Syllabus:

1. Simple stress, strain.

   Simple stress = P/A.

   (i) Actual stress

   (ii) Normal stress.

2. Complex stress.
3. Principal stress and theories of failure
4. S.F.D and B.M.D.
5. Torsion.
6. Bending stress and shear stress.
7. Thin cylinders.

Simple stress and strain:

* ~~Stress~~ is a tensor quantity.

(IS 456 : 2000)

* Young's modulus of concrete = $5000\sqrt{f_{ck}}$

  Young's modulus of steel = 2×10^5 N/mm²

  Young's modulus, $E = \frac{\sigma}{\varepsilon}$

Poission's ratio $\mu = \frac{\text{Lateral strain}}{\text{Longitudinal strain}}$.

$\mu = 0 - 0.5.$

Bottle cork $\boxed{\mu = 0.}$

Creep:

Long term deflection due to substaine loading.

Fatigue:

Failure due to cyclic loading.

IS 1348 : 1999

$E_C = 5700\sqrt{f_{ck}}$

Strength of Materials:

The internal resistance offered by the body to sustain external loading without any deformation.

Stress:

* It is the resistance offered by the body per unit area
* This resistance is due to cohesion b/w the particles
* It's unit is N/mm^2 (or) MPa.

where

$$1\ Pa = 1\ N/m^2$$

* It is a tensor quantity.

Broad Classification:

* simple stress
* complex stress
* Bending stress.
* shear stress.

Simple stress:

Axial stress

→ Tensile

→ compressive.

Other classification:

* Engineering stress.
* True stress.

Engineering stress:

$$\sigma_{engg} = \frac{P}{A_0}$$

A_0 - original c/s Area

True stress:

$$\sigma_{true} = \frac{P}{A_i}$$

A_i - Instantaneous c/s Area

Relationship b/w True & Engg stress

$$\sigma_{true} = \frac{P}{A_i} \times \frac{A_o}{A_o}$$

$$\boxed{\sigma_{true} = \sigma_{eng} \times \frac{A_o}{A_i}}$$

Strain (ε):

It is the ratio b/w change in length and original length.

Dimension – $M^0 L^0 T^0$.

Hooke's Law:

It is given by "Robert Hook" and it states ~~takes~~ that stress is directly proportional to the strain within the elastic limit

$$f \propto \varepsilon$$

$$f = E\varepsilon$$

$$\boxed{E = \frac{f}{\varepsilon}}$$

$$f = E\varepsilon.$$

$$\frac{P}{A} = E \times \frac{\delta L}{L}.$$

$$\boxed{\delta L = \frac{PL}{AE}}$$

Poisson's Ratio:

It is the ratio b/w lateral strain and longitudinal strain.

It is denoted by μ (or) $\frac{1}{m}$ (or) ν.

Ex:

Material	μ
Concrete	0.2
Steel	0.3
Glass	0.~~3~~ 1.8
Cork	0
Rubber	0.5

* μ lies between 0 – 0.5.

Elasticity:

* It is defined as the property of material due to which the material regains it's original shape after removal of loads.

* The materials can be classified into Elastic, plastic and Rigid.

Elastic:

When an elastic material is subjects to external loading undergoes deformation such that deformation disappears after removal of loading.

Plastic:

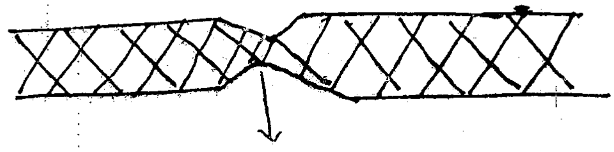
A plastic material undergoes continous deformation during the period of loading and deformation is permanent and the material does not regains its shape.

Rigid:

* A rigid material does not undergo a deformation when it is subjected to loading.

