

$= 145.25 > 14.43 \text{ kN}$ OK!

Step 11: for M20 bold prcl. 4.6

$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times 0.78 \frac{\pi \times 20^2}{4}$

$= 45.27 \text{ kN}$



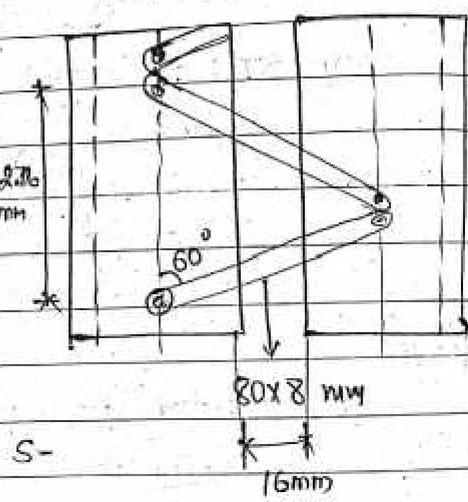
Step:12: assume $k_b = 0.5$

$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 8 \times 400}{1.25}$

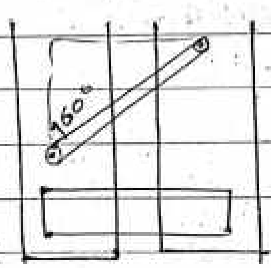
$= 64 \text{ kN}$

So Bolt value $= 45.27$ kN

no of bolt $= \frac{14.43}{45.27} = 0.318 \approx 1$ bolt



Design of tie plate:



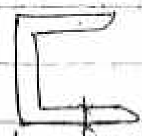
width of tieplate ≥ S-

180 - 2132
180
164

for weld:

Step 11: welding

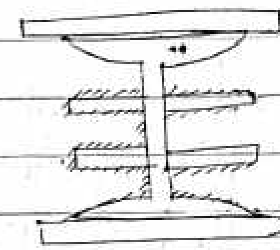
size of weld:



t_f = 10.4mm

$S_{max} = t_{min} - 1.5$ lacing thickness = 8 mm

$= 8 - 1.5 = 6.5mm$



$S_{min} = 5mm$

so, adopt, $S = 6$ mm

and

$t_e = 0.7S$

$= 4.2mm$

Then,

$f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25}$

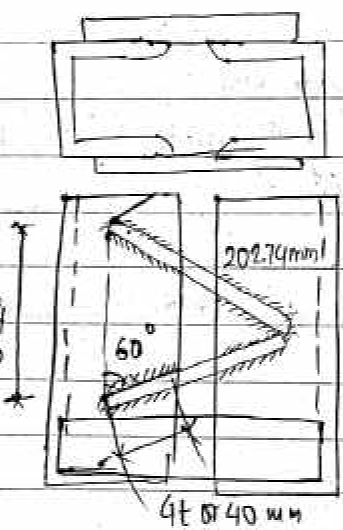
$= \frac{400}{\sqrt{3} \times 1.25} = 189.37$ Mpa

P (load carrying capacity) = $f_{wd} \times l_e \times t_e$

$14.43 \times 10^3 = 189.37 \times l_e \times 4.2$

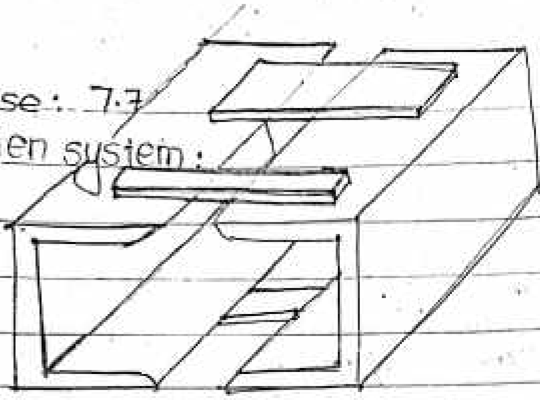
$\therefore l_e = 18.14mm$

so provide l_e = as per required.

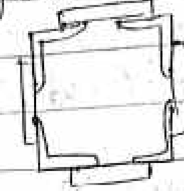


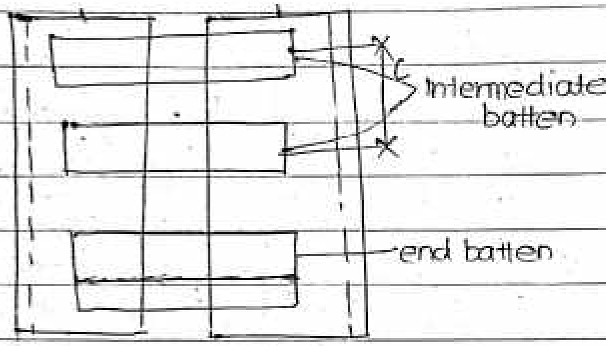
clause: 7.7

Batten system:



angle ⊥ n=4





Specification for design

(i) critical slenderness ratio is multiplied by 1.1 to obtain design slenderness ratio to consider shear deformation

(ii) Spacing of batten should be such that $\frac{C}{r}$ should be less than $0.7 \times$ critical slenderness ratio. and ≤ 50

(iii) Size of batten is determined by shear force and Bending moment acting on batten and given by:

$$V_b = \frac{V_t C}{NS}$$

V_t - transverse shear force

C - c/c distance bet^n batten

$$M = \frac{V_t C}{2N}$$

$S = (\cancel{S}' \pm 2C_y) = (S' \pm 2C_y)$

S' - spacing

(iv) width of batten shouldn't be less than c-c distance bet^n connection in batten:

$$\text{width} \geq (S' - 2C_y)$$

$$\text{thickness} \geq \frac{1}{50}(S' - 2C_y)$$

Width of intermediate batten = $(3/4)^{th}$ of end batten.

Step: 4 samma same:

Design a builtup column of span of 9m length with both hinged and carrying an axl load of 900kN. Builtup column should consists 2 channel section connected front to front using batten system:

Soln:

Step 1: Calculate the area required

$$\text{Area} = \frac{\text{factored load}}{0.6 f_y}$$

$$= \frac{1.5 \times 900 \times 1000}{0.6 \times 250}$$

$$= 9000 mm^2$$

Step: 2 Selection of channel section: such that

$$A \geq \frac{\text{Area}}{2}$$

$$\geq \frac{9000}{2}$$

$$\geq 4500 \ mm^2$$

$$\geq 45 \ cm^2$$

take MC300 @ 46.2 kg/m

$A = 58.8 \ cm^2$

$D = 300 \ mm$ $\qquad$ $I_{xx} = 7350 \ cm^4$

$t_f = 13.6 \ mm$ $\qquad$ $I_{yy} = 375$ "

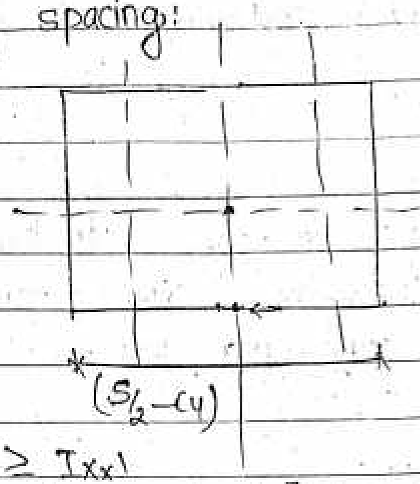
$t_w = 12 mm$ $\qquad$ $c_y = 2.22 \ cm$

$B = 94 \ mm$

$r_x = 11.2 \ cm$

$r_y = 2.52 cm$

Step: 3 finding spacing:



$I_{yy'} \geq I_{xx'}$

$\cancel{2}[I_{yy} + A \times (S/2 - C_y)^2] \geq \cancel{2}[I_{xx} + 0]$

or, $375 \times 10^4 + 58.8 \times 100 (S/2 - 22.2)^2 \geq 7350 \times 10^4$

or $\left(\frac{S}{2} - 22.2\right)^2 \geq 11862.25$

or $S/2 - 22.2 \geq 108.91$

or, $S \geq 262.22$

$\therefore S = 270$ mm

[required Area 0.6 भन्दा [illegible]
Area avilable [illegible]
Max [illegible]

Step: 4 Calculate slenderness ratio

$$\lambda_{actual} = \frac{KL}{r_{min}} = \frac{KL}{r_x}$$

$$= \frac{1 \times 9000}{112}$$

$$= 80.357$$

136
121

$$\lambda_{design} = 1.1 \times \lambda_{actual}$$

$$= 88.393$$

for $\lambda = 88.393$ & builtup section buckling class = c
table 9c

$f_{cd} = 123.41$ MPa

$\therefore$ load carrying capacity $= f_{cd} \times A$

$= 123.41 \times 58.8 \times 10^2$

$= 725.65$ kN

Total load carrying capacity $= 2 \times 725.65$

$= 1451.3$ kN > 900 kN (OK)

Batten system:

Spacing of batten can be calculated as

$\frac{C}{r_y} \leq 0.7 \times$ slenderness ratio

$\leq 0.7 \times 88.392$

$\frac{C}{2.52} \leq 50$

$\therefore C \leq 1260$ mm

adopt $C = 1000$mm

Now,

shear force on batten $(V) = 2.5\%$ of Axial load

$= \frac{2.5}{100} \times 1350$

$V = 33.75$ kN

$n = 2$ (no. of shear plane

$\therefore S = S' - 2c_y$

$= 270 - 2 \times 22.2$

$= 225.6$mm

so, shear force on batten $= \frac{VC}{NS} = \frac{33.75 \times 1000}{2 \times 225.6}$

$= 79.8$ kN

Moment on batten $= \frac{VC}{2N} = \frac{33.75 \times 1000}{2 \times 2}$

$= 8437.5$ kNm

Size of end batten:

Depth of end batten $\geq S$

≥ 225.6

≈ 230 mm

length of batten $= 250$ (Just less than S')

thickness $\geq \frac{1}{50} \times S = \frac{1}{50} \times 225.6$

$= 4.512$mm

≈ 8mm

adopt 250X 230X 8 mm of batten.

for intermediate batten:

depth of intermediate batten = $\frac{3}{4}$ x depth of Batten.

$= \frac{3}{4} \times 230$

$= 172.5$ mm

$\simeq 180$ mm

adopt: 250x180X 8 mm intermediate batten.

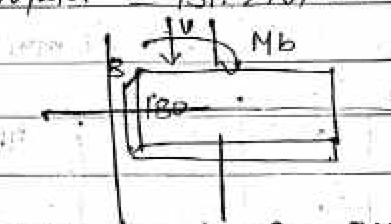
Check for size of batten

Pag. 59 : 8.4: Shear:

Shear stress on batten $= \frac{V}{180 \times 8} = \frac{74.8 \times 10^3}{180 \times 8} \leq \frac{f_y}{\sqrt{3}\gamma_{m0}}$

$= 51.94 N/mm^2 \leq \frac{250}{\sqrt{3} \times 1.10}$

$= 51.94 N/mm^2 \leq 131.2 N/mm^2$ (Ok!)



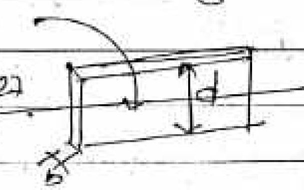
Page: 53. 8.2.1.2

designed bending stress $= \frac{M}{Z} \leq \frac{f_y}{\gamma_{m0}}$

$= \frac{8437.5}{\left(\frac{8 \times 180^2}{6}\right)} \leq \frac{250}{1.10}$ $Z = \frac{bd^2}{6} = \frac{8 \times 180^2}{6}$

$= 195.3 \leq 227.27$ MPa

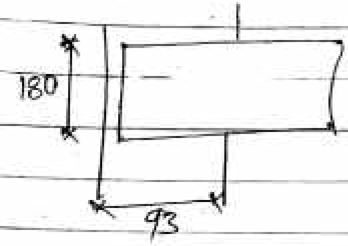
Ok!



(7)

Bolted connection:

adopt M20 bolt of pr.cl 4.6 with single line of bolt



no. of bolt $= \sqrt{\frac{6Pe}{mpV_{dsb}}}$

for M20 bolt, pr.cl -

$V_{dsb} = 45.27 \text{ kN}$

$m = 1$

$p = 60 \text{ mm} > 2.5 \times 20 = 50 \text{ mm}$, $e = 1.7 d_0$

$= 1.7 \times 22$

$= 33$

$\approx 40 \text{ mm}$

$\therefore n = \sqrt{\frac{6 \times 8437.5 \times 10^3}{1 \times 60 \times 45.27 \times 10^3}}$

$= 4.32$

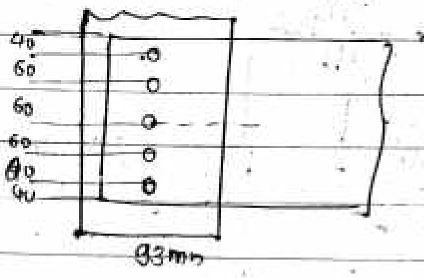
≈ 5

Since, 5 bolts can't accomodate on depth of 180 mm of batten so increase the size of batten as;

depth $= 60 \times 4 + 40 \times 2$

$= 240 + 80$

$= 320 \text{ mm}$



Then,

$\Sigma r_i^2 = 60^2 \times 2 + 120^2 \times 2 = 36000$

$r_{max} = 120 \text{ mm}$

$(F_{max}^{V}) = \frac{V_b}{N} = \frac{74.8}{5}$

$= 14.96 \text{ kN}$

Torsional force $= \frac{M \cdot r_{max}}{\Sigma r_i^2}$

$= \frac{8457.5 \times 10^3 \times 120}{36000}$

$= 28.125 \text{ kN}$

$\therefore R = \sqrt{14.96^2 + 28.125^2} \le$ Bolt value

$= 31.85 \text{ kN}$

$$\text{Bearing capacity } (V_{dpb}) = \frac{2.5 k_b d t (f_u/f_{ub})}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.606 \times 20 \times 8 \times 400}{1.25} = 77.568 \text{ kN}$$

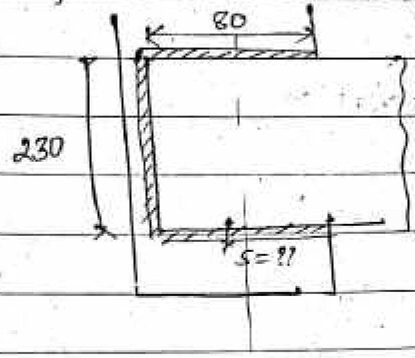
$$k_b \rightarrow \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, 1, \frac{400}{410}$$

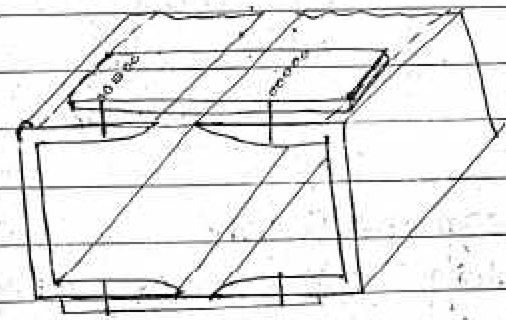
$$0.606, \quad , 1, 0.973$$

∴ Bolt value = 45.27 kN.

So, R = 31.85 kN ≤ 45.27 kN OK

for welded connection: (H.W)



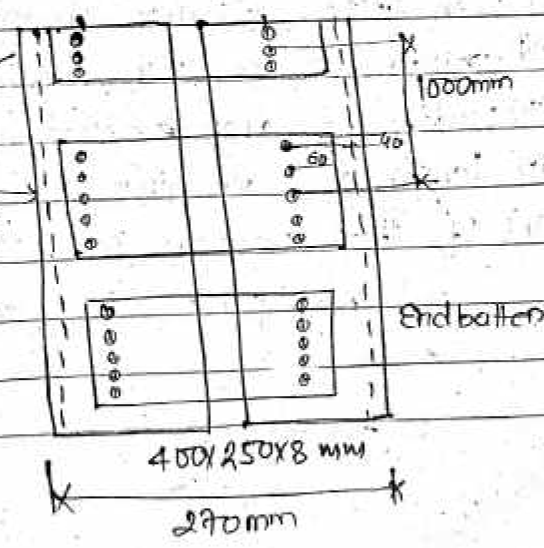


End batten

depth of end batten $= \frac{4}{3} \times 320$

$= 426.67$ mm

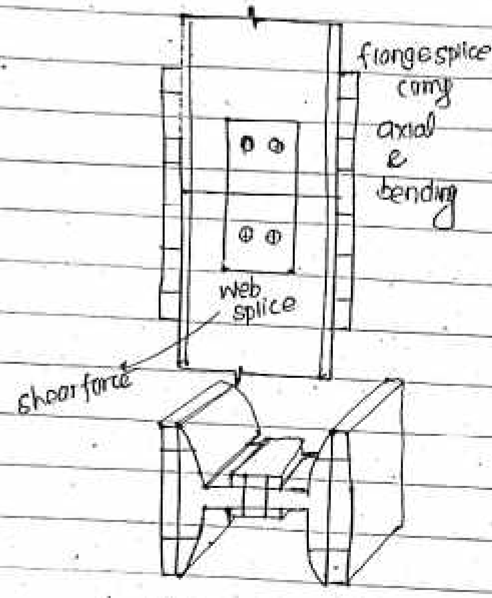
≈ 400 mm



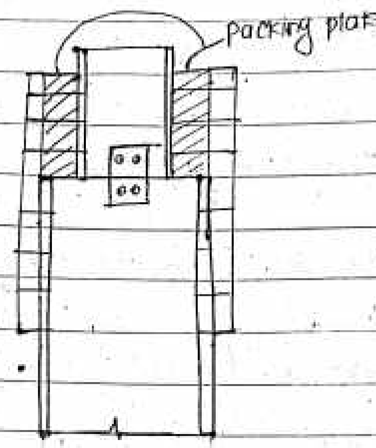
stiffner

Column splice :-

In case of tall building when two section of column are to be connected or in case of change in size column should be properly connected to facilitate the smooth transfer of load from upper to lower section. Such arrangement is column splice.



same size column.



Splicing column of different size.

Design specification:

(i) flange splice carries axial force and bending moment carried by column
(ii) web splice carries shear force carried by column.
(iii) splice acts as a short column of '0' slenderness ratio
(iv) width of flange splice should be equal to width of flange of column.
(v) length of splice is determined by no. of bolts required
(vi) If the section is milled (properly) only 50% of load is to be transferred by splice & its connection.

Design splice (column) to connect two ~~col~~ SC 220 carrying axial load of 500 kN (factored load) and Bending moment of 40 kN-m along with shearforce of 50 kN

Soln: Assuming properly milled section so only 50% of axial load is to be transferred by splice and its connection.

a) flang splice.

load on each flange splice $= (50\% \text{ of } 500) \times \frac{1}{2}$

$= \frac{250}{2}$

$= 125$ kN / splice.

section property for SC 220

$A = 89.8\ cm^2$

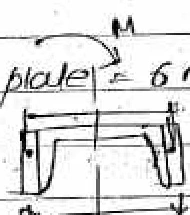
$D = 220$ mm

$B = 220$ mm

$t_w = 9.5$ mm

$t_f = 16$ mm

assume the thickness of flange plate = 6 mm



So, axial load on splice due to Bending moment $= \frac{M}{\text{lever arm}}$

$= \frac{40 \times 10^3}{(220 + \frac{6}{2} + \frac{6}{2})}$

$= \frac{40 \times 10^3}{0.226}$

$= \cancel{172.85}$ kN

$= 176.99$ kN

total load on each flange splice $= 125 + 177$

$= 302$ kN

for short column:-

$$fcd = \frac{fy}{\gamma_{mo}} = \frac{250}{1.10} = 227.27 \text{ Mpa}$$

Now, load carrying by flange splice = fcd x Area

$= 227.27 \times 220 \times t$

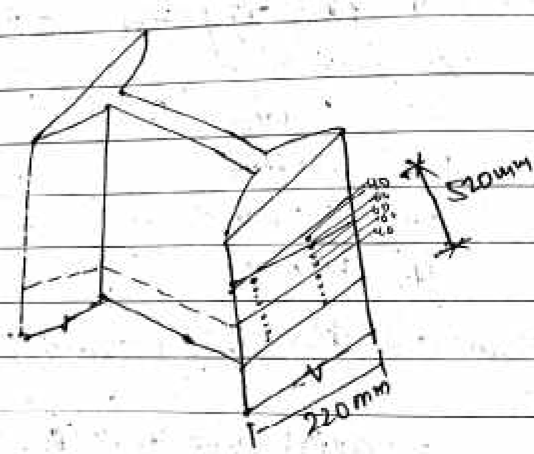
$= 499.99\, t \times 10^3$

so $220 \times t \times 227.27 \geq$ total load

$$t \geq \frac{301.99}{220 \times 227.27}$$

$t > 6.04$

take, $\therefore t \approx 8$ mm



adopt M20 bolt of pr.cl. 4.6

$V_{dsb} = 45.27$ kN

$p = 60$ mm

$e = 40$ mm

No. of bolts = $\frac{\text{total load}}{\text{Bolt value}}$

$= \frac{302}{45.27}$

$= 6.67$ nos

≈ 8 bolts.

total length of flange plate $= 2(2 \times 40 + 3 \times 60]$

$= 2[80 + 180]$

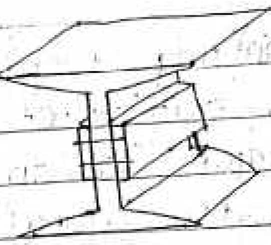
$= 520$ mm

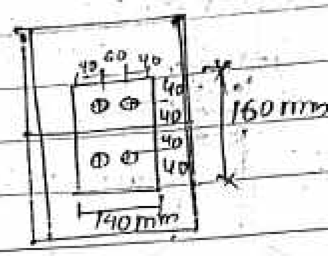
so, provide a plate of 520 x 220 x 8 mm
(flange splice)

Web splice :

Shear force = 50 KN

for M20 bolt of pr. cl. 4.6"





Shear capacity of bolt (V_{dsb}) = 2×45.27   if n = 2

= 90.54 kN

no of bolt = $\frac{50}{90.54}$ = 0.55

≈ 2 bolt/column

length of web splice = 4×40 = 160 mm

width of " " = 2×40 + 1×60

= 140 mm

for thickness

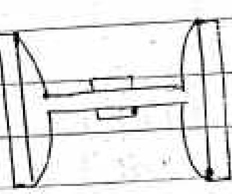
∴ $V = \frac{f_y A_v}{\sqrt{3} \times \gamma_{m0}}$

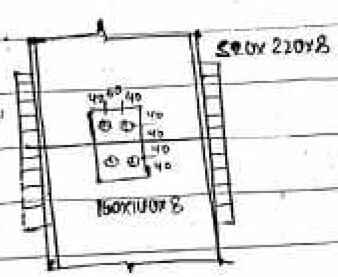
$50 \times 10^3 = \frac{250 \times 140 \times t}{\sqrt{3} \times 1.1}$

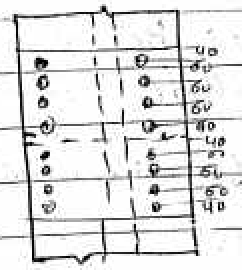
∴ t = 2.72 mm

≈ 6 mm

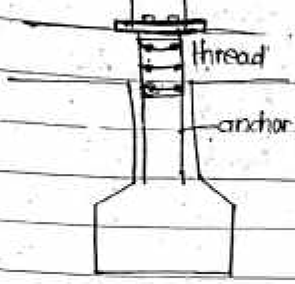
So adopt a web splice of 160×140×6 mm







Column base:



Stability of structure is determined by strength & stability of column base.
Major function of column base is to transfer load to footing (footing ~~pedestal~~) and distribute load in column such that it doesn't exceed bearing capacity of structure underneath.

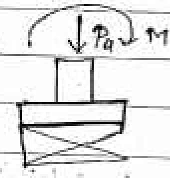
Depending upon load on column, there are two types of column bases:-
a) Slab base
b) Gussetted ~~plate~~ base

a) Slab base: if the load in column is moderate or small axial load along with very small bending moment, in such case slab base is suitable. In such case column base is entirely under compression.

compression = +ve

= -ve tension

$$\sigma = \frac{P}{A} \pm \frac{M}{Z}$$



Design steps:

1: Calculate bearing capacity of concrete.

Bearing capacity = $0.45 f_{ck}$ [LSM]

= $0.25 f_{ck}$ (WSM)

2. Calculate area required;

$$Area = \frac{P_u}{0.45 f_{ck}}$$

3. Calculate length & breadth of column base;

$$l = B = \sqrt{A}$$

4: Calculate pressure underneath column base

$$w = \frac{P}{Area} \pm \frac{M}{Z}$$

5. Calculate thickness of slab base as:

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

Page:- 7.4.3 (Slab base)

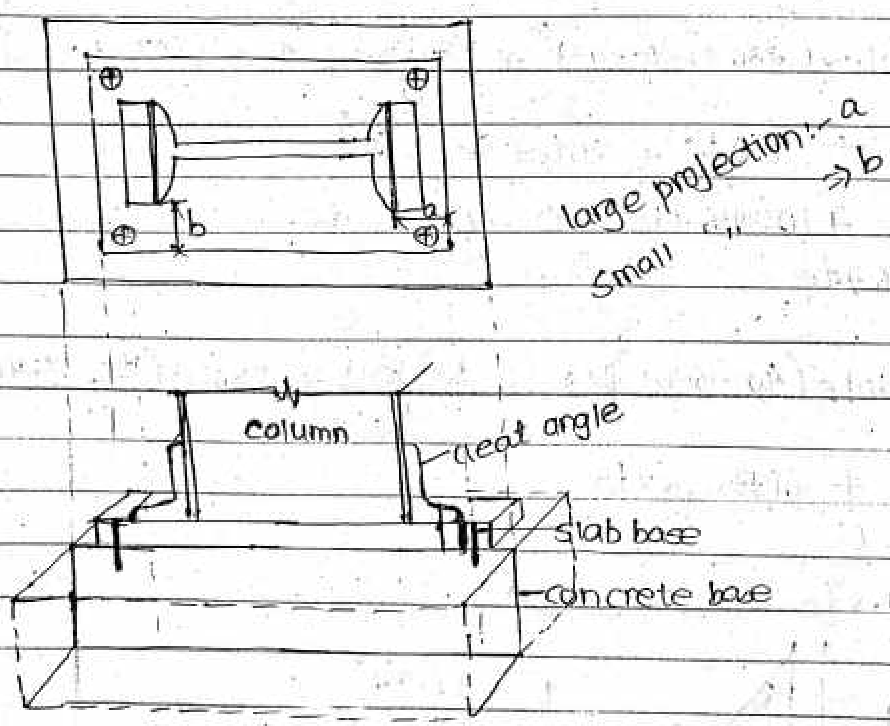
Page: 46

6: Connection design:- गर्नु पर्दैन

यदि moment आएर Tension developed भएमा बल्ट design गर्ने

a) Provide four M20 bolts (anchored) of min. anchored length 300mm

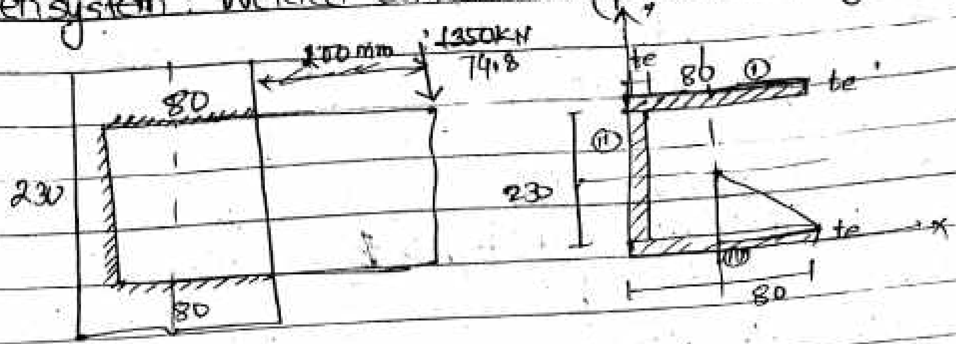
b) Provide cleat angle of 65×65×6 mm or 150×110×16 mm



Contd

Remaining part:

Batten system : welded connection: (parallel to flange) $t_e = 0.75$



Now,

	A_i	x_i	y_i
1	$80t_e$	40	$230+1.5t_e$
2	$230t_e$	$t_e/2$	$(115+t_e)$
3	$80t_e$	40	$t_e/2$
	$390t_e$		

$$\therefore \bar{x} = \frac{80\times 40t_e\times 2 + 230t_e\times\frac{t_e}{2}}{390t_e}$$

$$= \frac{6400t_e + 115t_e^2}{390t_e}$$

$$= 16.41 + 0.30t_e$$

$$\approx 16.41 \text{ mm}$$

$$\bar{y} = 115 + t_e \approx 115 \text{ mm}$$

Then,

$$I_{xx} = \frac{80t_e^3}{12} + 80t_e\times(230+1.5t_e-115)^2 + \frac{t_e\times 230^3}{12} + 230t_e\times(115+t_e-115)^2$$

$$+ \frac{80\times t_e^3}{12} + 80t_e\left(\frac{t_e}{2}-115\right)^2$$

$$= 1058000t_e + 1013916.6t_e + 0 + 1058000t_e$$

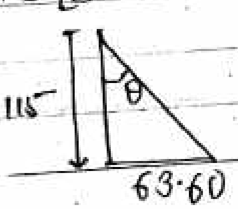
$$= 3129916.67t_e$$

$$I_{yy} = \left[\frac{80^3\times t_e}{12} + 80t_e(40-16.41)^2\right]\times 2 + \frac{t_e^3\times 230}{12} + 230t_e\left(\frac{t_e}{2}-16.41\right)^2$$

$$= 179371.42t_e + 61936.26\times t_e$$

$$= 236307.68\,t_e$$

$$\therefore I_p = 3366224.3\,t_e$$



$$\therefore \theta = 28.94^\circ$$

$$r_{max} = \sqrt{115^2 + 63.60^2}$$

$$= 131.41 \text{ mm}$$

Since, Axial load = ~~1350 kN~~ 74.8 kN

$\therefore e = \cancel{200 + 80 - 16.41}$

$= \cancel{263.60\ mm}$ 112.8 mm

$l_w = 230 + 80 \times 2$

$= 390\ mm$

Now,

vertical shear stress $(q) = \frac{P}{l_w \times t_e} = \frac{\cancel{1350}\ 74.8}{t_e \times 390} = \frac{\cancel{3.461}}{\cancel{t_e}}\ \frac{0.192}{t_e}$

Torsional " $q_T = \frac{P e r_{max}}{I_p}$

$= \frac{\cancel{1350}\ 74.8 \times \cancel{263.60}\ 8437.5 \times 131.41}{3366224.3\, t_e} = \frac{0.7697}{t_e}$

$= \frac{\cancel{13.89}}{\cancel{t_e}}\ \frac{0.3294}{t_e}$

$\sigma_x = q_T \cos\theta$

$= \frac{12.15}{t_e} = \frac{\cancel{0.6736}}{\cancel{t_e}}\ \frac{0.288}{t_e}$

$\sigma_y = \frac{3461}{t_e} + \frac{\cancel{6.72}\ 0.139}{t_e} = \frac{10.181}{t_e} = \frac{\cancel{0.5644}}{t_e} = \frac{0.927}{t_e}$

$\therefore$ Resultant $R = \sqrt{\sigma_x^2 + \sigma_y^2}$

$= \frac{15.851}{t_e} = \frac{0.438}{t_e}$ ~~fwd~~.

for safe design.

$R \leq f_{wd}$

$\frac{0.878 \times 10^3\ \cancel{15.851}}{t_e} \leq 189.37$

$\therefore t_e \leq \cancel{83.70\ mm}$ 4.64 mm

$t_e = 0.7S$

$\boxed{\therefore S = 6.629\ mm}$ = 7 mm

$\approx$ ~~8~~ mm

$\frac{0.438 \times 10^3}{t_e} \leq 189.37$

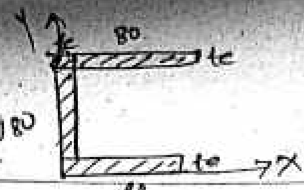
$\therefore t_e = 2.32$ mm

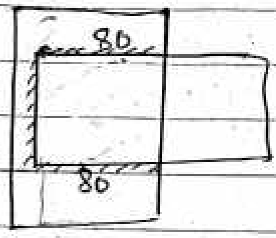
So, $t_e = 0.7S$

$\therefore S = 3.31$ mm

≈ 4 mm

Battlen system: Welding





	A_i	x_i	y_i
1	80 te	40	180+1.5te
2	180 te	te/2	90 + te
3	80te	40	te/2
	340te		

$$\text{So, } \bar{X} = \frac{80 \times 40 \times 2 + 180 \times te/2}{340}$$

$$= 18.82$$

$$\bar{Y} = 90$$

$$I_{xx} = \frac{2\times 80te^3}{12}^{\nearrow 0} + 80te \times (180 + 1.5te^{\nearrow 0} - 90)^2 + \frac{te \times 180^3}{12} + 180te(90 + te - 90)^2$$

$$+ 80te \times (0.5te - 90)^2$$

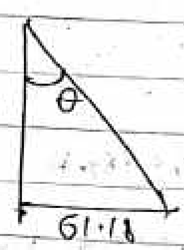
$$= 648000te \times 2 + 486000te + 0 + 0$$

$$= 1782000te$$

$$I_{yy} = 2\left[\frac{80^3 te}{12} + 80te(40 - 18.82)^2\right] + 180te(0.5te - 18.41)^2$$

$$= 218115.12$$

$$I_p = 2000115.125te$$



$$\theta = \tan^{-1}\left(\frac{61.18}{90}\right) = 34.21^\circ$$

$$=$$

$$r_{max} = 108.82$$

$P = 74.8$ kN

$M = 84375$ kN-m

$l_w = 340$mm

$$q = \frac{P}{l_w t_e} = \frac{0.22}{t_e}$$

$$q_T = \frac{M r_{max}}{I_p}$$

$$= \frac{8437.5 \times 108.82}{2000115.57}$$

$$= \frac{0.459}{t_e}$$

$$\therefore \sigma_x = q_T \cos\theta = \frac{0.379}{t_e}$$

$$\sigma_y = \frac{0.22}{t_e} + \frac{0.213}{t_e} = \frac{0.433}{t_e}$$

$$\therefore R = \sqrt{\sigma_x^2 + \sigma_y^2}$$

$$= \frac{0.575}{t_e}$$

Since

$$R \le f_{wd}$$

$$\frac{0.575 \times 10^3}{t_e} \le 189.37$$

$$\therefore t_e \ge 3.03 \text{ mm}$$

$$\approx 4 \text{ mm}$$

$$\text{So, } S = \frac{t_e}{0.7} = 5.71 \text{ mm}$$

$$\approx 6 \text{ mm}$$

Numerical:

Design slab base for SC 250 carrying an axial load of 500kN and grade of concrete used in pedestral is ~~M20~~ (M20)

soln:-

1: Bearing capacity of concrete $= 0.45 \times f_{ck}$

$$= 0.45 \times 20$$

$$= 9 \text{ N/mm}^2$$

2: Area required $= \dfrac{P}{0.45 f_{ck}} = \dfrac{500 \times 1.5}{0.45 \times 20}$

$$= 83333.33 \text{ mm}^2$$

3. Length & Breadth, $l = B = \sqrt{A} = \sqrt{83333.33}$

$$= 288.675 \text{ mm}$$

for SC 250

$D = 250\text{mm}$

$B = 250 \text{ mm}$

$t_f = 17 \text{ mm}$

4. Adopt a slab base of size; ~~350 x 350 mm~~ 400 x 400

4. Step 4: Pressure under column Base =

$$W = \frac{P}{A} \pm \frac{M}{Z}$$

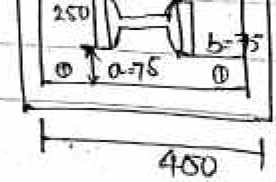
$$= \frac{500 \times 1.5 \times 10^3}{400 \times 400}$$

$$= 4.68 \text{ N/mm}^2 < 9 \text{ N/mm}^2 \quad \text{OK!}$$

5: Projection along x and y axis:

projection along x-axis := $b = 75$

" " y-axis $= a = 75$



so,

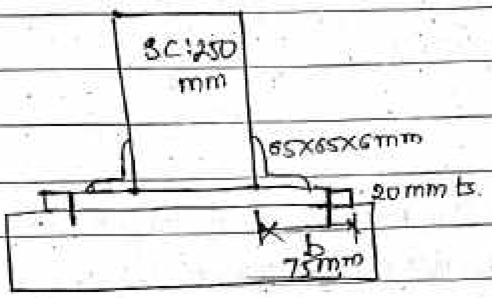
$$t_s = \sqrt{\frac{2.5 \times 4.68 (75^2 - 0.3 \times 75^2) \times 1.10}{250}}$$

$= 14.24 \text{ mm} \geq 17 \text{ mm}$

take,

$t_s = 20 \text{ mm}$

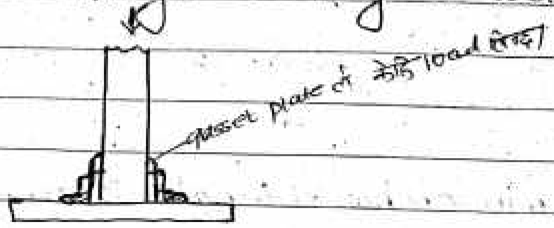
6: provide 4 anchor bolts of M20 of min anchored length 300mm
provide cleat angle of 65x65x6 mm



Column base

Gusseted plate:-

In case of large axial load or axial load with large B M depth of slab base is larger to overcome this limitation column are provided with Gusset plates so that load is transferred to the base through bearing as well as through Gusset plate



Design steps:

a) Bearing capacity of concrete $= 0.45 f_{ck}$

b) Calculate area of baseplate $= \dfrac{P_u}{0.45 f_{ck}}$

c) length of base plate is calculated such that there is min. projection beyond gusset angle. and width of Base plate is calculated as

$$B = \frac{A}{l}$$

d) Calculate the pressure underneath base plate.

$$\omega = \frac{P_u}{A} \pm \frac{M}{Z}$$

e) calculate thickness of base plate for Bending moment at most critical section

f) connection design:-

use gusset plate of 16 mm thickness

use gusset angle of 150x115x16mm (ISA)

(Vertical 2 bolts
H3. 1 bolts.)

Design gusseted base for a column ISHB 300 @ 58.5 kg/m carrying factored axial load of 1800 kN, use M25 Concrete for pedestal and M20 bolt of pr. cl 4.6

Solⁿ:-

for ISHB @ 58.5 kg/m

$D = 300$ mm

$B = 250$ "

$t_f = 10.6$ mm

$t_w = 7.6$ mm

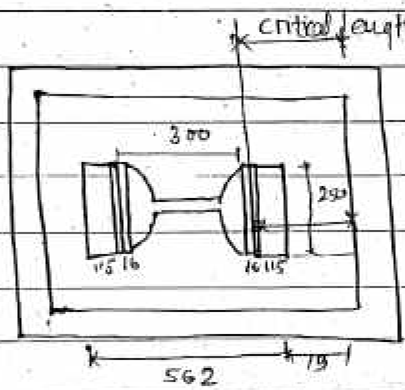
$A = 74.8\ mm^2$

adopt gusset plate of 16 mm thickness & gusset angle of 150 × 115 × 16 mm (ISA)

a) Bearing capacity of concrete $= 0.45 f_{ck}$

$= 0.45 \times 25$

$= 11.25\ N/mm^2$



b) Area required $(A) = \frac{P_u}{B.C} = \frac{1800 \times 10^3}{11.25}$

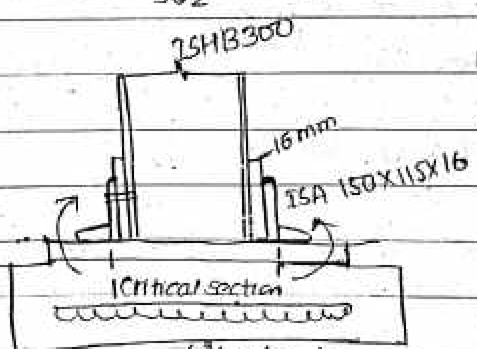
$= 160000\ mm^2$

$= 1600\ cm^2$

c) length of gusset plate $= 300 + 115 \times 2 + 16 \times 2$

$= 562$ mm

$\simeq 600$ mm



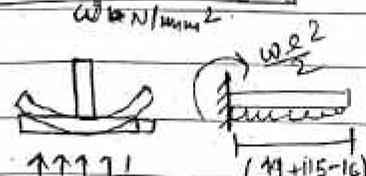
d) Width $(B) = \frac{A}{L} = \frac{160000}{600}$

$= 266.67$ mm

≈ 300

e) $\omega = \frac{P_u}{A} \pm \frac{M}{Z} = \frac{1800 \times 10^3}{300 \times 600}$

$= 10\ N/mm^2 < 11.25\ N/mm^2$



length of critical section = 118 mm

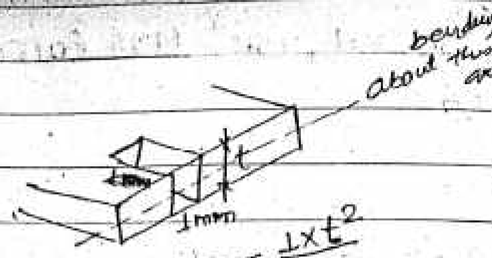
$C = 118mm$ (critical length)

B.M at critical section $= w \times 1 \times c \times \frac{c}{2}$

$= \frac{wc^2}{2}$

$= (10 \times 1) \times \frac{118^2}{2}$

$= 69620$ N.mm/mm ——(I)



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

Bending capacity $= \frac{Z_p f_y}{\gamma_{mo}}$

$= \frac{1 \times t^2}{6} \times \frac{250}{1.10}$

$=$ ——(II)

from (I) & (II)

$69620 = \frac{t^2 \times 250}{6 \times 1.10}$

$\therefore t = 42.87$ mm

$\approx 50mm$

f) 50% of load is transferred by bearing, only 50% load is to be transferred by connection for gusset plate.

$\therefore$ load for connection = 50% of factored load

$= .50 \times 1800$

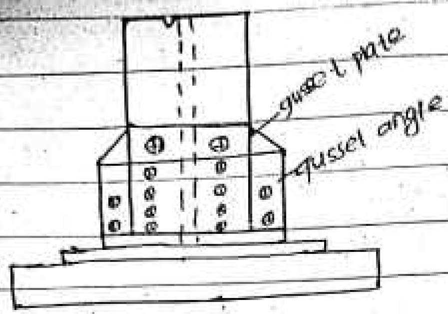
$= 900kN$

for M20 bolt of grd: 4.6

$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25} \times 1.787 \times \frac{20^2}{4} = 103.314$ kN

No of bolt $= \frac{900}{103.314} = 8.71$

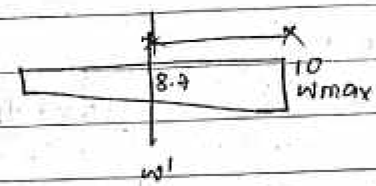
≈ 12



Eccentrically loaded column base:

$$\omega = \frac{P}{l \times B} \pm \frac{P \times e}{\frac{Bl^2}{6}}$$

$$= \frac{P}{lB}\left(1 \pm \frac{6e}{l}\right]$$



for tension in this section:-

$\omega = 0$

$$\omega = \left(\frac{w_{max} + \omega'}{2}\right) \times C \times \frac{L}{2}$$

$$\frac{P}{lB}\left(1 \pm \frac{6e}{l}\right) = 0$$

$$\therefore 1 = \frac{6e}{l}$$

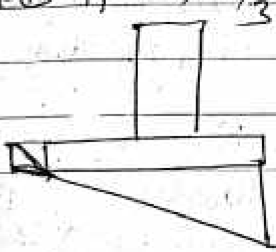
$$\boxed{\frac{l}{6} = e}$$

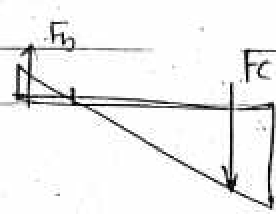
case① if $e \leq \frac{l}{6}$ ~~sect~~ column base is entirely under compression

$$\boxed{\left(\frac{P}{A} + \frac{M}{Z}\right) \leq 0.45 f_{ck}}$$

case② if $e \geq l/6$ and $e \leq \frac{l}{3}$ although column base is under tension it is neglisible hence designed similar to case①.