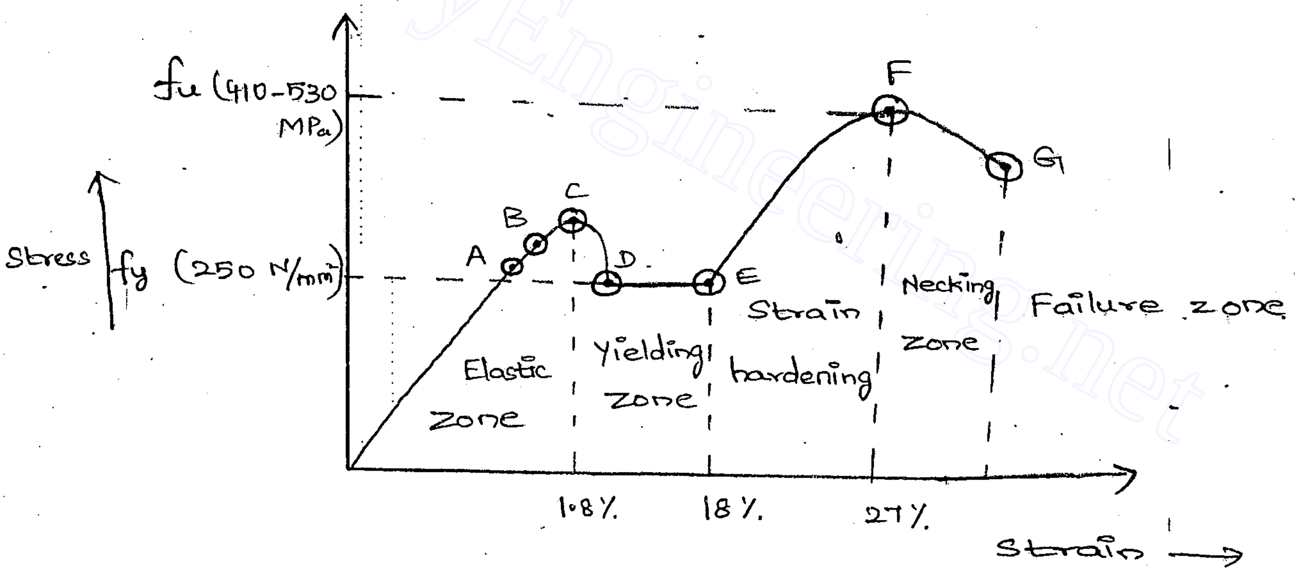
* Practically no material is purely plastic, elastic and rigid.

Stress - strain graph of Mild steel:



This type of failure is known as cup and cone fracture.

Stress - strain graph of Mild steel:



A — Limit of proportionality.

B — Elastic Limit

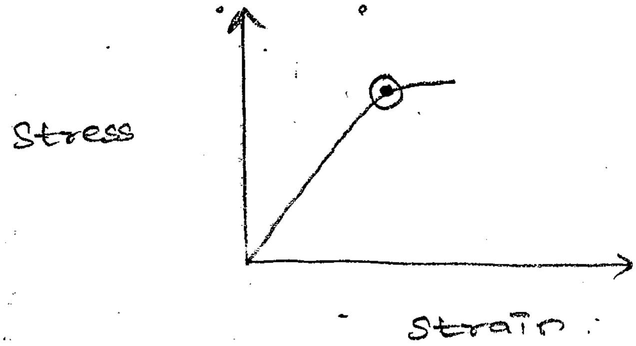
C — Upper yield point

D - E — Lower yield point

F — Ultimate yield point

G — Failure point.

Stress strain graph of glass.

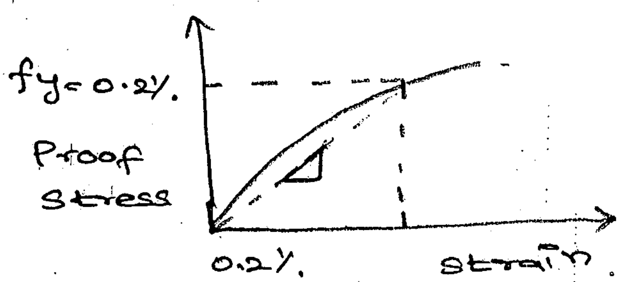


Proof - Stress:

When a material does not show any distinct yield point (Due to its nature) then 0.2% of proof strain is considered (a line drawn parallel to elastic line (or limit of proportionality of line) and the corresponding stress considered as the yield stress of a material and it is known as proof stress.

This behaviour is seen in Aluminium, cast iron and high strength steel.

Stress - strain graph of HYSD:



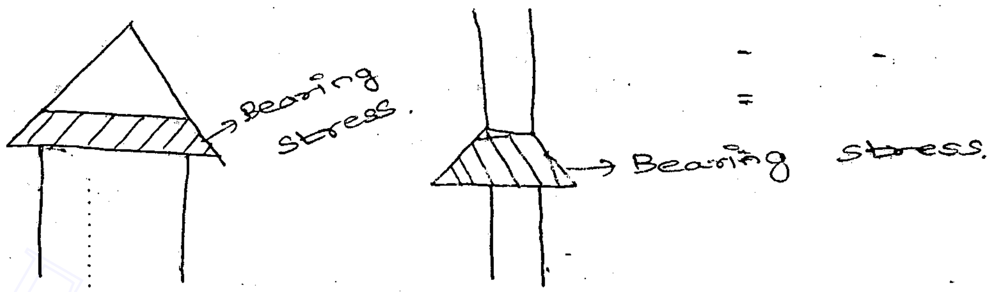
Proof strain.