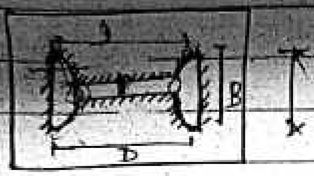
case③ if $e > l/3$, tension in column base is significant





no. anchor bolt $= \frac{\frac{1}{T_{du}}}{f_b} = \frac{f_b}{T_{du}}$

Welded connection



$l_w = 2 \times B + 2(B - t_w) + 2(D \ldots)$

Now, ISHB 300

$D = 300$ mm

$B = 250$ "

$t_f = 10.6$ "

$t_w = 7.6$ "

$A = 74.8\,cm^2$

factored load = 1800 kN

Now,

length of weld $= 2 \times 250 + 2(300 - 2 \times 10.6) + 2(250 - 9.4)$

$= 1539$ mm

load from column be transferred by weld connection

for, size of weld

let, $S = 8$ mm

and, $l_e = l_w - (\text{end turn}) \times 2S$

$= 1539 - 12 \times 2 \times 8$

$= 1347$ mm

throat thickness $(t_e) = 0.7S$

$= 0.7 \times 8$

$= 5.6$ mm

Strength of weld / mm $= \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times$ Area

$= \frac{410}{\sqrt{3} \times 1.25} \times 5.6 \times 1$

$= 1060.48$ N/mm (for shop weld)

$\therefore$ Required length of weld $= \frac{\text{axial load}}{\text{strength / mm}}$

$= \frac{1800 \times 10^3}{1060.48}$

$= 1697$ mm $>$ ~~1539~~ 1347 mm

not ok!

again increased the size of weld as $S = 12mm$

so, $l_e = l_w - 2 \times 2S$
$= 1539 - 2 \times 2 \times 12$
$= 1251 \, mm$

throat thickness $= t_e = 0.7S$
$= 0.7 \times 12$
$= 8.4 \, mm$

strength of weld/mm $= \frac{8.4 \times 1 \times 410}{\sqrt{3} \times 1.25}$
$= 1590 \, N/mm$ (shop weld)

Required length of weld $= \frac{\text{Axial factload}}{\text{Strength/mm}}$
$= \frac{1800 \times 10^3}{1590}$
$= 1132 \, mm < 1251 \, mm$ ok.

Hence, provide the size of ~~weld~~ weld $= 12 \, mm$
Required length of weld $= 1132 \, mm$

CH: 5 Tension Members:

→ Members ~~are~~ the action of axial tension or member under tension due to BM are tension members.

failure in tension member

6.2.

① Yielding of section

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

2) Rupture on tearing of section:-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}},$$

where, $A_n = \left[\left\{(b - n d_o) + \frac{\Sigma p_i^2}{4g}\right\} t\right]$

3). Block shear failure: [Cl. 6.4]

$$T_{db} = \left[A_{vg} f_y / (\sqrt{3} \gamma_{mo}) + 0.9 A_{n} f_u / \gamma_{m1}\right]$$

or

$$= \left[0.9 A_{vn} f_u / (\sqrt{3} \gamma_{m1}) + A_g f_y / \gamma_{mo}\right]$$

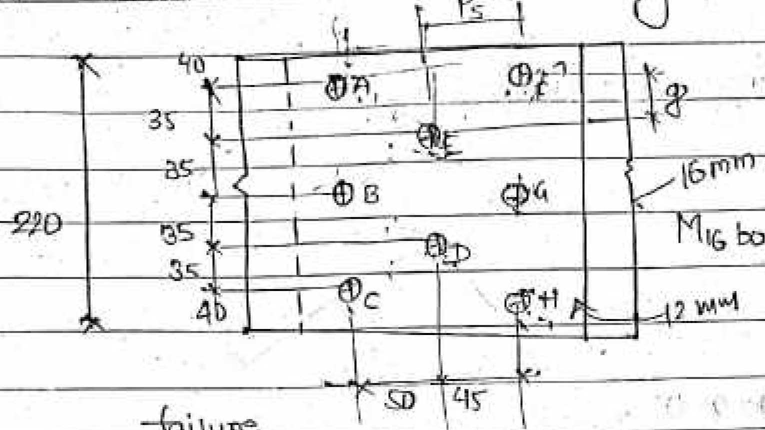
Net area:

$$A_n = \left[\{b - n d_o\} + \frac{\Sigma p_i^2}{4g}\right] \times t$$

also,

$\cancel{A_n} = T_{dn} = \alpha A_n f_y / \gamma_{m1}$

Calculate Net area for following connection:



H3 failure हुन्छ।

M_{16} bolts:

failure

① Along A-B-C

$$A_n = \left[(B - nd_0) + \frac{\Sigma p_1^2}{4g}\right] \times t$$

$$= [(220 - 3\times 18)] \times 12$$

$$= 1992 \text{ mm}^2$$

(ii) along A-E-B-C

$$A_n = \left[(B - nd_0) + 2\frac{\Sigma p_1^2}{4g}\right] t$$

$$= \left[(220 - 4\times 18) + 2\frac{50^2}{4\times 35}\right] \times 12$$

$$= \cancel{1990.28}\ 2204.57$$

2 - due to two inclination

(iii) along A-F-B-D-C

$$A_n = \left[(220 - 5\times 18) + 4\times\frac{50^2}{4\times 35}\right] \times 12$$

$$= 2417.14$$

(iv) along A-E-D.

$A_n = \cancel{\ \ }\ 2206.28$

(v) along AEDC

$A_n = 2204.57$

(vi) along A-F-G-H, A-B-C

$A_n = 1992$

(vii) along AEGH $A_n = 2163.85$

(viii) along FEGDH

$A_n = 2254.28 \text{ mm}^2$

(ix) along F-E-D

$A_n = 2165.57 \text{ mm}^2$

along F-E-G-H

$A_n = \cancel{1940.57}\ 2123.14 \text{ mm}^2$

for failure

no of bolt होरे

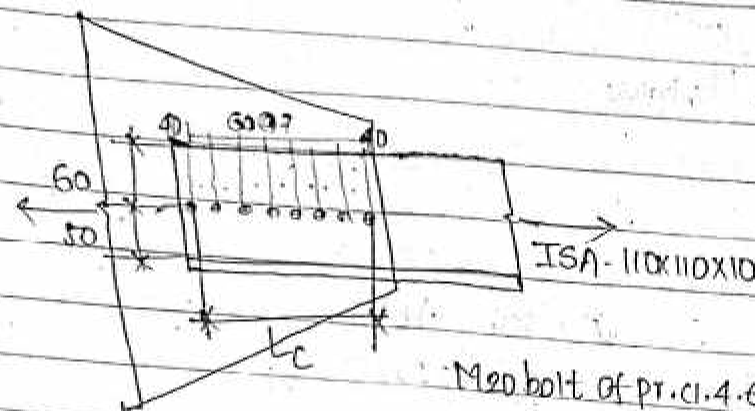
p कम भएको inclination पिर को

arrangement लिने

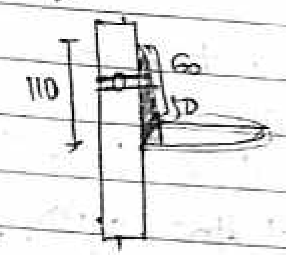
[Tension word
Tensile capacity]

So, $A_n = 1992\ mm^2$

– Calculate Tensile capacity of the following connection:



M20 bolt of pr. cl. 4.6



Now, design shear strength $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{m1}}(A_n n_n + A_s n_s)$

$= \frac{400}{\sqrt{3} \times 1.25} \, 0.7871 \times \frac{20^2}{4}$

$= 45.272\ kN$

Total design shear strength $= 45.272 \times 8$

$= 362.18\ kN$ —— (I)

$p = 60$

$e = 40$

$\therefore \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25$

$0.60,\ 0.659$

Bearing capacity $= \frac{2.5\,k_b\,d\,t\,(f_u/f_{ub})}{1.25}$

$(V_{dpb}) = \frac{2.5 \times 0.60 \times 20 \times 10 \times 400}{1.25}$

$= 96.96\ kN$

$\therefore$ Total $V_{dpb} = 96.96 \times 8$

$= \cancel{768\ kN}\ 775.68\ kN$ —— (II)

(iii) Yielding capacity of plate / angle $(T_{dg}) = \dfrac{A_g f_y}{\gamma_{m0}}$

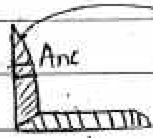
$$= \frac{2110 \times 250}{1.10}$$

$$= 479.54 \text{ kN}$$

(iv) Rupture on tearing of section ;

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta A_{go} f_y / \gamma_{m0}$$

$\boxed{\cancel{A_{nc} = (500 - 8 \times 22) \times 10 = 3240 \text{ mm}^2}}$ connected leg net area



$$A_{nc} = [(110 - 10) - 22] \times 10 + \tfrac{1}{2}\, 10 \times 10$$

$$= 830 \text{ mm}^2$$

$$A_{go} = 100 \times 10 + 50 = (110 - 10) \times 10 + \tfrac{1}{2} \times 10 \times 10$$

$$= 1050 \text{ mm}^2$$

so, $\beta = 1.4 - 0.076 (w/t) \frac{f_y}{f_u} (b_s / L_c) \le f_u \gamma_{m0} / f_y \gamma_{m1}$
≥ 0.7

$$= 1.4 - 0.076 (110/10) (150/420) \tfrac{250}{410} \le \frac{410 \times 1.10}{250 \times 1.25}$$
≥ 0.7

$$= 1.218 \le 1.44$$

$b_s = w + w_1 - t \ge 0.70$

$= 110 + 50 - 10$

$= 150$

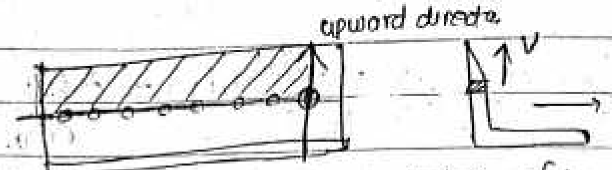
$L_c = 60 \times 7 = 420$

$\therefore \beta = 1.218$

so, $T_{dn} = \dfrac{0.9 \times 830 \times 410}{1.25} + \dfrac{1.218 \times 1050 \times 250}{1.10}$

$= 535.6 \text{ kN}$

(v). Block shear failure:



shear dr^n
$A_{vg} = [40+7\times60]\times10 = 4600\ mm^2$ 7.5 bolt मात्र पर्छ।

$A_{vn} = [(40+7\times60) - 7.5\times22]\times10 = 2950\ mm^2$

tension dr^n
$A_{tg} = 60\times10 = 600\ mm^2$

$A_{tn} = (60-0.5\times22)\times10 = 490\ mm^2$

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = 748.24\ kN$$

$$= \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}} = 639.143\ kN$$

so, $T_{db} = 639.143\ kN$

so, Tensile capacity of angle = 362.16 kN

Design unequal angle to carry a factored axial load of 250kN (factored tensile load)

soln:- use M20 bolt of prcl 4.6

from yielding criteria:

$$T_{dg} = \frac{f_y A_g}{\gamma_{m0}}$$

$$250\times10^3 = \frac{250\times A_g}{1.10}$$

$\therefore A_g = 1100\ mm^2$
$= 11\ cm^2$

also,

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$$

$$250\times10^3 = \frac{0.8\times A_n 400}{1.25}$$

$\therefore A_n = 9.52\ cm^2$

Selection of unequal angle having area $> 11 cm^2$

take ISA . $90 \times 60 \times 10$ $A = 14 cm^2$

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \left(1 \times 0.78 \times \pi \times \frac{20^2}{4}\right)$$

$$= 45.27 \text{ KN}$$

$$\therefore \text{no of bolts} = \frac{\text{load}}{V_{dsb}}$$

$$= \frac{250}{45} = 5.5$$

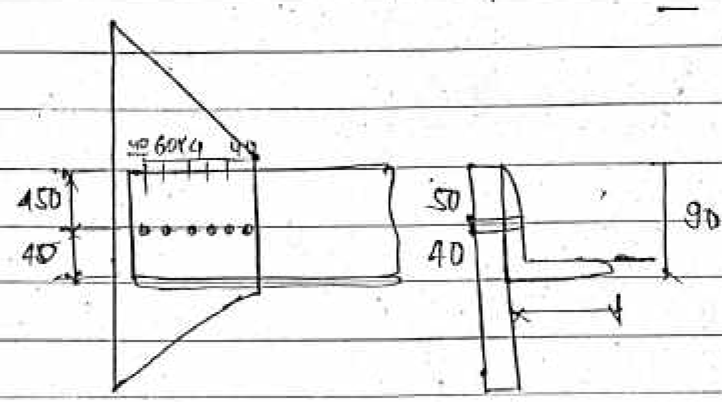
take $n = 6$

pitch $p \geq 2.5d \approx 50$

≈ 60 mm

$e \geq 1.5 d_0 = 1.5 \times 22 > 33$ mm

≈ 40 mm



$$V_{dpb} = \frac{2.5 k_b \, d t (f_u/f_{ub})}{1.25}$$

$$= \frac{2.5 \times 0.606 \times 20 \times 10 \times 400}{1.25}$$

$$= 96.96 \text{ kN}$$

Total $= 96.96 \times 6$

$= 581.76 > 250$ kN OK!

Tdn & block check gara:

$$A_{go} = (90-10)\times10 + \frac{10\times10}{2}$$
$$= 805\ mm^2$$

$w = 90$

$b_s = w + w_1 - t$

$= 90+40-10$

$= 120\ mm$

$L_c = 5\times60 = 300\ mm$

$$\beta = 1.4 - 0.076\times(w/t)\times(f_y/f_u)\times\left(\frac{b_s}{L_c}\right)$$
$$= 1.233 \le 1.44$$

$$T_{dn} = \frac{0.9\times585\times410}{1.25} + \frac{1.233\times805\times250}{1.10}$$
$$= 398.27 > 250$$

$A_{vg} = (40+5\times60)\times10$

$= 3400$

$A_{vn} = 2410\ mm^2$

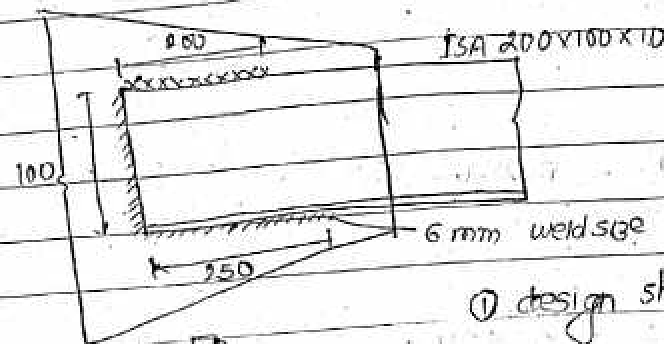
$A_{tg} = 50\times10 = 500\ mm^2$

$A_{tn} = (50-0.5\times22)\times10 = 390\ mm^2$

$$T_{db} = \frac{3400\times250}{\sqrt{3}\times1.10} + \frac{0.9\times390\times410}{1.25}$$
$$= 561.26 > 250\ kN$$

end turn

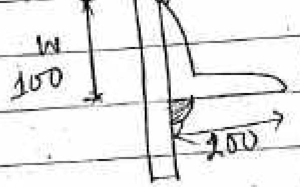
Calculate tensile load carrying capacity of following angle section.



① design shear capacity of weld $(f_{wd}) = \frac{f_u}{\sqrt{3} \times 1.25}$

$= \frac{410}{\sqrt{3} \times 1.25}$

$= 189.37$ KN



$L_c = \frac{250+200}{2} = 225$ mm

$b_s = w = 100$

total load carried by weld $= f_{wd} \times t_e \times l_e$

$= 189.37 \times 0.7 t \times 550$

$= 189.37 \times 0.7 \times 6 \times 550$

$= 437.4$ kN

$l_w = 200 + 100 + 250$

$= 550$

for ISA 100×100×10

$A = 19 cm^2$

(ii) Yielding capacity of angle $= \frac{f_y A_g}{1.10}$

$= \frac{250 \times 19 \times 10^2}{1.10}$

$= 431.8$ kN

$A_{nc} = (100 - \frac{10}{2}) \times 10$

$= 950 mm^2 = A_{go}$

$\beta = 1.4 - 0.076 \times \frac{100}{10} \times \frac{250}{410} \times \frac{100}{225}$

$= 1.19 \geq 0.7$

$\leq \frac{410 \times 1.1}{250 \times 1.25} = 1.44$

(iii) Rupture of Tearing:

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta A_{go} f_y / \gamma_{m0}$

$= \frac{0.9 \times 950 \times 410}{1.25} + \frac{1.19 \times 950 \times 250}{1.10}$

$= 537.37$ KN

(iii) Block shear failure:-

$A_{vg} = A_{vn} = (200+250) \times 10 = 4500 \, mm^2$

$A_{tn} = A_{tg} = 100 \times 10 = 1000 \, mm^2$

$$T_{db} = \frac{4500 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 1000 \times 410}{1.25} = 885.67 \, KN$$

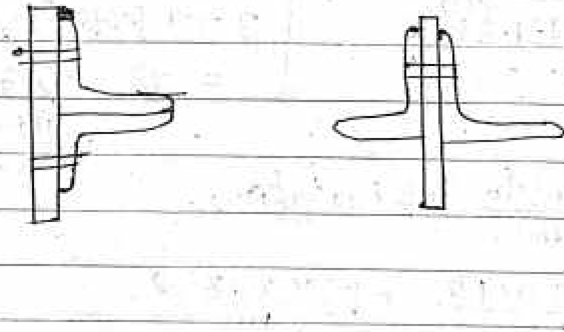
$$\text{or, } \frac{0.9 \times 4500 \times 410}{\sqrt{3} \times 1.25} + \frac{1000 \times 250}{1.10} = 994.224 \, KN$$

So, Tensile load carrying capacity - 431.8 KN

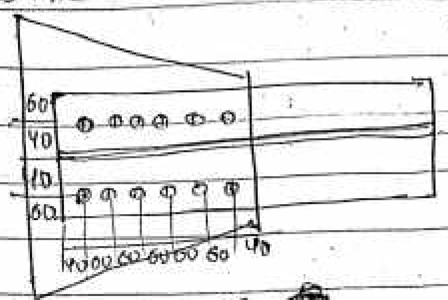
Calculate load carrying capacity of 2 ISA 100X100X10 connected to a gusset plate 16 mm in
(i) same side of gusset plate
(ii) opposite " " " "

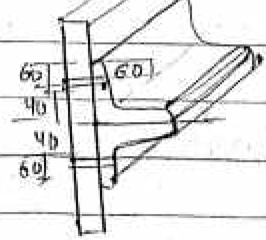
सबैको value single लाई निकालेर 2ले multiply गर्ने ।

Same side.



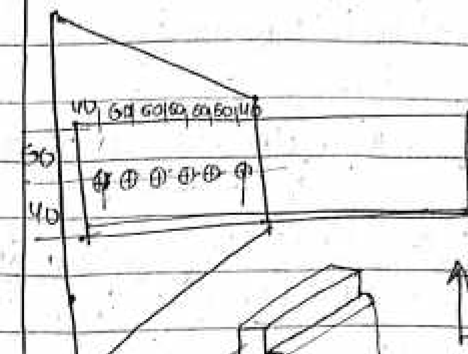
(i) Same side:

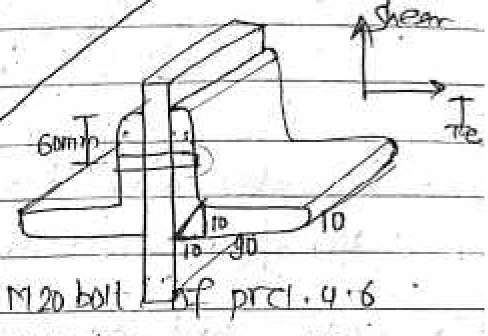




H.W JK

opposite side:





assume M20 bolt of prcl 4.6

plate thickness = 16 mm

angle:

2 ISA 100 × 150 × 10

Yielding capacity of angle (T_{dg})

$= \frac{A_g f_y}{1.10}$

$= \frac{1900 \times 250}{1.10}$

$= 431.82$ KN

Total yielding $= 2 \times 431.82 = 863.636$ KN —— (ii)

Rupture strength $T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} \times f_y}{\gamma_{m0}}$

$= \frac{0.9 \times 410 A_{nc}}{1.25} + \frac{\beta \times A_{go} \times 250}{1.10}$ (ii)

$A_{nc} = [(100-10) - 22] \times 10 + \frac{10 \times 10}{2} = 730 \text{ mm}^2$

$A_{go} = (100+10) \times 10 + 50 = 950 \text{ mm}^2$

$L_c = 60 \times 5 = 300$, $b_s = w + w_1 - t = 130$

$\beta = 1.4 - 0.076 \times \left(\frac{100}{10}\right) \times \frac{250}{400} \times \frac{130}{300}$

$= 1.2 > 0.7$

≤ 1.44

$\therefore T_{dn} = 474.586$ KN

Total $T_{dn} = 2 \times 474.586$

$= \underline{949}.172$ KN —— (iv)

(i) design shear strength of bolt

$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25} \left(1.78 \frac{\pi d^2}{4}\right)$

$= \frac{400}{\sqrt{3} \times 1.25} \times 1.78 \times \pi \times \frac{20^2}{4}$

$= 103.31$ KN

Total $V_{dsb} = 103.31 \times 6 = 619.884$ —— (I)

$V_{dpb} = \frac{2.5 k_b d t (f_u/f_{ub})}{1.25}$

$= \frac{2.5 \times 0.606 \times 20 \times 16 \times 400}{1.25}$

$= 155.136$ KN

Total $V_{dpb} = 6 \times 155.136$

$= 930.816$ KN

—— (II)

Lug angle if the length of connection is restricted an extra angle section is provide

Design strength due to block shear:

$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right] \text{ or } \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

$A_{vg} = (40 + 60\times5)\times10 = 3400\ mm^2$

$A_{vn} = [(40 + 60\times5) - 5.5\times22]\times10 = \cancel{3504\ mm^2}\ 2190\ mm^2$

$A_{tg} = 60\times10 = 600\ mm^2$

$A_{tn} = (60 - 0.5\times22)\times10 = 490\ mm^2$

$$\text{So, } T_{db} = \left(\frac{3400\times250}{\sqrt{3}\times1.10} + \frac{0.9\times490\times410}{1.25}\right) \text{ or } \left[\frac{0.9\times2190\times410}{\sqrt{3}\times1.25} + \frac{600\times250}{1.10}\right]$$

$= 590.78$ or $\underline{509.613}$ kN

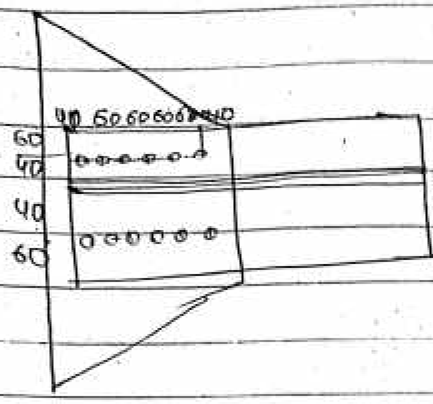
So, Total $T_{db} = 2\times509.613$

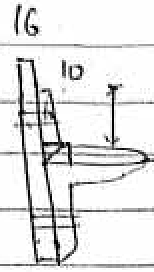
$= 1019.227$ kN —— (v)

Hence, load carrying capacity = 619.884 kN

Soln:-

On same side:





Given M20 bolt

$d = 20\,mm$

$d_0 = 22\,mm$

$t_p = 16\,mm$

ISA = 100 X 100 X 10 mm

(i) $V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25}\left(0.78\pi \frac{d^2}{4}\right)$

$= \frac{400 \times 0.78 \times \pi \times 20^2}{\sqrt{3} \times 1.25 \times 4}$

$= 45.272\ kN$

Total $V_{dsb} = 45.272 \times 6 \times 2 = 271.632\ kN \times 2$

$= 543.264\ kN$

(ii) Bearing capacity $= \frac{2.5 k_b d t (f_u / f_{ub})}{1.25}$

$= \frac{2.5 \times 0.606 \times 20 \times 10 \times 400}{1.25} = 96.96\ kN$

Total $V_{dpb} = 96.96 \times 6 \times 2 = 581.76\ kN \times 2 = 1163.52\ kN$

(iii) Yielding strength of angle $= T_{dg} = \frac{A_g f_y}{1.10} = \frac{1900 \times 250}{1.10} = 431.818\ kN$

Total $T_{dg} = 2 \times 431.818 = 863.636\ kN$

(iv) Design strength due to rupture.

$T_{dn} = \frac{0.9 A_{nc} f_u}{1.25} + \frac{\beta A_{go} f_y}{1.10}$

$A_{nc} = (100 - 10 - 22) \times 10 + \frac{1}{2} \times 10 \times 10$

$= 730\ mm^2$

$\beta = 1.4 - 0.076 \times \frac{100}{10} \times \frac{250}{410} \times \frac{130}{300}$

$= 1.19 \geq 0.7$

≤ 0.94

$A_{go} = (100 - 10) \times 10 + 50 = 950\ mm^2$

$W = 100$

$W_t = 40$ $\quad b_s = 140 - 10 = 130$

$t = 10$ $\quad L_c = 60 \times 5 = 300$

$\therefore T_{dn} = 472.427$

$\therefore$ Total $T_{dn} = 944.855\ kN$

Block shear:

$A_{vg} = (40 + 60 \times 5) \times 10 = 3400\ mm^2$

$A_{vn} = (40 + 60 \times 5 - 4.5 \times 22) \times 10 = 2190\ mm^2$

$A_{tg} = 60 \times 10 = 600\ mm^2$

$A_{tn} = (60 - 0.5 \times 22) \times 10 = 490\ mm^2$

$\therefore T_{db} = \left(\frac{A_{vg} f_y}{\sqrt{3} \times 1.10} + \frac{0.9 A_{tn} f_u}{1.25} \right)$ or $\left[\frac{0.9 A_{vn} f_u}{\sqrt{3} \times 1.25} + \frac{A_{tg} f_y}{1.10} \right]$

$= \left(\frac{3400 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 490 \times 410}{1.25} \right)$ or $\left(\frac{0.9 \times 2190 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.10} \right)$

$= 590.782\ kN$ or 509.613

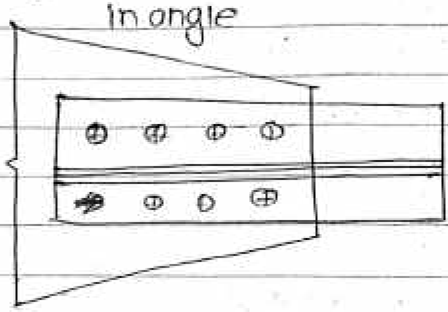
So, Total $T_{db} = \underline{1019.227\ kN}$

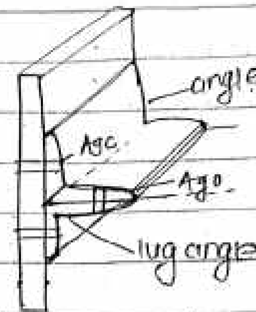
Hence, Tensile capacity $= 271.632\ kN \times 2 = 543.64\ kN$

Lug angle: 10.12 (83/84)

if the length of connection is restricted an extra angle section is provided to support main tension member which share (load) load carried by tension member hence reducing no of bolts & length of connection.

in angle

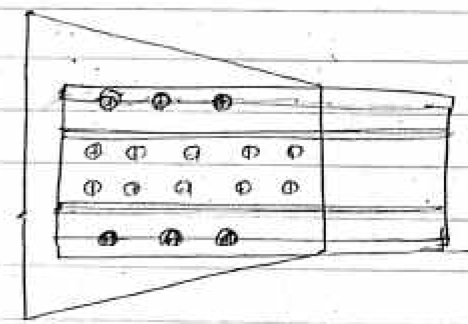


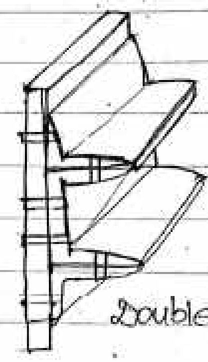


load outer leg:
by

$$\left(\frac{100\,A_{go}}{A_{go}+A_{go}}\right)$$

in channel section

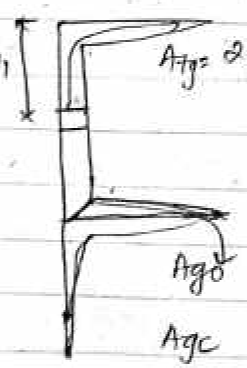




Double lug angle

MC 400

$A_{vg} = A_{vn}$



$A_{tg} = 200 \times t_w + 100 \times t_f - t_f - t_w$

lug angle

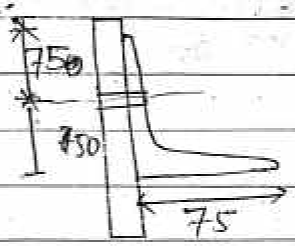
w.r.t outstanding leg load $= \frac{A_{go}}{(2A_{go}+A_{gc})} \times$ load

Design criteria 10.12.2

load carried by lug angle & its connection to gusset plate is 20% in excess of load carried by outstanding leg whereas load carried by connection betn lug angle & main member is 40% in excess of load carried by outstanding leg in case of single lug angle.

Design a single (unequal) angle to carry a factored tensile load of 400 KN using M16 bolts of pr cl 4.6. Design with & without lug angle:

a) without lug angle:-



$W = 75$

$W_t = 50 \quad t = 10 \quad \therefore b_s = 75 + 50 - 10$

$= 115$

① Area required $= \dfrac{T \times \gamma_{m0}}{f_y} = \dfrac{400 \times 10^3 \times 1.10}{250}$

$= 1760 \ mm^2$

$= 17.60 \ cm^2$

Select ISA $125 \times 75 \times 10$ mm having $A = 19.0 \ cm^2$

connection of design:-

(i) $V_{dsb} = \dfrac{f_{ub}}{\sqrt{3}} \times 0.78 \dfrac{\pi \times 16^2}{4}$

$= 28.974 \ KN$

(ii) $V_{dpb} = \dfrac{2.5 \times 0.556 \times 16 \times 10 \times 400}{1.25}$

$= 71.168 \ KN$

$\therefore$ no of bolts $= \dfrac{400}{28.974} = 13.8 \approx 14$ nos

Total bearing capacity

$= 14 \times 71.168$

$= 996.352 \ KN > 400 \ KN$

pitch $p \geq 2.5 d_0 = 2.5 \times 18 = 40$

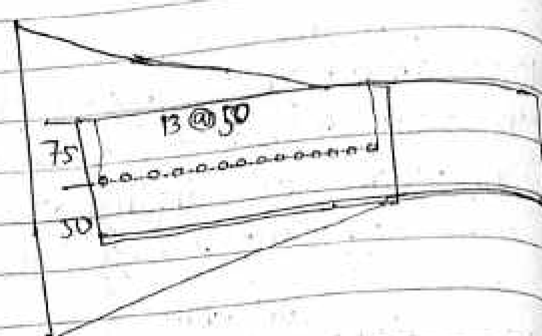
≈ 50

edge $e \geq 1.5 d_0 = 27 \approx 30 mm$

$k_b = 0.556$

Tensile capacity of ange:

connected longer leg to gusset:

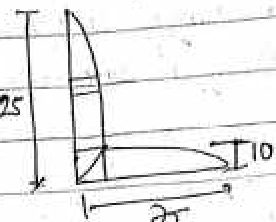


$b_s = 115$

$t = 10$

$W = 75$

$L_c = 13 \times 50 = 650 \text{ mm}$



$\beta = 1.4 - 0.076 \times \left(\frac{75}{10}\right) \times \frac{250}{410} \times \frac{115}{650}$

$= 1.34 \quad > 0.7$

$A_{nc} = (115 - 18) \times 10 = 970 \text{ mm}^2 + 50$

$+ 50 \quad = 1020 \text{ mm}^2$

$A_{go} = (75 - 10) \times 10 + \frac{10 \times 10}{2} = 700 \text{ mm}^2$

$\therefore T_{dn} = \frac{0.9 \times 1020 \times 410}{1.25} + \frac{1.34 \times 700 \times 250}{1.10}$

$= 514.285 \text{ kN} \quad > 400 \text{ kN}$

For Block shear strength

$A_{vg} = (30 + 50 \times 13) \times 10 = 6800 \text{ mm}^2$

$A_{vn} = (30 + 50 \times 13 - 13.5 \times 18) \times 10 = 4370 \text{ mm}^2$

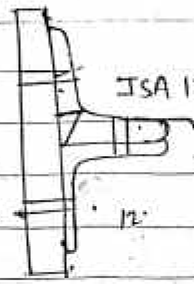
$A_{tg} = 75 \times 10 = 750 \text{ mm}^2$

$A_{tn} = (75 - 0.5 \times 18) \times 10 = 660 \text{ mm}^2$

$T_{db} = \left(\frac{6800 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 660 \times 410}{1.25}\right)$ or $[170.43 \times A_{vn} + 227.27 A_{tg}]$

$= 1087.20643 \text{ kN}$ or. $915.23 \text{ kN} > 400 \text{ kN}$

With Lug angle



$A_{go} = (75 - \frac{10}{2}) \times 10$
$= 700\ mm^2$

$A_{gc} = (125 - \frac{10}{2}) \times 10$
$= 1200\ mm^2$

load carried by connected leg $= \frac{1200}{(1200+700)} \times 400\ kN$
$= 252.6\ kN$

load carried by outstanding leg $= \frac{700}{(1200+700)} \times 400\ kN$
$= 147.368\ kN$

Now, for M_{16} bolt of prcl 4.6
$V_{dsb} = 28.97\ kN$

so no of bolts by connected leg $= \frac{252.6}{28.97}$
$= 8.71 \approx 9$ mm

Design strength of lug angle:- 1.2×147.368
$= 176.8\ kN$

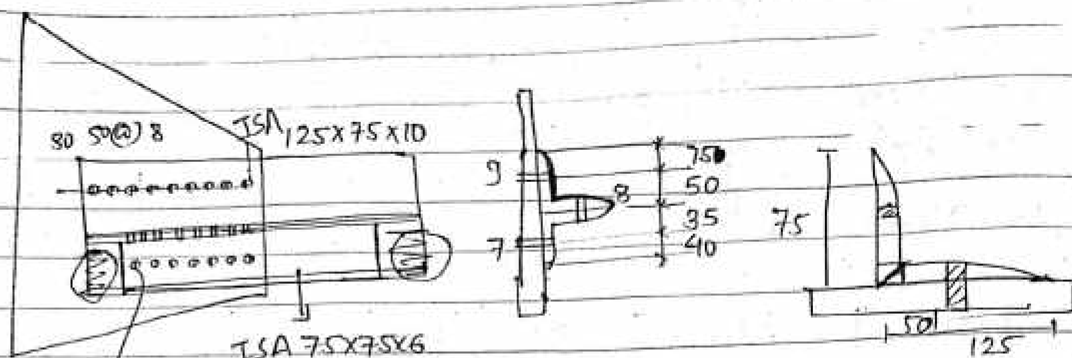
area required for lug angle $= \frac{176.8 \times 10^3 \times 1.1}{250}$
$= 7.78\ cm^2$

Select, lug angle: 75X75X6 mm of area $8.6\ cm^2$

no. of bolt, required for ~~connect~~ lug angle to gusset plate
$= \frac{176.8}{28.97} = 6.10 \approx 7$ nos.

load, to be carried by connection betn lug angle and main angle section $= 1.4 \times 147.368$
$= 206.36\ kN$

$\therefore$ no of bolts $= \frac{206.36}{28.97} = 7.14 \approx \underline{8}$ nos.



M16 bolt of pr cl: 4.6

$W = 75 \quad w_1 = 50 \quad b_s = 115$

$\beta = 1.40 - 0.076 \times \frac{75}{10} \times \frac{250}{410} \times \frac{115}{410}$

$= 1.30 \ (> 0.7) \ (< 1.44)$

so $\beta = 1.30$

Tdn & Tdb for main angle :-

For Tdn: $T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$

$= \frac{0.9 \times 1020 \times 410}{1.25} + \frac{1.30 \times 520 \times 250}{1.10}$

$= 454.74 > 400\text{ kN}$ OK!

$A_{nc} = (125 - 10 - 18) \times 10 = 970 + \frac{1}{2} \times 10 \times 10 = 1020$

$A_{go} = (75 - 10 - 18) \times 10 + 50 = 520$

For Tdb: $A_{vg} = (30 + 50 \times 8) \times 10 = 4300 \text{ mm}^2$

$A_{vn} = (30 + 50 \times 8 - 7.5 \times 18) \times 10 = 2950 \text{ mm}^2$

$A_{tg} = 75 \times 10 = 750 \text{ mm}^2$

$A_{tn} = (75 - 0.5 \times 18) \times 10 = 660$

so,

$T_{db} = (131.21 A_{vg} + 295.2 A_{tn})$ or $(170.45 A_{vn} + 227.27 A_{tg})$

$= 759.035$ or 673.221

$\therefore T_{db} = 673.221 > 400\text{ kN}$ (OK!)

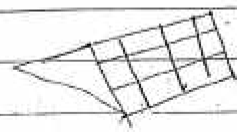
CH:7 Flexure member (14-16)

→ Member under Bending moment aswell as shear force as a major action

→ it is generally a horizontal member

→ Types of flexure Member

1. Beam
2. Stringer
3. lintel
4. Purlin
5. Rafter etc.



Section used in flexure member

(i) Rolled - I-section

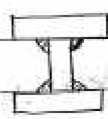
(ii) wide flange beam

(iii) Angle section

(iv) channel "

(v) tubular "

(vi) Plate girder —



(vii) BOX girder et.

Types of section of flexure member :- according to local buckling

(i) Plastic section Class - 1

(ii) compact " " - 2

(iii) Semi - compact " " - 3

(iv) Slender " " - 4

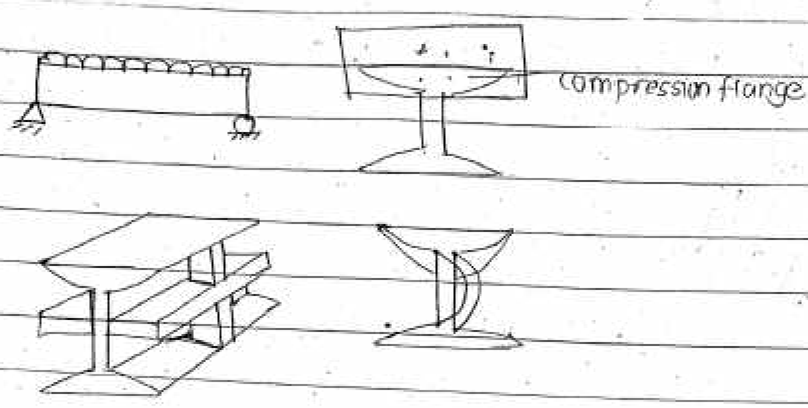
Elements of flexure member

(i) outstanding

(ii) Internal

(iii) Tapered

Types of beam as per lateral support provided to compression flange
1. laterally supported
2. laterally unsupported.



laterally supported beam:-

Design steps

① calculate effective length of Beam: 8.1.1 page: 52

2) calculate total load

3) Calculate max BM & Shear force

4) calculate plastic section modulus $Z_{p\,required} = \left(\frac{M_{max} \cdot \gamma_{m0}}{f_y} \right)$

5. Select section such that, $Z_p > Z_{required}$ Table 46 page 138

6. ~~Se~~ classify the section (tab. 2. page 18)

7. check for shear force (8.4.1 page 59)

8. " " B.M (8.2.1.2 page. 53)

9. " " deflection (table: 5, page 31)

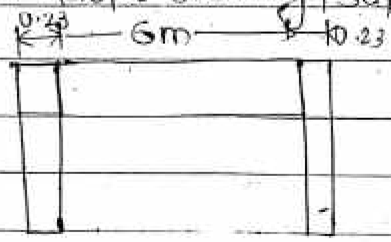
10. " " web buckling [8.4.2 page 59 (8.7.3 page. 67)

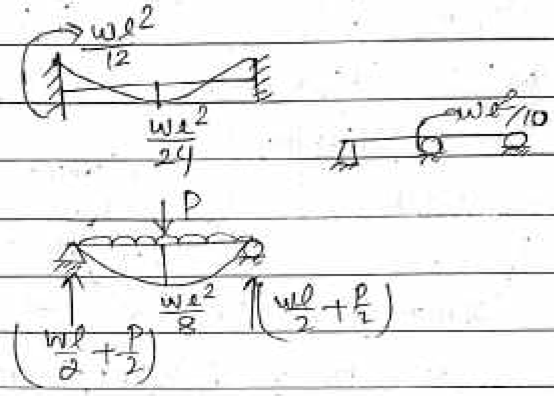
11. check for web crippling clause. 8.7.4 page. 67

Design a beam section having 6m clear span carrying a UDL of 46 kN/m ∵ compression flange is laterally supported throughout the section. Use Fe 410

Soln:-

① Step 1: Adopt bearing/support width 230 mm





$$\text{length eff} = 6 + \frac{0.23}{2} + \frac{0.23}{2}$$
$$= 6.23\ m$$

2. Step: 2 load calculation:

① UDL = 46 kN/m

ⓘⓘ Self wt = 1 kN/m (assume) always

∴ Total wt = 47 kN/m

factored UDL = 47 × 1.5

= 70.5 kN/m

(no center load)

Step: 3 $B.M = \frac{wl^2}{8} + \cancel{\frac{Pl}{4}}^{0} = \frac{70.5 \times 6.23^2}{8}$

$= 342.028\ kN/m$

Step: 4 $V_{max} = \frac{wl}{2} + \cancel{\frac{P}{2}}^{0}$

$= \frac{70.5 \times 6.23}{2}$

$= 219.606\ kN$

Step: 5: $Z_{p(req)} = \frac{M_{max} \cdot \gamma_{m0}}{f_y} = \frac{342.038 \times 10^6 \times 1.10}{250}$

$= 1.5041 \times 10^6\ mm^3$

$= 1504.1\ cm^3$

Step. 6: Select, ISMB 450, $Z_p = 1533.36\ cm^3 > Z_p$ required.

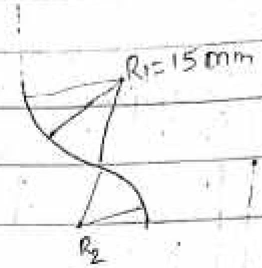
Then,

$I_{xx} = 30400 \ cm^4$

$I_{yy} = 839 \ cm^4$

$t_f = 17.4 mm$

$t_w = 9.4 mm$



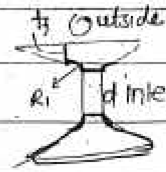
$h = D = 4\cancel{5}0 mm = 450 mm$

$B = 1\cancel{5}0 mm = 150 mm$

$Z_{ex} = 1350 cm^3$

$r_x = 182 mm$

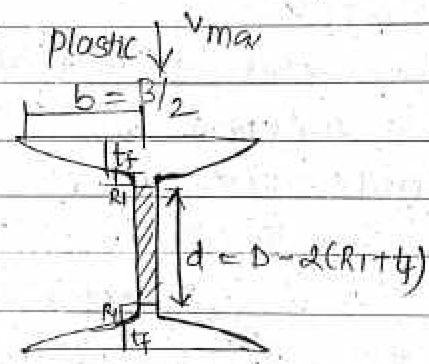
$r_y = 301 mm$



d internal of web $= D - R_1 - R_1 - 2t_f$

For outstanding element of compression flange for rolled section

$$\frac{b}{t_f} = \frac{75}{17.4} = 4.33 \le 9.4\varepsilon$$ plastic



$$b = \frac{B}{2} = \frac{150}{2} = 75$$

$$d = D - 2(R_1 + t_f)$$
$$= 450 - 2(15 + 17.4)$$
$$= 385.2 mm$$

$$\frac{d}{t_w} = \frac{385.2}{9.4} = 40.98 \le 84\varepsilon$$ (plastic) (Neutral axis at mid depth)

Hence, the section is plastic.

Step: 7: 8.4.1

$$V_d = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m_0}} = \frac{h t_w \times f_y}{\sqrt{3} \times 1.10} = \frac{450 \times 9.4 \times 250}{\sqrt{3} \times 1.10}$$

$= 555.34 \ KN > 219.6 \ KN$ (OK)

$$V_d' = 0.6 V_d$$
$$= 0.6 \times 555.34$$
$= 333.026 \ KN > 219.6$ so, it is low shear case

< 219.6 High shear case

Page: 53

Step: 8 :

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$$

$$= \frac{1.0 \times 1533.36 \times 10^3 \times 250}{1.10} \le \frac{1.2 Z_{ex} f_y}{\gamma_{m0}}$$

$$\le \frac{1.2 \times 1350 \times 10^3 \times 250}{1.10}$$

$$= 3485\ kN\text{-}m \le 368.18\ kN\text{-}m$$

$$\therefore M_d = 348.5\ kN\text{-}m \ge 342.03\ kN\text{-}m$$

Step: 9:

not factored load

$$\delta = \frac{5}{348} \frac{W l^4}{E I_x} + \frac{P l^3}{48 E I_x}$$

$$= \frac{5}{348} \times \frac{47.05 \times (6230)^4}{2 \times 10^5 \times 30400 \times 10^2}$$ page: 31

$= \cancel{22.74}mm$

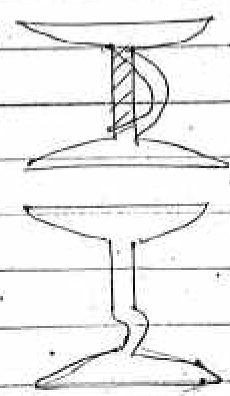
$= 15.163mm$

~~Step: 10~~ $\delta_{per} = \frac{Span}{300}$ for brittle

$$= \frac{6230}{300}$$

$= 20.76mm >> 15.163\ mm$ (OK!)

Step: 10 :



$\frac{d}{t_w} = 40.98 \le 67\,\varepsilon$

no need of web buckling check.

(page 59) 8.4.2

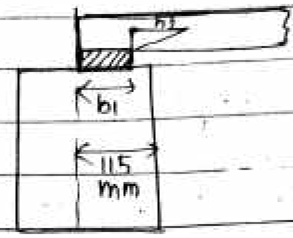
Other wise we have to check

If $\frac{d}{t_w} > 67\,\varepsilon$

Step: 11 Web crippling

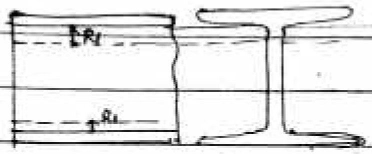
stiff bearing length (b_1)

$= 100$ mm



$n_2 = 2.5(t_f + R_1)$

$= 2.5(17.4 + 15) = 81$ mm



$\therefore F_w = \dfrac{(b_1 + n_2)\, t_w * f_y}{\gamma_{m_0}}$

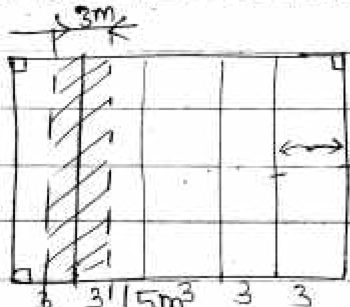
$= \dfrac{(100+81) \times 9.4 \times 250}{1.10}$

$= \cancel{386691}$ kN $\geq V_{max}$

$= 386.681$ kN $\geq V_{max}$

Design a steel beam of 6m span supporting a slab of 150 mm (RCC) which is provided with floor finish (punning) of 25mm, it also supports a live load of 6 kN/m^2. The beam is provided at 3m c-c spacing in a hall of 6x15 m. The beam is laterally supported throughout the span.

Soln:-



$\gamma_{rcc} = 25$ kN/m^3, $\gamma_{pcc} = 24$ kN/m^3

end beam → half load

intermediate → full load

← critical for

Step 1:- assume support width = 230 mm

$l_{eff} = 6 + 0.23$

$= 6.23$ m

$= 6230$ mm

Step: 2: load calculation

(i) slab load $= \gamma_{rcc} \times V = \dfrac{25 \times 3 \times 0.15 \times 6.23}{6.23} = 11.25$ kN/m

(ii) floor finish $= 24 \times 3 \times 0.025 \times \dfrac{6.23}{6.23} = 1.8$ kN/m

(iii) live load $= \dfrac{6 \times 3 \times 6.23}{6.23} = 18$ kN/m self load = 1 kN/m

Total load = 32.05 kN/m

$\therefore$ factored load = 48.075 kN/m

Step: 3 $BM = \frac{wl^2}{8} + \frac{pl}{4}$

$= 233.241$ kN-m

Step 4: $V_{max} = \frac{wl}{2} + \frac{P}{2}$

$= \frac{48.075 \times 6.23}{2}$

$=$ ~~132.965 kN~~

$= 149.753$ kN

Step: 5

$Zp_{(req)} = \frac{M_{max} \cdot \gamma_{m0}}{f_y}$

$= \frac{233.241 \times 1.10}{250}$

$= 1026.2\ cm^3$

Step: 6 select: ISMB 400 $Z_p = 1176.18\ cm^3 > Z_p$ required

Then

$I_{xx} = 20500\ cm^4$

$I_{yy} = 622\ cm^4$

$t_f = 16$ mm

$t_w = 8.9$ mm

$h = D = 400$ mm

$B = 140$ mm

$Z_e = 1020\ cm^3$

$r_x = 162$ mm

$r_y = 28.2$ mm

$r_1 = 14$ mm

for outstanding element of compression flange for rolled section

$$\frac{b}{t_f} = \frac{B}{2t_f} = \frac{140}{2\times16} = \frac{70}{16} = 4.3 \leq 9.4\epsilon \quad \text{(plastic)}$$

$$d = D - 2(t_f + R_1) = 400 - 2(16+14)$$
$$= 400 - 60$$
$$= 340 \text{ mm}$$

$$\frac{d}{t_w} = \frac{340}{8.9} = 38.20 \leq 84\epsilon \quad \text{(plastic)}$$

Hence, section is plastic.

Step: 7.

$$V_d = \frac{A_v f_y}{\sqrt{3}\gamma_{m0}} = \frac{h t_w \times f_y}{\sqrt{3}\times1.10} = \frac{400\times8.9\times250}{\sqrt{3}\times1.10} =$$
$$= 467.128 \text{ kN} > V_{max}$$

$$V_d' = 0.6 V_d = 280.277 \text{ kN} > V_{max}$$ ✓ low shear case

Step: 8 $$M_d = \frac{\beta_b \times Z_p f_y}{\gamma_{m0}} = \frac{1\times1176.18\times250}{1.10} \leq \frac{1.2\times1020\times250}{1.10}$$
$$= 267.295 \leq 278.181$$
$$\therefore M_d = 267.295 \text{ kN-m} \geq 233.241 \text{ (OK!)}$$

Step: 9 : $$\delta = \frac{5}{348}\frac{wl^4}{EI_x} + \frac{Pl^3}{48EI_x}$$

$$= \frac{5}{348}\times\frac{32.05\times6230^4}{2\times10^5\times20500}$$

alternatively:

assume, $V = 300\ kN < 467.128\ kN$ OK!

$V' = 0.6V_d = 280.276 < 300$ (High shear case)

8.2.1.3 for High shear case

step: 8

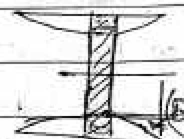
$$M_{dv} = M_{dv} = M_d - \beta(M_d - M_{fd}) \le \frac{1.2 Z_e \times f_y}{\gamma_{m0}}$$

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} = 267.314\ kN$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 300}{467.128} - 1\right)^2$$

$$= 0.0809$$

$$M_{fd} = \frac{Z_{fd} \times f_y}{\gamma_{m0}}$$



$(D/2 \times t_w)$ A

$$Z_{fd} = Z_p - 2\left[\frac{D}{2} \times t_w \times \frac{D}{4}\right] \times 10^{-3}$$

$y - D/4$

$=$

$= 820.18\ cm^3$

$$\therefore M_{fd} = \frac{820.18 \times 250}{1.10} = 186.405\ kN\text{-}m$$

$$\therefore M_{dv} = 267.314 - 0.0809(267.314 - 186.405)$$

$$= 260.77 \qquad \le \frac{1.2 \times 1020 \times 250}{1.10}$$

$$\le 278.181\ kN$$

$\therefore M_{dv} = 260.77 \ge M_{max}$

Design a steel beam of span 7m (leff) carrying a total UDL of 30 kN/m including self wt: depth of beam is restricted to 300mm & laterally supported throughout.

soln:-

Step 1: $l_{eff} = 7m$

Step 2: load calculation

Total UDL = 30 kN/m

~~factored~~ " = 30×1.5

= 45 kN/m

Step: 3. $V_{max} = \frac{Wl}{2} = \frac{45\times7}{2} = 157.5 kN$

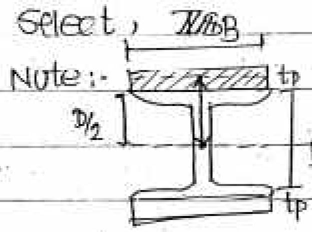
$M_{max} = \frac{wl^2}{8} = \frac{45\times7^2}{8} = 277.625$ kN

step: 4: $Z_p(\text{required}) = \frac{M_{max} \cdot \gamma_{m0}}{f_y} = \frac{\cancel{157}\,157.5\times1.10}{250}$

$= 122.75\ cm^3$

Select, ISMB

Note:-



if $Z_{p\,req} > Z_p$

$Z_{plate} = Z_{preq} - Z_p$

$Z_{plate} = 2\times(B\times t_p)\times(D/2 + \cancel{t_p/2}^{0})$

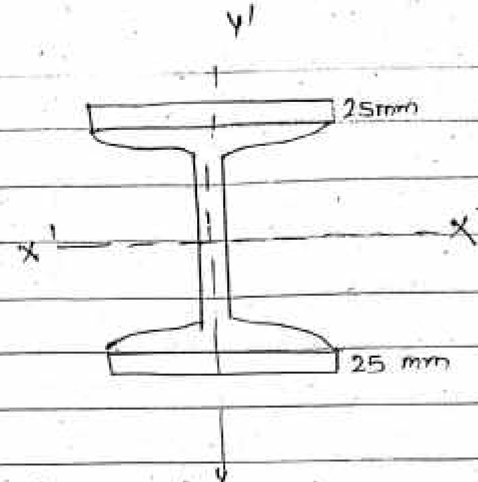
$= 2A\times y$

$\therefore t_p = \frac{Z_{plate}}{B \cdot D}$

thickness of extra plate = $\frac{Z_{plate}}{B\times D}$

for ISMB 250

$Z_p = 465.71 cm^3$	$Z_{ex} = 410\ cm^3$
$I_{xx} = 5130\ cm^4$	$D = 250$ mm
$I_{yy} = 335$ "	$B = 125$ mm
$r_1 = 13$ mm	$t_f = 12.5$ mm
	$t_w = 6.9$ mm



$Z_{plate} = Z_{p\,req} - Z_p$

$= 1212.75 - 465.71$

$= 747.04$

$\therefore t_p = \dfrac{Z_{plate}}{BD} = \dfrac{747.04}{250 \times 125}$

$= 2.39\,cm$

$\approx 2.5\,cm$

$I_{xx'} = [I_{xx} + Ah^2]_{MB} + [I_{xx} + Ah^2]_{plate} \times 2$

$= (5130 + 0) + \left[\left(\dfrac{12.5 \times 2.5^3}{12} + 12.5 \times 2.5 \times \left(\dfrac{2.5}{2} + \dfrac{2.5}{2}\right)^2\right] \times 2\right.$

$= 16978.96\,cm^4$

$Z_{xx'} = \dfrac{I_{x1}}{y_{max}} = \dfrac{16978.96}{15}$

$= 1131.93\,cm^3$

$Z_p' = (Z_p)_{MB} + (Z_p)_{plate}$

$= 465.71 + 2 \times 12.5 \times 2.5 \times \left(12.5 + \dfrac{2.5}{2}\right)$

अब नयाँ Value use गर्नु

$= 1325.086\,cm^3 \geq 1212.75\,cm^3$ Ok!

Shear area – web को area मात्र

no need to check high / low shear.

$\eta_2 = 2.5(t_f + R_1 + t_p)$

Flexure Member:

laterally unsupported Beam:

→ Torsional Buckling, hence flexural Kapacity in calculate considering the effect

Bending capacity is calculated as 8.2.2

Design laterally unsupported beam of span 3.5m carrying UDL of 20 KN/m including self wt.

Step 1: assume 230 mm support beam

$l_{eff} = 3.5 + 0.23$

$= 3.73 m$

Step: 2 load calculation: UDL = 20 kN/m

factored UDL = 20×1.5

$= 30 \ kN/m$

Step: 3 $V_{max} = \frac{Wl}{2} = \frac{30 \times 3.73}{2} = 52.5 kN$

$M_{max} = \frac{Wl^2}{8} = \frac{30 \times 3.73^2}{8} = 52.17 kN\text{-}m$

Step: 4 $Z_{p(req)} = \frac{M_{max} \gamma_{mo}}{f_y}$

$= \frac{52.17 \times 1.10 \times 10^6}{250}$

$= 229.548 \ cm^3$

Step: 5 select section such that,

$Z_p \geq 1.5 Z_{p \, req}$

Page 138.

ISMB 250 $Z_p = 465.71 cm^3$

$I_{xx} = 5130 \ cm^4$

$I_{yy} = 335 \ cm^4$

$r_x = 104 \ mm$

$r_y = 26.5 mm$

$B = h = 250 \ mm$

$B = b_f = 125$

$t_f = 12.5$

$t_w = 6.9$

$Z_{ex} = 10.39$

$R_1 = 13 \ mm$

Step: 6

$$b = \frac{B}{2} = \frac{125}{2} = 62.5\,mm$$

$$d = 250 - 2(12.5 + 13)$$

$$= 199\,mm$$

$$\frac{b}{t_f} = \frac{62.5}{12.5} = 5 \leq 9.4\epsilon \quad \text{plastic}$$

$$\frac{d}{t_w} = \frac{199}{6.9} = 28.8 \leq 84\epsilon \quad \text{plastic}$$

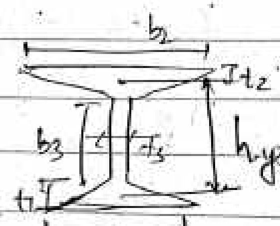
section is plastic.

Step: 7:

$$V_d = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m0}} = \frac{250 \times 6.9 \times 250}{\sqrt{3} \times 1.10} = 226.347 \geq 55.95\,KN \text{ OK!}$$

Step: 8 page 54. 8.2.2

$$G = \frac{E}{2(1+\mu)} \quad [\mu = 0.3]$$



$$G = \frac{2 \times 10^5}{2 \times 1.3} = 76923.07\,N/mm^2$$

$$I_w = (1 - \beta_f)\beta_f\, I_y \cdot h_y^2$$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = \frac{1}{2}$$

$$h_f = 250 - t_f = 250 - 12.5 = 237.8\,mm$$

$$I_w = (1 - 0.5) \times 0.5 \times 335 \times \left(\frac{237.8}{10}\right)^2$$

$$= 47240.234\,cm^6$$

$$I_t = \frac{\Sigma b_i t_i^3}{3} = \frac{2 \times 12.5^3 \times 125}{3} + \frac{225 \times 6.9^3}{3}$$

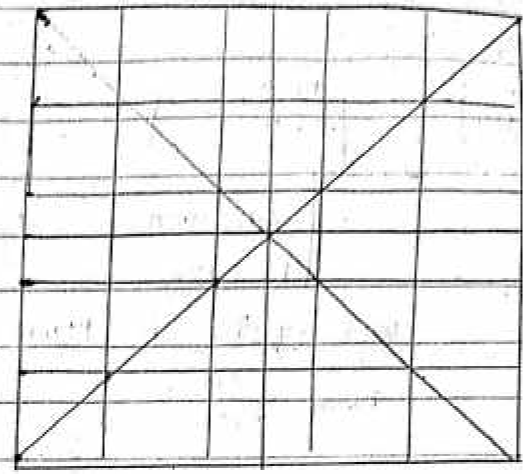
$$= 187398.592\,mm^4$$

Page :: 58

$L_{LT} = 0.7 \times L$

$= 0.7 \times 3.73$

$= 2.611\ m$



$$M_{cr} = \sqrt{\left\{\frac{\pi^2 E I_y}{(L_{LT})^2}\left[G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right]\right\}}$$

$= 1.65 \times 10^8$

$\lambda_{LT} = 0.84 \leq 0.86 \quad \lambda_{LT} = 0.84$

$\alpha_{LT} = 0.21$

$\phi_{LT} = 0.92$

$\chi_{LT} = 0.7721$

$f_{bd} = \dfrac{\chi_{LT} \cdot f_y}{\gamma_{m0}} = 175.468\ N/mm^2$

$M_d = \beta_d Z_p f_{bd} = 1 \times 465.71 \times 175.468 \times 10$

$= 81.7 \geq 52.17$

Step: 9 $\quad \delta_{max} = \dfrac{5}{384} \dfrac{w l^4}{EI}$

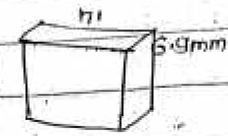
$= 4.91 mm$

$\delta_{per} = \dfrac{3.73 \times 10^3}{300}$

$= 12.437\ mm > 4.91$ OK!

Step: 10 : $\dfrac{d}{t_w} = 28.84 \leq 67\,\epsilon$

no need to check web buckling



$b_1 = 100mm$

$n_1 = 125$

Web length $= d = 199$ mm

$r_{min} = \frac{t_w}{\sqrt{12}} = \frac{6.9}{\sqrt{12}}$

$= 1.99$ mm

$\lambda = \frac{0.7d}{r_{min}} = \frac{0.7 \times 199}{1.99}$

$= 70$

Buckling class is C

$f_{cd} = 152 \ N/mm^2$

Aल 8.7.3 लो आठवा

$\therefore$ load carrying capacity $= f_{cd} \times A$

$= 152 \times (b_1 + n_1) \times t_w$

$= 152(225) \times 6.9$

$= 235.98$ kN

Step: II Stiff bearing length $= b_1 = 100$

load dist^n along H_3. $n_2 = \cancel{2.0}(t_f + R_1)$

$= \cancel{2.0}(12.5 + 13)$

$= \cancel{63.75}$ 25.5 mm

vertically $= 1 \times (R_1 + t_f)$

$= 1(13 + 12.5)$

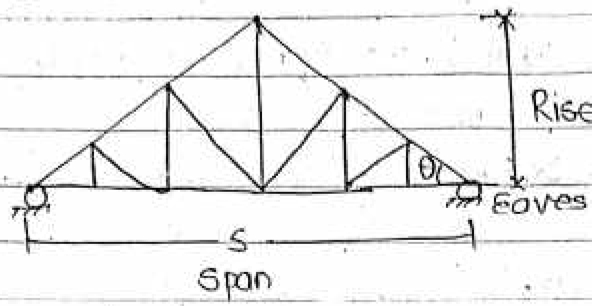
$= 25.5$ mm

$f_w = (b_1 + n_2) \times \frac{t_w f_{yw}}{\gamma_{m0}} = \frac{(100 + \cancel{63.75}\ 25.5) \times 6.9 \times 250}{1.10}$

$= 196.806 \geq V_{max}$ OK

CH-8 Roof truss:

If we need a large space without column, then its provided with truss. they are under action of wind load.



Pitch (p):- it is a ratio of riser to span. Generally taken as $\left(\frac{1}{6}\right)$

economical spacing is when, $t = p + r$

t = cost of truss.

p = cost of purlin

r = cost of roofing material

Dead load: Dead load of truss & roofing materials [IS-875] part 1

(i) Generally taken as $(20 + 6.6L)\ N/m^2$

when $LL \le 2\ kN/m^2$

(ii) it is multiplied by $\left(\frac{LL}{2}\right)$ if $LL > 2\ kN/m^2$

live load = (LL) (IS-875) part: 2

Live load on roof

Inaccessible = $0.75\ kN/m^2$ for 10° slope(θ), it is decrease by $0.02\ kN/m^2$

Accessible = $1.5\ kN/m^2$ [for every degree rise in θ]

3) Wind load (IS 875) Part 3 [NBC-104]

Basic wind speed (Annex A)

(i) NBC gives the ~~basic~~ two distinct basic wind velocity in case of Nepal.

47 m/s - ht from MSL $\leq$ 3000m

55 m/s - " " " $\geq$ 3000m

(ii) Design wind speed (V_d) = $V_b\, k_1 k_2 k_3$

$$\boxed{V_d = k_1 k_2 k_3 v_b}$$

(Risk factor) k_1 = table 1 — life expectancy & v_b

k_2 = Terrain factor Tab. 2

k_3 = Topographic " [Annex-C]

(iii) Pressure calculation at roof $(P) = 0.6 V_d^2$ N/m^2

(iv) Design pressure (P_d) = $[(C_{pe} - C_{pi}) P_d$

$\therefore$ force = $P_d \times A$

C_{pe} = external pressure coeff] table (4+1) = 5

[6.2.3] - $C_{p,i}$ = internal " "]

page 27

= ±0.2 [permeability -5%] normal permeability

= ±0.5 [5 to 20%]

= ±0.7 (permeability >20%]

Wind pr

calculate wind pressure on a roof truss having height to Eaves 12m located in central Ktm and designed life expectancy of 50yr & having total Ht of truss ~~fro~~ from ground of 15m, span of truss 20 m. Max dimension of building is 40m

Soln:-

1) basic wind speed. $V_b = 47$ m/s [Ht above MSL < 3000]

2). designed wind speed $V_d = k_1 k_2 k_3 V_b$

$k_1 = 1$

terrain category = 3

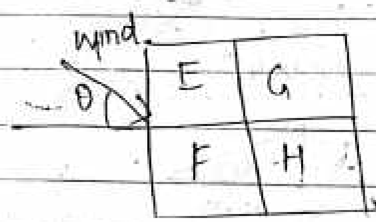
Eaves. Height = 12 m , Class: B

	k_2
10	0.88
15	0.94
12	0.904

$\therefore k_2 = 0.904$

$k_3 = 1$

$\therefore V_d = 1 \times 0.904 \times 1 \times 47 = 42.488$ m/s



3) wind pressure $(P) = 0.6 V_d^2$

$= 1083.13$ N/m^3

$C_{pi} = \pm 0.2$ [for normal permeability]

$\therefore P_d = (C_{pi} - C_{pe})\, P$

$C_{pe} \Rightarrow$

$\frac{h}{w} = \frac{12}{20} = 0.6 > 0.5$

Case (1)

$\theta = \tan^{-1}(3/10) = 16.69°$

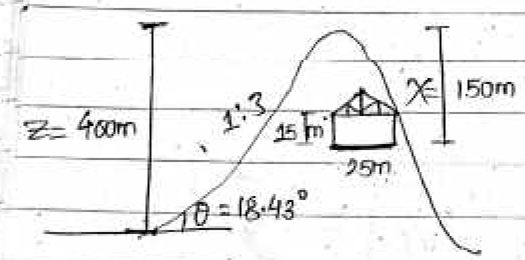
	EF	GH	EG	FH
C_{pe}	-0.8324	-0.5331	-0.8	-0.60
C_{pi}	+2	+2	+2	+2

Pd

$C_{pi} = +0.2$	$C_{pi} = -0.2$
$P_d(EF) = -1118.88$ Mpa	$P_d(EF) = -684.543$ Mpa
$P_d(GH) = -793.94$ "	$P_d(GH) = -360.682$ "
$P_d(EG) = -1083.138$ "	$P_d(EG) = -649.882$ "
$P_d(FH) = -866.51$ "	$P_d(FH) = -433.255$ "

So, $P_d = 1118.88$ Mpa.

Calculate wind pressure on a roof of a building located at chandragiri having ht to top from bottom of hill is 400m and avg. slope of 1:3, building is located at 150m downward from the crest, permeability of building is normal & having life expectancy of 100 yrs. Ht of building is 15m to the eaves & pitch of $\frac{1}{4}$ with span of truss 25m.



Soln:-

(i) $V_b = 47$ m/s

(ii) $V_d = k_1 k_2 k_3 V_b$

$k_1 = 1.07$

$k_2 = 0.97$ for Terrian category 2 Building class 'C'

$k_3 = 1 + CS$

$\theta > 17^\circ$ so $C = 0.36$

$Z = 400$m

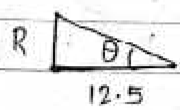
$H = 15$m

$X = 150$m

$l_e = \frac{Z}{0.3}$ for $\theta > 17^\circ$

$= \frac{400}{0.3}$

$= 1333.33$ m



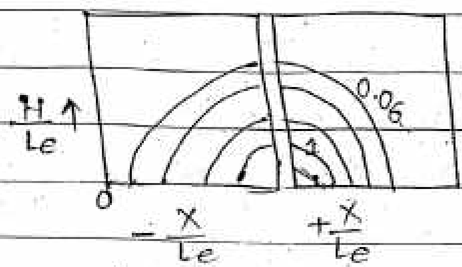
$\frac{R}{12.5} = \frac{1}{4}$

$\therefore R = \frac{12.5}{4}$

$= 3.125$m

$$\frac{X}{Le} = \frac{150}{1333.33} = 0.1125$$

$$\frac{H}{Le} = \frac{15}{1333.33} = 0.01125$$



$\therefore S = 1$

$\therefore k_3 = 1 + 0.36 \times 1$

$= 1.36$

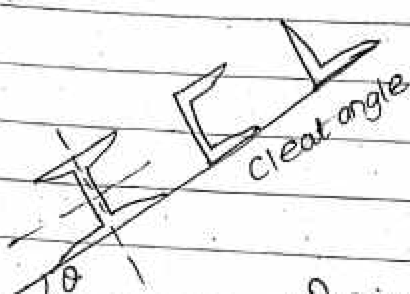
$\therefore Vd = 1.07 \times 0.97 \times 1.36 \times 47$

$=$

Design of purlin

It is a inclined section placed horizontally on the roof of truss.
There are 3 section particulary used in purlins.

- I-section
- Channel section
- Angle section.



load both direction मा moment आउँछ,
combined action check गर्ने

Design of I section & channel section purlin

1. calculate effective length
2. calculate total load
3. calculate max. BM and SF
4. calculate Plastic section modulus required,

$$Z_{p.req} = \frac{M_z \gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right) \times \frac{M_y \gamma_{mo}}{f_y}$$

take, $\left(\frac{d}{b}\right) = 2$

5. Select section such that. $Z_p > Z_{p\,req}$.
6. Section classification
7. check for shear force
8. check for BM
9. check for deflection
10. check for combined bending. $\left(\frac{1}{\frac{M_{dz}}{M_z}} + \frac{1}{\frac{M_{dy}}{M_y}}\right) \leq 1.0$

Design I section purlin for following data:

(i) Span of truss = 12m

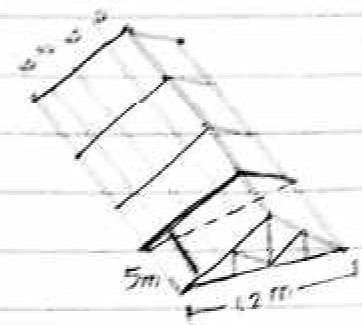
(ii) spacing of truss = 5m

slope of roof = 30°

wind load = $1.5\ kN/m^2$

Dead load = $200\ N/m^2$

live load = $0.75\ kN/m^2$



Step:1 Effective length of purlin = 5m

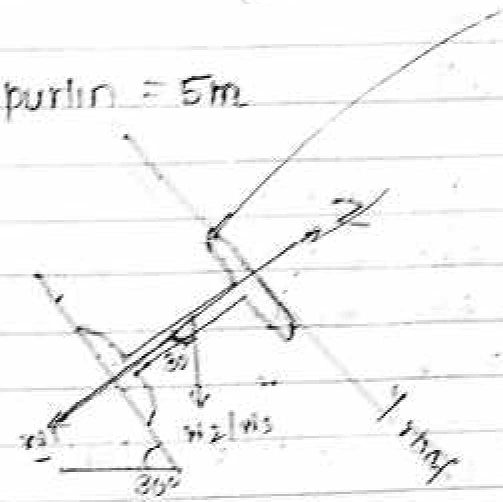
Step:2 load calculation

$W_1 = 1.5 \times 2\ kN/m$

$= 3\ kN/m$

$W_2 = 0.2 \times 2 = 0.4\ kN/m$

$W_3 = 0.75 \times 2 = 1.5\ kN/m$



factored load along z axis $= [W_1 + (W_2+W_3)\cos\theta] \times 1.5$

$= 6.97\ kN/m$

factored load along y axis $= (W_2+W_3)\sin\theta \times 1.5 = 1.42\ kN/m$

Step: 3. $M_z = \frac{w l^2}{10}$ $\qquad M_y = \frac{w_y l^2}{10}$

$= \frac{6.97 \times 5^2}{10}$ $\qquad = \frac{1.42 \times 5^2}{10} = 3.563\ kN\text{-}m$

$= 17.425\ kN\text{-}m$

Step: 4. $Z_{p\,req} = \frac{M_z \gamma_{m0}}{f_y} = \frac{17.425 \times 10^6 \times 1.10}{250} + \frac{5 \times 3.563 \times 10^6 \times 1.10}{250}$

$+ 2.5 \left(\frac{b}{d}\right) \times \frac{M_y \gamma_{m0}}{f_y} = 155.9\ cm^3$

take ISMB 200 having $Z_p = 253.86 cm^3$

$h = 200$ mm

$b_f = 100$ "

$t_w = 6.7$ "

$t_f = 10.0$ "

$I_{xx} = 2120 \, cm^4$

$R_1 = 11$ mm

$Z_z = Z_{ex} = 212.4 \, cm^3$

$Z_{ey} = 27.4 \, cm^3$

Step: 6: $b = \frac{100}{2} = 50$ mm

$d = 200 - 2(t_f + R_1)$

$= 200 - 2(10+11)$

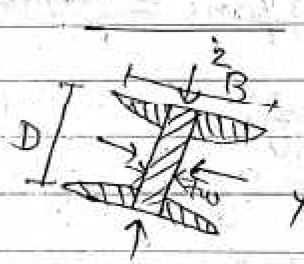
$= 158$

$\frac{b}{t_f} = \frac{50}{10} = 5 \leq 9.4\epsilon$ (plastic)

$\frac{d}{t_w} = \frac{158}{5.7} \leq 27.7 \leq 84\epsilon$ (plastic)

Step: 7:

$V_z = \frac{W_z L}{2} = \frac{6.97 \times 5}{2} = 17.42$ KN



$V_y = \frac{W_y L}{2} = \frac{1.42 \times 5}{2} = 3.55$ KN

$V_{dz} = \frac{A_v f_y}{\sqrt{3} \times 1.10} = \frac{D \times t_w \times 250}{\sqrt{3} \times 1.10}$

$= 149.5 > 17.142$ KN OK!

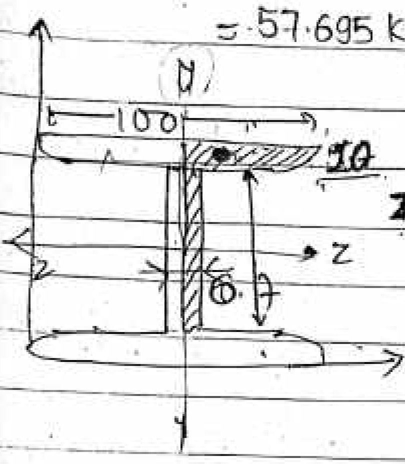
$V_{dy} = \frac{2 A_v f_y}{\sqrt{3} \times 1.10} = \frac{B \times t_f \times 250 \times 2}{\sqrt{3} \times 1.10}$

$= \cancel{209.91}$ 262.431 KN > 3.5 KN OK!

(1 × 4)

$$M_{dz} = \frac{\beta_b \times Z_{pz} \times 10^3 \times 250}{1.1}$$

$= 57.695$ kN-m > 17.42 kN-m $\quad Z_{py}$



$$Z_{py} = 4\left(\frac{B}{2} \cdot t_f \cdot \frac{B}{4}\right) + 2 \times \left[(D - 2t_f)\frac{t_w}{2} \times \frac{t_w}{4}\right]$$

$$= 4 \times \left(\frac{100}{2} \times 10 \times \frac{100}{4}\right) + 2\left[(200 - 2 \times 10) \times \frac{5.7}{2} \times \frac{5.7}{4}\right]$$

$= 51462.05\ mm^3$

$= 51.462\ cm^3$

$$\therefore M_{dy} = \frac{51.46 \times 10^3 \times 250}{1.10}$$

$= 11.681$ kN-m

Step: 9 $\quad \delta = \frac{5}{384} \times \frac{wL^4}{EI}$

$= 13.37$ mm

$$\delta_{per} = \frac{l}{180} = \frac{5000}{180} = 27 \text{ mm} > \delta \text{ (OK!)}$$

Z_{ex} (Z_p)

Step: 10:

$$\left(\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}}\right) \leq 1.0$$

$$\left(\frac{17.42}{57.69} + \frac{3.563}{11.681}\right) \leq 1.10$$

$\therefore 0.61 \leq 1.0$ OK!

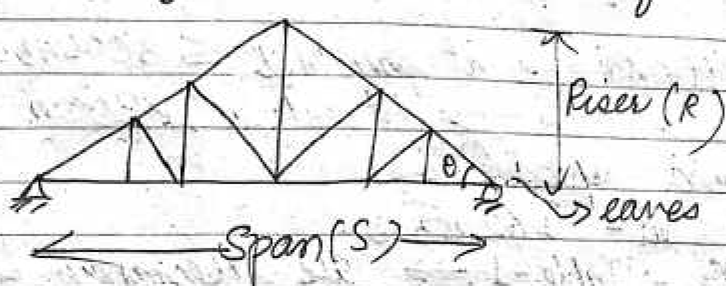
Same quest but use channel section:-

समस्या :- Z_p हुँदैन। Z_x सँग compare गर्नै

$\left.\begin{array}{l} Z_{pz} \\ Z_{py} \end{array}\right\}$ calculate गर्नै

Roof truss Chapter - 8

→ If we need a large space without column then it is provided with steel truss. They're under the action of wind load.



→ pitch (p) :- It is a ratio of riser to span. Generally taken as $\left(\frac{1}{6}\right)$

→ Economical spacing is when $t = p + r$

t = cost of truss

p = " " " purlin

r = " " " roofing material.

*) Dead load (IS 875 part-1).

Dead load of truss and roofing material.

Generally taken as $(20 + 6.6L)\ N/m^2$, when live load $(LL) \leq 2\ kN/m^2$

It is multiplied by $\left(\frac{LL}{2}\right)$ if $LL > 2\ kN/m^2$

→ Live Load (LL) (IS-875 part 2)

Live load on Roof

i) Inaccessible $= 0.75\ kN/m^2$

(ii) Accessible $= 1.5\ kN/m^2$

for $10°$ slope (θ), it decreases by $0.02\ kN/m^2$ for every degree rise in 'θ'.

$L = S$.

Wind load (IS-875 part 3) (NBC-104)

→ Basic wind speed (Annex A) (V_b).

NBC gives two distinct basic wind velocity in case of Nepal.

$47 m/sec \rightarrow$ ht. from MSL $\leq 3000m$.

$55 m/s \longrightarrow$ ht for MSL $> 3000m$

Design wind speed (V_d)

$$V_d = K_1 K_2 K_3 V_b.$$

Risk coefficient K_1 = Table-1 → life expectancy & V_b.

K_2 = Terrain factor (Table-2)

K_3 = topographic factor (Annex-C)

→ Pressure on roof $(P) = 0.6\,V_d^2$ Pascal or N/m^2

→ Design pressure on roof $(P_d) = (C_{pe} - C_{pi})\,P$

force $= P_d \times A$

where C_{pe} → external pressure coefficient

(6.2.3) C_{pi} → internal " " ← Table 5

(6.2.3.1)

(6.2.3.2) $C_{pi} = \pm 0.2$ (permeability – 5%)

↳ Normal Permeability.

$= \pm 0.5$ (Permeability (5% to 20%))

$= \pm 0.7$ (" (> 20%))

Q) Calculate wind pressure on a roof truss having height ~~to~~ to eaves; 12m located in central Kathmandu and designed life expectancy of 50 years and having total height of truss from ground of 15 meters. Span of truss is 20m. Maximum dimension of building = 40m

Ans Sol^n. - Basic wind speed (V_b) = 47 m/sec

साधारण $K_1 = 1$. (Table 1.)

(Table 2), Pg. 8 → (eave height) K_2.

Terrain category 3.

page 11. → class B → dimension (20m to 50m)

Height 12

10	~~0.91~~ 0.88
15	~~0.97~~ 0.94

$$\frac{10-15}{0.88-0.94} = \frac{12-15}{x-0.94}$$

$$x = 0.904$$

$k_3 = 1$ → generally 1.

Design wind ~~pressure~~ velocity (V_d) $= K_1 K_2 K_3 V_b$

$= 1 \times 0.904 \times 1 \times 47$

$= 42.488$ m/s

Wind pressure (P) $= 0.6 \times V_d^2$

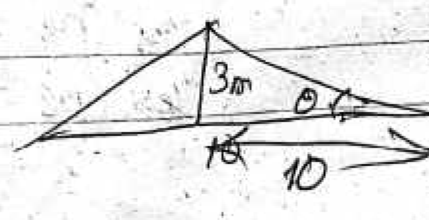
$= 0.6 \times 42.488^2$

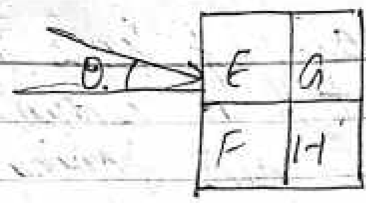
$= \cancel{13}\ 1083.138$ N/m^2.

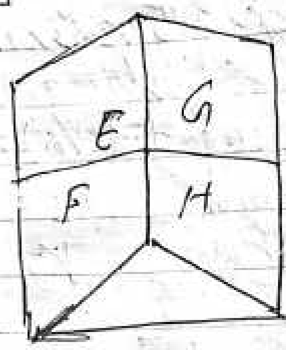
$C_{pi} = \pm 0.2$ (normal permeability).

$$\frac{h}{w} = \frac{\cancel{12}\,6}{\cancel{20}\,10} = 0.6 \quad 0.5 < 0.6 < 1.5$$

$$\theta = \tan^{-1}\frac{3}{10} = 16.69^\circ$$







	EF	GH
10	-1.1	-0.6
20	-0.7	-0.5

$$\frac{10-20}{-1.1+0.7} = \frac{10-16.69}{-1.1+x}$$

$\therefore x =$

~~(Cpe)EG =~~ $(Cpe)_{EF} = -0.832$

$$\frac{10-20}{-0.6+0.5} = \frac{10-16.69}{-0.6+x}$$

$(Cpe)_{GH} = -0.533$

$$\frac{10-20}{-0.8+0.8} = \frac{10-16.69}{-0.8+x}$$

$\therefore x = -0.8$

$(Cpe)_{EG} = -0.8.$

$(Cpe)_{FH} = -0.6$

for $Cpi = +0.2$

$Pd = (Cpe - Cpi)P$

for $(Cpe)_{EF} = -0.832$, $Pd =$ ~~684.54 N/m²~~ $-1117.8 N/m^2$

for $Cpe = -0.533$, $Pd = -793.94 N/m^2$

for $Cpe = -0.8$, $Pd = -1083.138 N/mm^2$

for $Cpe = -0.6$, $Pd = -866.5 N/m^2$

for $C_{pi} = -0.2$

$$P_d = (C_{pe} - C_{pi}) P.$$

for $C_{pe} = -0.832$, $P_d = -684.5 N/m^2$

for $C_{pe} = -0.533$, $P_d = -360.68 N/m^2$

for $C_{pe} = -0.8$, $P_d = -649.88 N/m^2$

for $C_{pe} = -0.6$, $P_d = -433.25 N/m^2$.

Maximum design pressure $= 1174.8 N/m^2$

$= 1.117 KN/m^2$

Q) Calculate wind pressure on a roof of a building located at Chandragiri Hill having height to top from bottom of hill of 400 m and average slope of 1:3. The building is located at 150m downwards from crest. The permeability of ~~building~~ is normal and having life expectancy of 100 yrs. The height of building is 15m to the eaves and ~~peak~~ pitch of $\frac{1}{4}$ width span of truss 25 m. The building is 25×60 m in dimension

Ans $V_b = 47$ m/sec

$K_1 = 1.07$

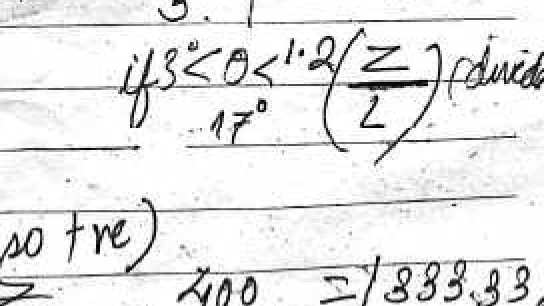
Terrain category - 2

Building class is 'C'

$K_2 = 0.97$

For K_3 :-

$K_3 = 1 + C_s$



$\theta = \tan^{-1}\left(\frac{1}{3}\right) = 18.43^\circ$

if $3^\circ < \theta < 17^\circ$ $1.2\left(\frac{Z}{L}\right)$ (divide

$C = 0.36$

$Z = 400m$. $H = 15m$.

$x = 150m$ (downwards so +ve)

Pg. no. 55 $L/L_e = \frac{Z}{0.3} = \frac{400}{0.3} = 1333.33 m$

$\frac{X}{L_e} = 0.1125$.

$\frac{H}{L_e} = \frac{15}{1333.33} = 0.01125$.

(Cliff and encarpment.)

From fig 14 : $s = 1$.

$$K_3 = 1 + C \times s$$
$$= 1 + 0.36 \times 1 = 1.36$$

~~X) Partios.~~

Purlin :-

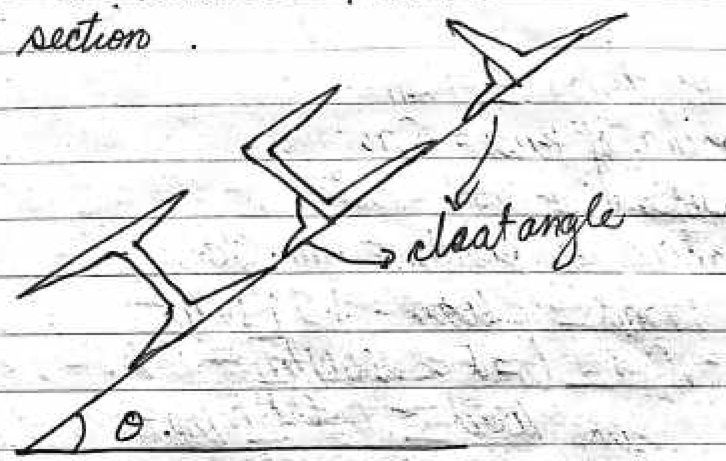
Design of Purlin:

→ It is an inclined section placed horizontally on the roof of truss

→ There are three section particularly used in purlins

a) I-section (b) channel section.

c) Angle section.



Q) Design steps of I-section or channel section purlin

1) Calculate effective length.
2) Calculate total load.
3) Calculate maximum BM & shearforce
4) Calculate plastic section modulus required.

$$Z_{p,req} = \frac{M_z \gamma_{m0}}{f_y} + 2.5\left(\frac{d}{b}\right)\frac{M_y \gamma_{m0}}{f_y}$$

Generally $\frac{d}{b} = 2$

5) Select section such that $Z_p > Z_{p,req}$.
6) Classify the section
7) Check for shear force,
8) Check for Bending moment
9) Check for deflection 10) ~~Check for combined bending~~

10) Check for combined bending

$$\left(\frac{M_z}{M_{dz}}\right)+\left(\frac{M_y}{M_{dy}}\right) \le 1$$

Q) Design I section for following data:-

Span of truss = 12m

Spacing of truss = 5m

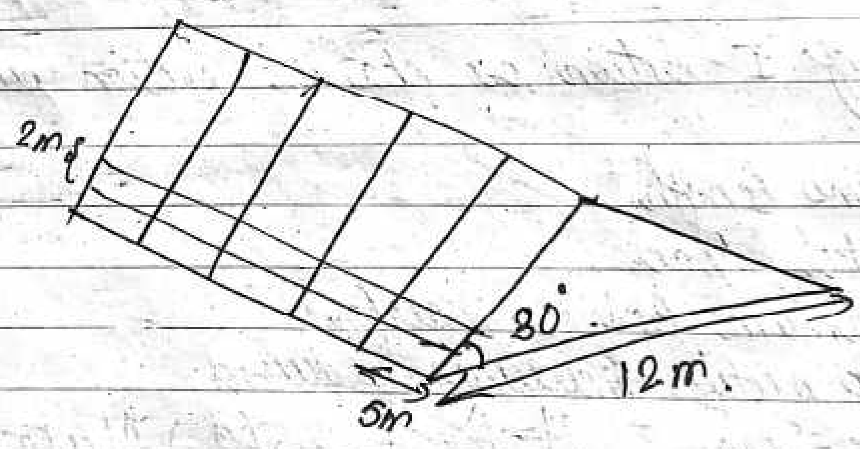
Slope of roof = 30°

Spacing of purlin = 2m c/c.

Wind load = 1.5 KN/m^2

Dead load = 200N/m^2

Live load = 0.75KN/m^2



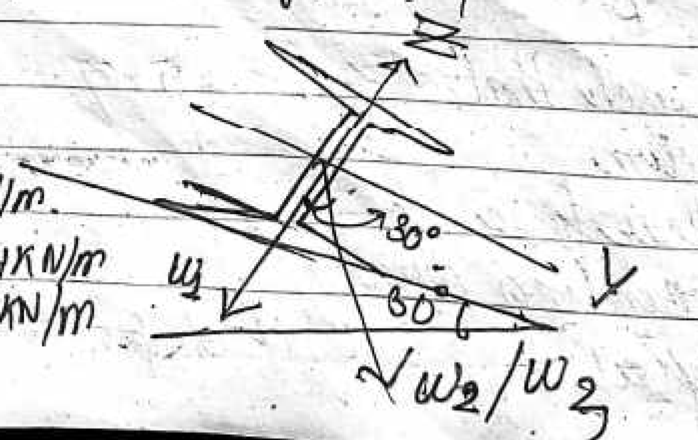
Step 1 :- Effective length of purlin = 5m

Step 2.

W_1 = 1.5×2 = 3KN/m.

W_2 = 0.2×2 = 0.4KN/m

W_3 = 0.75×2 = 1.5KN/m



Factored Load along z-axis $= \{w_1 + (w_2 + w_3)\cos\theta\} \times 1.5$

$= \{3 + (0.4 + 1.5)\cos 30°\} \times 1.5$

$= 6.97$ kN/m.

factored load along y-axis $= [(w_2 + w_3)\sin\theta] \times 1.5$

$= 1.42$ kN/m.

load along z axis effects Y-axis.

Step 3 :- M for continuous purlin section.

$$M_z = \frac{wl^2}{10} = \frac{6.97 \times 5^2}{10} = 17.425 \text{ kNm}.$$

$$M_y = \frac{1.42 \times 5^2}{10} = 3.55 \text{ kNm}.$$

Step 4 :- $Z_{pz} = \frac{M_z \gamma_{m0}}{f_y} + 2.5\left(\frac{d}{b}\right)\frac{M_y \gamma_{m0}}{f_y}$

$= \frac{17.42 \times 10^6 \times 1.1}{250} + \frac{2.5 \times 2 \times 3.56 \times 10^6 \times 1.1}{250}$

$= \cancel{199056 \text{ mm}^3 = 199.056 \text{ cm}^3}$

$= 155504.8 \text{ mm}^3.$

$= 155.505 \text{ cm}^3$

see Zp only from 38 other all from IS 808.

Step 5 : ISMB200.

$Z_p = 253.86 \text{ cm}^3$.

$D = h = 200$mm. $B = B_f = 100$mm.

$T = t_f = \cancel{6.7mm}$ 10mm; $t = t_w = 6.7$mm.

$I_{xx} = 2120 \text{ cm}^4$, $I_y = 137 \text{ cm}^4$, $R_1 = 11$mm.

~~$Z_y = Z_{ey} = 223.5 \text{ cm}^3$, $Z_z = Z_{ez} = 253.86 \text{ cm}^3$~~

$Z_z = Z_x = Z_{ex} = 212.4\ cm^3$

$Z_y = Z_{ey} = 27.4\ cm^4$

step 6 : $b = \frac{100}{2} = 50mm.$

$d = 200 - 2(t_f + R_1) = 158mm$

$\frac{b}{t_f} = \frac{50}{10} = 5 \leq 9.4\varepsilon$ (plastic)

$\frac{d}{t_w} = \frac{158}{6.7} = \underline{27.7} \leq 84\varepsilon$ (Plastic).

Hence, section is plastic.

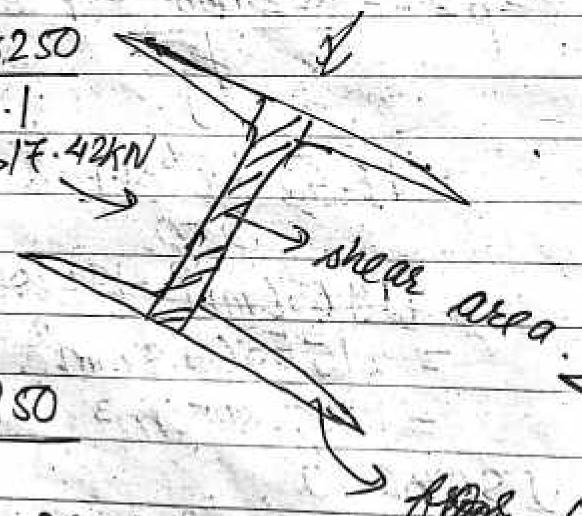
step7 : $V_z = \frac{W_z l}{2} = \frac{6.97 \times 5}{2} = 17.42kN$

$V_y = \frac{W_y l}{2} = \frac{1.42 \times 5}{2} = 3.55kN.$

$V_{dz} = \frac{A_v f_y}{\sqrt{3}\,\gamma_{mo}} = \frac{D \times t_w \times 250}{\sqrt{3} \times 1.1}$

$= 149.5kN > 17.42kN$

OK.



$V_{dy} = \frac{A_v f_y}{\sqrt{3}\,\gamma_{mo}}$

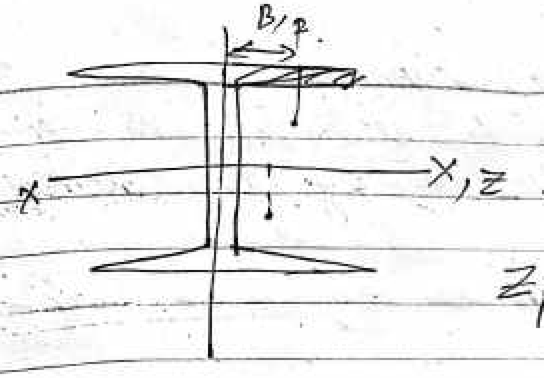
$= \frac{2 \times B \times t_f \times 250}{\sqrt{3} \times 1.1}$

$= \cancel{209}\ 262.43kN.$

Step 8 : $M_{dz} = \frac{\beta_b \times Z_{pz} f_y}{\gamma_{mo}}$

$= \frac{1 \times 253.86 \times 250 \times 10^3}{1.1}$ ~~57695.45~~

$= 57.69\ kN\text{-}m > 17.4$ OK



$$Z_{py} = 4 * \left[\frac{B}{2} \times t_f \times \frac{B}{4}\right] + 2 * \left[(D - 2t_f) \times \frac{t_w}{2} \times \frac{t_w}{4}\right]$$

$$= 51.46\ cm^3$$

$$M_{dy} = \frac{Z_{py} f_y}{\gamma_{mb}} = \frac{51.46 \times 10^3 \times 250}{1.1}$$

$$= 11.681\ kN\text{-}m > 3.563\ kNm$$

OK

Step 9 : $\delta = \frac{5}{384} \frac{wl^4}{EI}$

$$= \frac{5}{384} * \frac{\cancel{6.96} \times 5000^4 \times \cancel{6.6}\ 4.645}{2 \times 10^5 \times 2120 \times 10^4}$$

$$= \cancel{18.38\ mm}\ 8.91\ mm$$

$$\delta_{per} = \frac{l}{180} = \frac{5000}{180} = 27.78\ mm > 13.37\ mm\ OK.$$

Step 10 . M_d $\left(\frac{M_z}{M_{dz}}\right) + \left(\frac{M_y}{M_{dy}}\right)$

$$= \left(\frac{17.425}{57.69}\right) + \left(\frac{3.563}{11.681}\right)$$

$$= 0.61 < 1\ \text{OK}$$

HW - Same question for Channel section.
Compare Z_x see IS808 not Z_p.
Z_{pz} & Z_{py} also calculate.

Angle ~~sectn~~ Section Purlin
Design steps

1. Calculate effective length
2. Calculate total load and assume all the loads perpendicular to surface.
3. Calculate maximum bending moment & shear force.
4. Calculate section modulus required as:-

$$Z = \frac{M_z}{1.33 \times (0.66 f_y)}$$

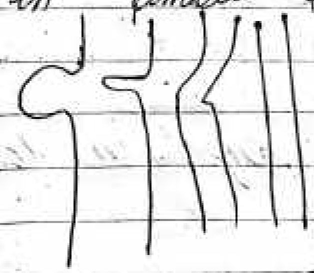
5. Select section such that $Z_x > Z_{req.}$ and depth not less than $\frac{span}{45}$ and width not less than $\frac{span}{60}$.
6. Check for bending moment capacity.

Chapter (9-12)

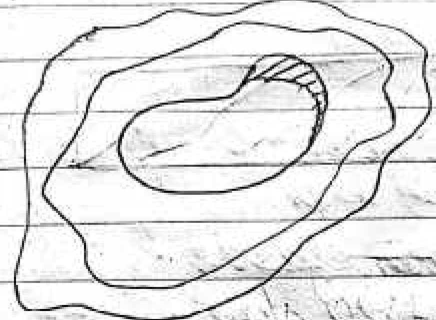
Timber Structure

a) Defects in timber ~~structures~~

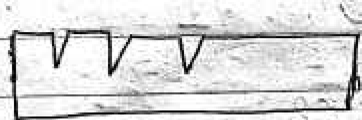
knots



b) shake



c) check.



(d) warping, etc.

*) Structural timber

A well seasoned timber without any major defect and having better to strength to weight ratio is known as structural timber.

Factors affecting strength of timber

(i) Moisture :- Moisture content should be in between 10 - 15% but moisture content of 12% is preferred. It should be free of free moisture.

(ii) Density :- Higher density represents higher amount of wood which means higher strength.

(iii) Defects :- There should not be major defects in timber.

(iv) Slope and type of grain

less slope → not good
more slope → good.

Types of timber as per ~~grain~~ grade.
1. Select grade. → 1:20 slope.
2. Grade-I → 1:15 – 1:20
3. Grade-II → 1:12

IS-883 pg-10 6.3
Except 'E' all values are multiplied by 1.16 and 0.84

*) Timber method is calculated by working stress method. if factored load is given then calculate using axial load

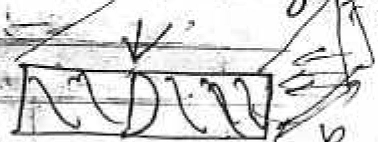
*) Grouping of timber (5.1.1 Table, pg 2)

(if notmentioned inside taken)
Working stress ⇒ Elastic method.

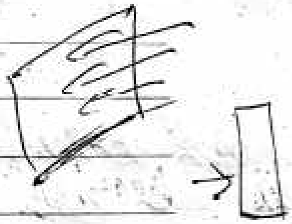
*) Design of Timber Beam:-
(i) Bending
(ii) shear (iii) Bearing (iv) deflection

Design a sal wood timber beam of span 4m carrying UDL of 16KN/m (8KN/m dead load + 8KN/m live load).

Sol^n Table-1.
for salwood Pg. 3.
$E = 126.7 \times 100 \ N/mm^2$.
$\sigma_b = 16.9 N/mm^2$ Shear stress (horizontal). d
$\tau_u = 0.94 N/mm^2$
$\sigma_{cb} = 4.6 N/mm^2$
$\rho = 805 Kg/m^3$. b



(i) Bending: (assume 200* 400 beam)
load calculation: (i) DL = 8KN/m
(ii) LL = 8KN/m.



(iii) Self weight = $\dfrac{\rho \times 0.2 \times 0.4 \times l \times g}{l}$

$= 805 \times 0.2 \times 0.4 \times 10$
$= 0.644 KN/m$.

total load = 16.644 KN/m.
$M_{max} = \dfrac{wl^2}{8} = \dfrac{16.644 \times 4.23^2}{8}$

Assume 230 mm support width
Effective length $= 4 + \dfrac{0.23}{2} + \dfrac{0.23}{2}$.
$= 4.23$.

Since, $M = \sigma \times Z$
or, $37.266 \times 10^6 = 16.9 \times Z$

$\therefore Z = 2202721.89 mm^3$. (Elastic section modulus.

$$Z = \frac{bd^2}{6}$$

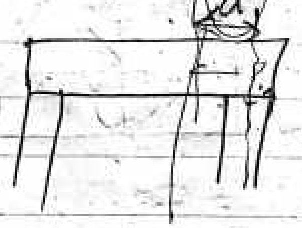
Let, ~~b=2d~~ $d = 2b$.

$$\frac{b(2b)^2}{6} = Z$$

$\therefore b = 148.9 mm$.

Let $b = 150mm$; $d = 300mm$.

(ii) For shear (support ko centre dx 'd' distance मा beam is critical in shear)



Shear stress $(\tau) = \frac{V}{bd} \times 1.5$

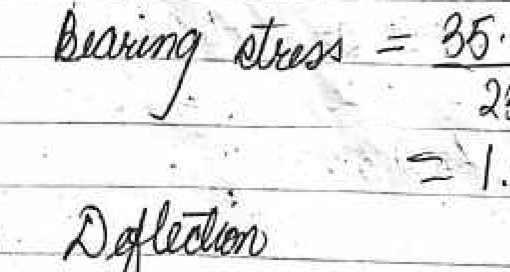
$$= \frac{30.2 \times 10^3 \times 1.5}{150 \times 300}$$

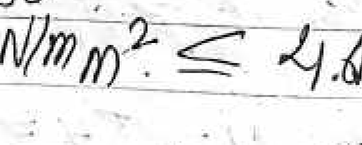
~~$= 0.671 N/mm^2 \leq 0.94 N/mm^2$~~

$\Longrightarrow = 1.0065 N/mm^2 \leq 0.94 N/mm^2$ ~~ok~~

(iii). Bearing :- Bearing force $= \frac{wl}{2} = \frac{16.644 \times 4.23}{2}$

$= 35.2 kN$





Bearing stress $= \frac{35.2 \times 10^3}{230 \times 150}$

$\frac{wl}{2}$

$= 1.02 N/mm^2 \leq 4.6 N/mm^2$ ok

(iv) Deflection

Take 200 × 400

$V = w(l/2 - d)$

Load for deflection = $2 \times DL + 0.75 \times LL$

$= 2 * (8 + 0.644) + 0.75 * 8$

$= 23.288 \, kN/m$.

$$\delta_{max} = \frac{5}{384} \frac{wl^4}{EI} = \frac{5}{384} \times \frac{23.288 * 4230^4}{126.7 \times 10^2 \times \left[\frac{150 \times 300^3}{12}\right]}$$

$= 22.7 mm$

$$\delta_{per} = \frac{span}{240} = \frac{4230}{240} = 17.62 \; mm \leq 22.7 mm$$

not ok.

Increase size to 200 x 400mm

$\delta_{max} = 7.18 mm \leq \delta_{per}$ OK.

$\delta \propto \frac{1}{I}$

$$\frac{\delta_{max}}{\delta_{per}} = \frac{I_{new}}{I}$$

$$I_{new} = \frac{\delta_{max}}{\delta_{per}} \times I$$

7) Adopt 200 x 400mm sal wood timber beam

Pg.no.11 7.5. Flexural

7.5.7.2.

$W = w * l$.

7.5.9. deflection

Design a deodar beam of span 5m to carry UDL of 20 KN/m if support of 200 mm is provided. Also note that timber is of select grade and to be used in terrace.

Ans $l_{eff} = l + \frac{0.2}{2} + \frac{0.2}{2}$

$= 5.2m$

For deodar wood. Terrace $\rightarrow$ outside.

$\rho = 557 Kg/m^3$

$E = 94.8 \times 10^2 N/mm^2$

$\sigma_b = 8.7 N/mm^2 \times 1.16 = 10.092 \ N/mm^2$.

$\sigma_{cd} = (2.1 N/mm^2) \times 1.16 = 2.436 \ N/mm^2$

$\tau_v = 0.7 N/mm^2 \times 1.16 = 0.812 \ N/mm^2$

Load calculation :- (200 * 400mm beam)

(i) UDL = 20 KN/m.

(ii) self weight = $557 \times 0.2 \times 0.4 \times 10$

$= 0.445 \ KN/m$.

total load. = 20.445 KN/m.

(i) Bending :

$M_{max} = \frac{wl^2}{8}$

$= \frac{20.445 \times 5.2^2}{8}$

$= 69.106 \ KNm$

$M = Z \times \sigma_b$ $\qquad \frac{M}{Z} = \sigma_b.$ $\qquad$ assume $d = 2b.$

We know, $M = \sigma_b \times Z$

$$\text{or, } 69.106 \times 10^6 = 10.09 \times \left[\frac{b \times \cancel{d}(2b)^2}{6}\right] \quad \because \text{(assume } b = 2b\text{]}$$

$$b = 217.39\,mm$$
$$\approx 250\,mm$$
$$d = 500\,mm.$$

since $d > 300mm$.

$$k_3 = 0.81 \times \left[\frac{d^2 + 89400}{d^2 + 55000}\right] \quad \begin{array}{c}\text{IS 883}\\ (\text{Cl. } 7.5.4)\end{array}$$

$$= \cancel{0.81} \quad 0.901$$

Moment carrying capacity $= (k_3 Z) \times \sigma_b$

$$= 0.901 \times \left[\frac{250 \times 500^2}{6}\right] \times 10.09$$

$$= 94.7\,kN\text{-}m \geq 69.106\,kNm$$

(ii) Shear :- $V_{max} = w\left(\frac{l}{2} - d\right)$

$$= 20.445\left(\frac{5.2}{2} - 0.5\right)$$

$$= 42.93\,kN.$$

$$\tau = 1.5\frac{V_{max}}{bd} = \frac{1.5 \times 42.93}{250 \times 500}$$

$$= 0.515\,N/mm^2 \leq 0.81\,N/mm^2 \text{ OK}.$$

(iii) Bearing stress.

Bearing force $= \frac{wl}{2}$

$$= \frac{20.445 \times 5.2}{2} = 52.135\,KN$$

Bearing stress = $\frac{52.135 \times 10^3}{250 \times 200}$

$= 1.04\ N/mm^2 \leq 2.436\ N/mm^2$ OK.

(iv) Deflection :-

$$\delta max = \frac{5}{384}\frac{wl^4}{EI}$$

$$= \frac{5}{384} * \frac{20.445 \times 5200^4}{94.8 \times 10^2 \times \left[\frac{250 \times 500^3}{12}\right]}$$

$$= 7.88mm.$$

$$\delta per = \frac{Span}{240} \quad (7.5.9)$$

$$= \frac{5200}{240}$$

$$= 21.67mm \geq 7.88mm \text{ OK}$$

Adopt 250 x 500 mm deodar timber beam

Design of Columns :-

Types :

i) Solid column

ii) Box/Built-up column

iii) Spaced column.

Solid column:

i) Short column : $\frac{l}{d} \leq 11$.

d = least lateral dimension

$\sigma'_{cp} = \sigma_{cp}$.

(ii) Intermediate column :- $11 < \frac{l}{d} \leq K_8$.

$$\sigma'_{cp} = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_8 d}\right)^4\right]$$

(iii) long column : $\frac{l}{d} > K_8$

$$\sigma'_{cp} = \frac{0.329E}{\left(\frac{l}{d}\right)^2}$$

l = length of column

σ_{cp} = Permissible compressive strength parallel to grain.

σ'_{cp} = modified " " " ,,

$$K_8 = 0.702\sqrt{\frac{E}{\sigma_{cp}}}$$

0.584 in K_8 & K_{10} should be replaced by 0.702 in code

$f = \sigma$ in code

Pg. 14 → $l = s$, $\sigma = f$.

6-7marks.

8) Calculate load carrying capacity of teak wood column of length 5m and having lateral dimension of 300×400 mm Timber is of grade II.

Ans For teak wood (UP),

$E = 99.7 \times 100 \, N/mm^2$

σ_{cp} (compression parallel to grain) $= 9.4 \times 0.84$
$= 7.896 N/mm^2$

Now, $\frac{l}{d} = \frac{5000}{300}$ d = least lateral dimension

$= 16.67 > 11.$

$K_8 = 0.702 \sqrt{\frac{E}{\sigma_{cp}}} = 0.702 \sqrt{\frac{99.7 \times 10^2}{7.896}}$
$= 24.945$

$11 < \frac{l}{d} \leq K_8$, so it is intermediate column.

$$\sigma_{cp}' = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_8 d}\right)^4\right]$$

$$= 7.896 * \left[1 - \frac{1}{3} * \left(\frac{5000}{24.945 \times 300}\right)^4\right].$$

$= \cancel{6.72}\ 7.87 N/mm^2$

Then, load carrying capacity $= \sigma_{cp} \times A$
$= 7.37 \times 300 \times 400$
$= 884.4 kN$

le
s

Q) Design Sal wood, timber column of 4m length to carry axial load of 300kN.

Ans: For sal wood.

$$E = 126.7 \times 10^2 \, N/mm^2$$

$$\sigma_{cp} = 10.6 \, N/mm^2$$

Then $A_{req} = \frac{P}{\sigma_{cp}} = \frac{300 \times 10^3}{10.6}$

$= 28301.886 \, mm^2$.

Assume, square column.

$$b = d = \sqrt{2830.886}$$

$$= 168.23 mm$$

$$= 200 mm$$

Now, $\frac{l}{d} = \frac{4000}{200} = 20 > 11$

$$K_8 = 0.702 \sqrt{\frac{E}{\sigma_{cp}}}$$

$$= 0.702 \sqrt{\frac{126.7 \times 10^2}{10.6}}$$

$$= 24.27$$

$11 < \frac{l}{d} \leq K_8$, so it is intermediate column

$$\sigma'_{cp} = \sigma_{cp} \left[1 - \frac{1}{3} \left(\frac{l}{K_8 d} \right)^4 \right]$$

$$= 10.6 * \left[1 - \frac{1}{3} \left(\frac{4000}{24.27 \times 200} \right)^4 \right]$$

$= \cancel{7.69 \, N/mm^2}$ $8.97 \, N/mm^2$

load carrying capacity $= \sigma'_{cp} \times A_{gr}$

$= \cancel{7.69} \times 8.97 \times 200 \times 200$

$= 358.82 kN > 300 kN$

(ii) Box / Built-up column:

a) short column:

$$\frac{l}{\sqrt{d_1^2+d_2^2}} \leq 8$$

$$\sigma'_{cp} = \sigma_{cp}$$

(b) Intermediate column.

$$8 < \frac{l}{\sqrt{d_1^2+d_2^2}} \leq K_g$$

$$\sigma'_{cp} = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_g\sqrt{d_1^2+d_2^2}}\right)^4\right]$$

(iii) long column:

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

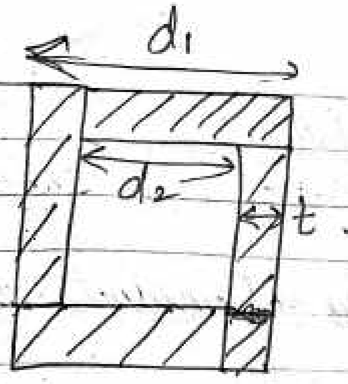
$$\sigma'_{cp} = \frac{0.329UE}{\left(\frac{l}{\sqrt{d_1^2+d_2^2}}\right)^2}$$

d_1+d_2 = least external & internal dimension

$$K_g = \frac{\pi}{2}\sqrt{\frac{UE}{5\sigma_{cp}}}$$

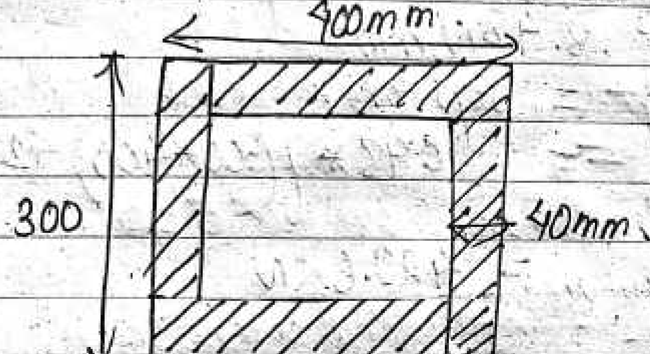
U = modification factor depending upon thickness of plank

200 * 300.



Cl. 7.6.2.5

Q) Calculate load carrying capacity of 6m box column made of oak wood having following section.



→ Sol^n:- for oak wood (West Bengal) @ 874 kg/m^3.

$E = 126.3 \times 10^2$ N/mm^2

$f_{cp} = 9.6$ N/mm^2

$d_1 = 300mm$, $d_2 = 300 - 2 \times 40 = 220mm$.

Then, $\frac{l}{\sqrt{300^2 + 220^2}} = 16.128 > 8$.

t	U
25	0.8
50	0.6

$$\frac{25-50}{0.8-0.6} = \frac{25-40}{0.8-x}$$

$\therefore x = 0.68$

$$K_g = \frac{\pi}{2}\sqrt{\frac{UE}{5\sigma_{cp}}} = \frac{\pi}{2} * \sqrt{\frac{0.68 \times 126.3 \times 10^2}{5 \times 9.6}}$$
$$= 21$$

Since, $8 < \frac{l}{d} < K_g$, so it is intermediate column.

$$\sigma'_{cp} = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_g\sqrt{d_1^2+d_2^2}}\right)^4\right]$$

$$= 9.6\left[1 - \frac{1}{3}\left(\frac{6000}{21\sqrt{300^2+220^2}}\right)^4\right]$$
$$= \cancel{7.934 N/mm^2}\ 8.48 N/mm^2$$

Load carrying capacity $= \sigma'_{cp} * A$
$$= 8.48 * [(300*400) - (220 \times 320)]$$
$$= 420.6 KN$$

Q) ~~Design a box column using~~

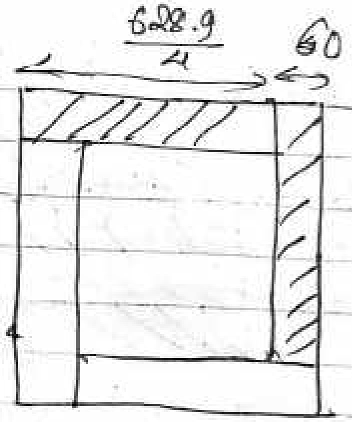
Design a box column using 60mm thick salwood plank to carry 400KN load in a column of 3.5m length

Ans Sol^n :- for salwood :-

$$E = 126.7 \times 10^2 \ N/mm^2$$
$$\sigma_{cp} = 10.6 N/mm^2$$
$$Area_{req} = \frac{400 \times 10^3}{10.6}$$
$$= 37735.85 \ mm^2$$

length of total plank $= \frac{37735.85}{60}$
$$= 628.9 mm.$$

Each side of column $= \frac{628.9}{4} + 60 = 217.25 mm$
$$\simeq 250 mm$$



~~$d_1 = 250 - 60$~~
~~$= 190$~~ ~~$d_2 = 250 -$~~ $d_1 = 250$

$d_2 = 250 - 2 \times 60 = 130\,mm$

$$\frac{l}{\sqrt{250^2 + 130^2}} = 12.42 > 8$$

l	U
25	0.8
50	0.6

$$\cancel{\frac{25-50}{0.8-0.6} = \frac{25-60}{0.8-x}}$$

~~$x =$~~

$U = 0.6$.

$$K_g = \frac{\pi}{2}\sqrt{\frac{0.6 \times 126.7 \times 10^2}{5 \times 10.6}}$$

$= 18.81$.

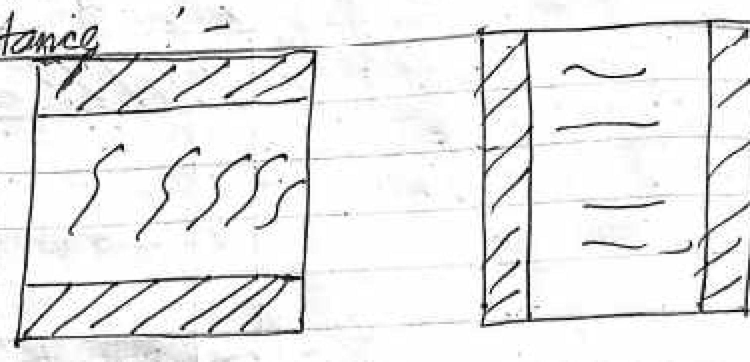
$$\cancel{8} \frac{l}{\sqrt{d_1^2 + d_2^2}} < K_g$$

$$\sigma'_{cp} = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_g\sqrt{d_1^2 + d_2^2}}\right)^4\right]$$

$= 9.93\ N/mm^2$

Load carrying capacity $= 9.93 * [(250 \times 250) - (130 \times 130)]$
$= 452.81\ kN > 400\,kN$ ok

Moment of resistance :-



*) Column under Bending & Axial combined.

$$\left(\frac{\sigma_{b,cal}}{\sigma_b}\right) + \left(\frac{\sigma_{c,cal}}{\sigma_{cp}}\right) \leq 1.0$$

$\sigma_{cp} \Rightarrow$ compressive permissible stress.

$\sigma_{b,cal} = \frac{M}{Z}$ = calculated Bending stress.

if as for σ_{cp} of in any condition we use σ'_{cp}.

$\sigma_{c,cal} = \frac{P}{A}$ - calculated compression stress.

Q) Check the safety of a solid column 300mm * 400mm of Sal wood having unsupported length of 4m and acted by axial force of 200KN and bending moment of 25KNm!

Ans Sol^n :- for salwood

$E = 126.7 \times 10^2 N/mm^2$.

$\sigma_b = 16.9 N/mm^2$.

$\sigma_{cp} = 10.6 N/mm^2$

Bending :-

$$\sigma_{b,cal} = \frac{M}{Z} = \frac{25 \times 10^6}{\frac{400 \times 300^2}{6}}$$

Minimum 'Z'

$= 4.17\ N/mm^2 \leq 16.9 N/mm^2$ (OK).

Compression: $\sigma_{c,cal} = \frac{P}{A} = \frac{200 \times 10^3}{300 \times 400}$

$= 1.67 N/mm^2.$

$\frac{l}{d} = \frac{4000}{300} = 13.33 > 11.$

$K_8 = 0.702 \sqrt{\frac{E}{\sigma_{cp}}}.$

$= 0.702 \cdot \sqrt{\frac{126.7 \times 10^2}{10.6}}$

$= \cancel{0.702}\ 24.27$

Since,

$11 < \frac{l}{d} \leq K_8$, it is intermediate column.

$$\sigma'_{cp} = \sigma_{cp} \left[1 - \frac{1}{3}\left(\frac{l}{K_8 d}\right)^4\right]$$

$= 10.28 N/mm^2 \geq 1.68 N/mm^2$ (OK).

Finally $\left(\frac{\sigma_{b,cal}}{\sigma_b}\right) + \left(\frac{\sigma_{c,cal}}{\sigma_{cp}}\right)$

$= \frac{4.17}{16.9} + \frac{1.68}{10.28}$

$= 0.41 \leq 1.0$ (OK).

Hence the section is safe.

*) Design salwood timber column of 5m to carry an axial load of 250KN and bending moment of 35KNm

Solⁿ: For sal wood.

$E = 126.7 \times 10^2\ N/mm^2$.

$\sigma_b = 16.9\ N/mm^2$

$\sigma_{cp} = 10.6\ N/mm^2$

$A_{req} = \frac{P}{A} = \frac{250 \times 10^3}{10.6}$

$= 23584.9\ mm^2$

Assume square column

$d = b = \sqrt{A_{req}}$

$= 153.57\ mm$

Adopt 200×300 (double calculated size column)

Chapter (10-12)

Timber structure

Spaced column:

→ All parameters similar to solid column only effective length is to be calculated.

$l_{eff} = \frac{l}{\sqrt{R.F}}$.

R.F = reduction factor

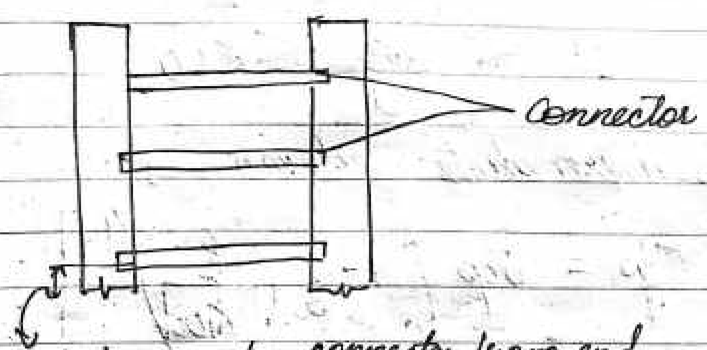
= 2.5 for distance of connector at $\frac{l}{20}$ from support

= 3 " " " at $\frac{l}{10}$ to $\frac{l}{20}$ " "

K_{10} instead of K_8.

$K_{10} = 0.702\sqrt{\dfrac{2.5E}{\sigma_{cp}}}$ & for long column.

$$\sigma'_{cp} = \frac{0.329E \times 2.5}{\left(l/d\right)^2}.$$



Q) Calculate load carrying capacity of a spaced column consisting of 2 planks 60×300 connected by connectors placed at 200mm from support if the length of the column is 4m. The column is made of sal wood.

Sol^n :-

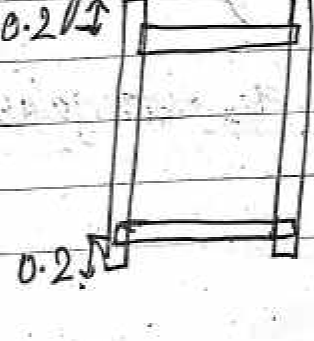
Distance of connector = 200mm

which is exactly $\frac{l}{20}$ $\left(\frac{4000}{20}\right)$ from support

Hence, R.F = 2.5

Pg. no. 14 7.6.3

(l_c) length of end connector to end connector $= 4 - 2\times0.2$
$= 3.6m$



$$l_{eff} = \frac{l_c}{\sqrt{2.5}} = 2.27m.$$

$$\frac{l_{eff}}{d} = \frac{2.27}{0.06} = 37.94 > 1$$

$$K_{10} = 0.702\sqrt{\frac{2.5E}{\sigma_{cp}}}$$

For salwood,
$E = 126.7 \times 10^2\ N/mm^2$
$\sigma_{cp} = 10.6\ N/mm^2$.

$= \cancel{39.83}\ 38.87$

Since, $11 < \frac{l_{eff}}{d} \leq K_{10}$

it is intermediate column.

$$\sigma'_{cp} = \sigma_{cp}\left[1 - \frac{1}{3}\left(\frac{l}{K_{10}d}\right)^4\right]$$

$$= 10.6\left[1 - \frac{1}{3}\left(\frac{37.94}{38.37}\right)^4\right]$$

$= 7.22\ N/mm^2$

load carrying capacity $= \sigma'_{cp} * A$
$= 7.22 * (2 * 60 * 300)$
$= 259.92\ kN$

Q) Design a spaced column to be used as a 8m long column carrying 250kN axial load and made of oak

Q) Design a circular salwood column of 8.5m length and having to carry 200kN axial load.

$\frac{l}{d}$ → for rectangular / square.

Ans $E = 126.7 \times 10^2\ N/mm^2$
$\sigma_{cp} = 10.6\ N/mm^2$

Area required = $\frac{200\times10^3}{10.6}$ = ~~188.67 mm²~~ 18867 mm²

Taking equivalent square section

$$d = \sqrt{18867}$$
$$= 137.35$$
$$\simeq 150\,mm$$

Equivalent circle :-

$$\frac{\pi}{4} * D^2 = 150^2$$

$$D = \ldots$$

(*) Nail connection in ~~kee~~ timber

-> Pitch $\geq$ 5d in tension.

$\geq$ 10d in compression

end distance $\geq$ 2.5 * d.

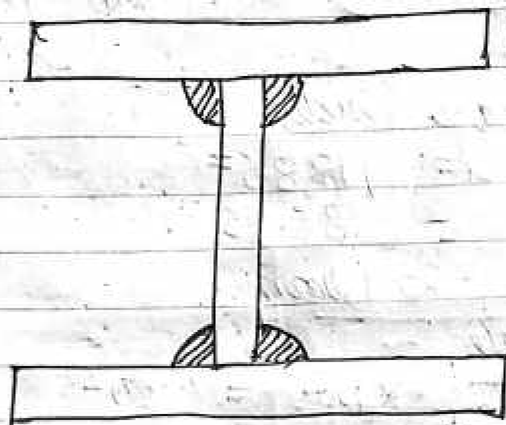
Minimum thickness of plank = 15mm.

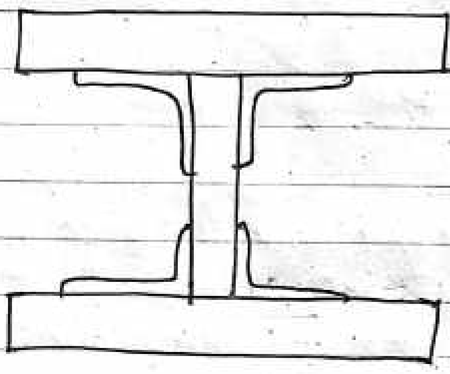
maximum thickness " " = 100mm.

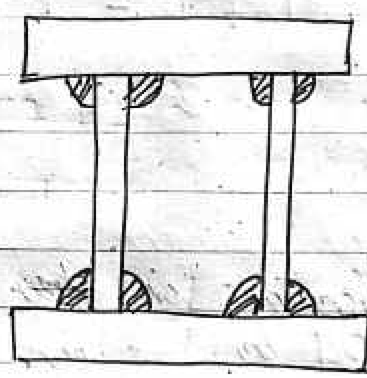
width of plank $\leq$ 8 * thickness of plank.

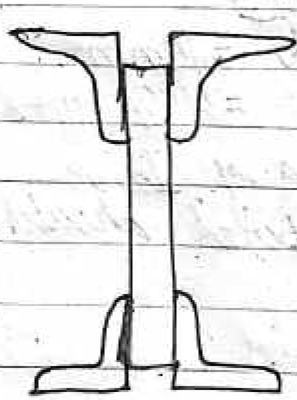
Width of gusset plate combined should be less than main member thickness

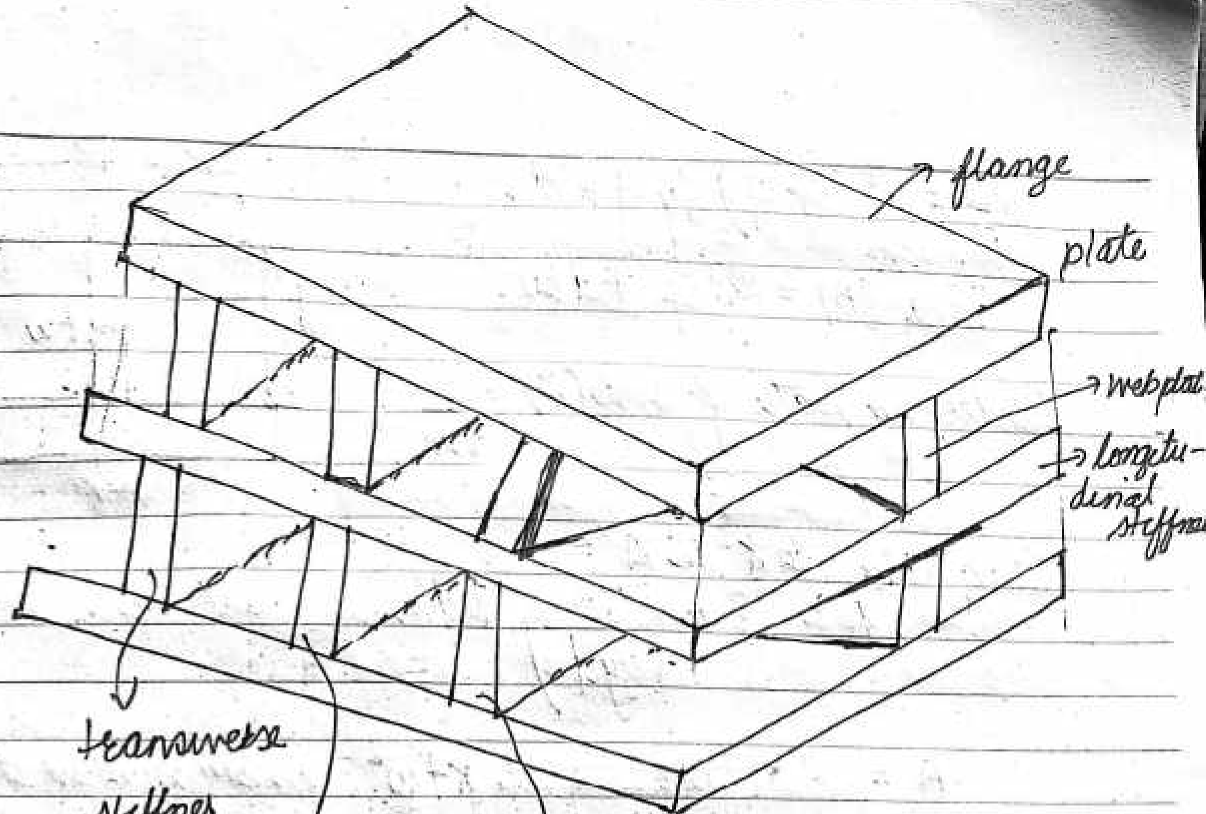
Plate girder (~~when~~ at high shear condition).











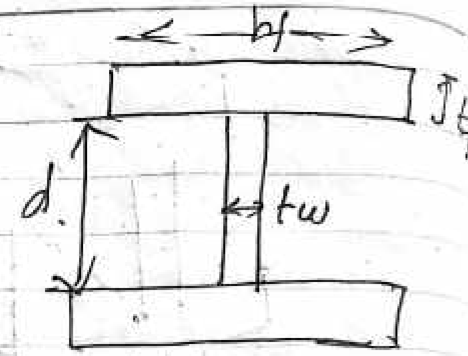
Componenents of plate girder: (8.6)

1) flange plate
2) Web - plate
3) splices (web & flange splice)
4) stiffner.

Economical depth of plate girder.

Depth taken between $\frac{1}{6}$ to $\frac{1}{15}$ of span but $\frac{1}{8}$ to $\frac{1}{12}$ of span is preferred. Reducing area may increase depth causes difficulty in transportation.

$$M_z = \left[(b_f t_f) f_y\right] * d.$$



Area $(A) = 2 b_f t_f + d t_w.$

Slenderness ratio of web $(K) = \dfrac{d}{t_w}.$

Using M_z & K in 'A'

$$A = 2 * \left(\frac{M_z}{f_y d}\right) + d t_w.$$

$$A = 2 * \left(\frac{M_z}{f_y K t_w}\right) + K t_w^2.$$

$$\frac{dA}{dt_w} = 2 * \left(\frac{M_z}{f_y K}\right) * \left(-\frac{1}{t_w^2}\right) + 2K t_w.$$

for minima, $\dfrac{dA}{dt_w} = 0$

$$0 = -2 * \left(\frac{M_z}{f_y K}\right) \frac{1}{t_w^2} + 2K t_w$$

$$\left(\frac{2 M_z}{f_y K}\right) = 2K t_w^3.$$

$$\therefore\ t_w = \left(\frac{M_z}{f_y K^2}\right)^{0.33}$$

$$A = 2\left(\frac{Nz}{fy \times tw}\right) + d\,tw$$

$$A = 2 * \left(\frac{Mz}{fy \times \frac{d}{k}}\right) + d * \frac{d}{k}.$$

D.w.r. to 'd'

$$\frac{dA}{dd} = -\frac{2 * Nz}{fy\, d^2} + \frac{2d}{k}$$

for minima

$$\frac{Z\,Nz}{fy\,d^2} = \frac{Zd}{K}$$

$$\left[\therefore\ d = \left(\frac{Nz\,K}{fy}\right)^{0.33}\right]$$

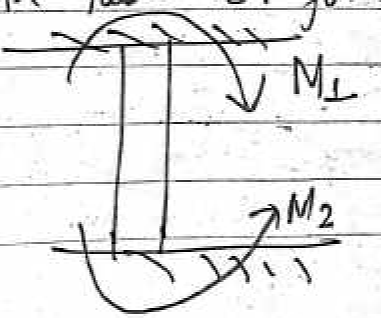
Gulshan

(*) Design a plate girder (web splice), column column under eccentric loading (9.1, cl. 9.3.2.2). Pg. 71.

n_y = x-direction load capacity.

$\lambda_y = \sqrt{\frac{fy}{fcc}}$ In λ_y use r_y. In λ_x use r_x.

Pg. no. 72 Table 18. for Cmy & cmx.



$M_1 \leq M_2$

$\cancel{\psi = \frac{M_2}{M_1}}$. $\psi = \frac{M_1}{M_2}$.

Cmfd is smaller of Cmy & Cmz.

~~$M_2 = \psi$~~

Cl. 9.3.1.3.

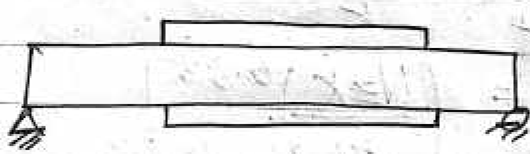
In case of weld we use -

(*) Curtailment of plate

V_{max}

~~when moment is max at sides but max~~

When shear force is max at sides but min. at middle then we curtail plate at middle.



as BM is max at middle in simply supported beam.