Normal stresses classified into 3 :

* Axial stress
* Bearing stress
* Bending stress.

Bearing stress:

compressive stress arising when one body is supported by another is called bearing stress.



Bending stress:

Bending tensile stress and Bending compressive stress is produced when it is subjected to bending.

Shearing stress:

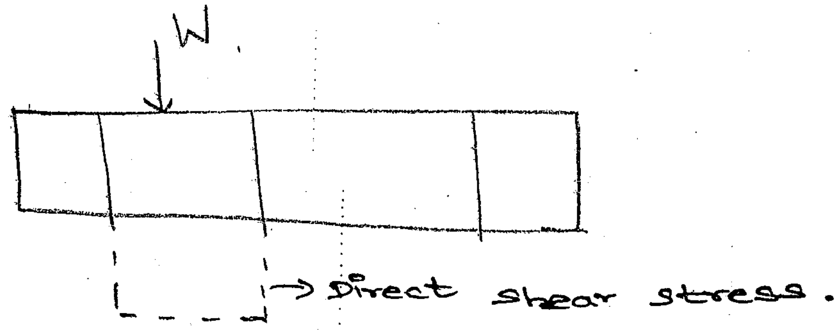
It is the stress acting in the plane of a section.

There are 2 kinds of shearing stress

* Direct shear stress.
* Indirect shear stress.

Direct shear stress:

Shear stress created due to direct action of forces in trying to cut through the materials.

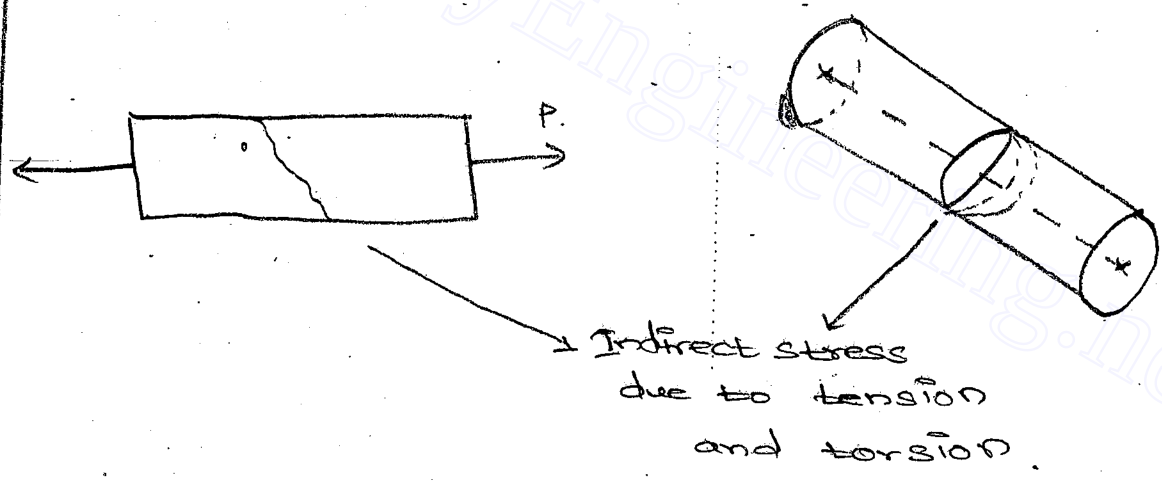


Indirect shear stress:

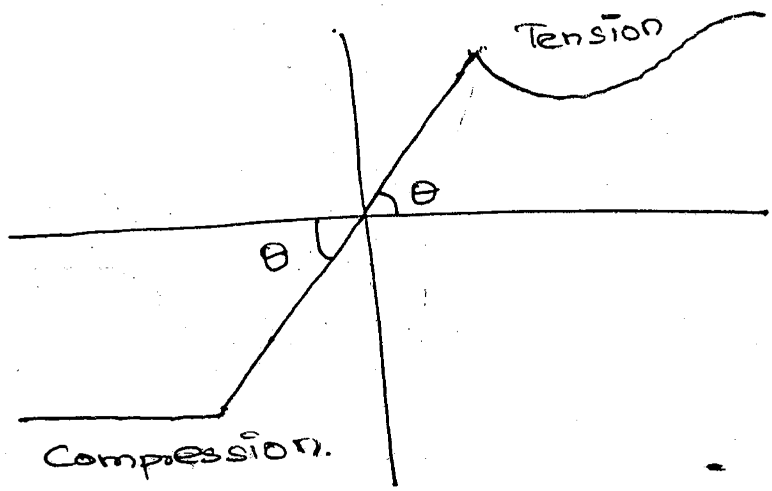
Indirect shear stress is due to.

(a) Tension (or) Compression.

(b) Torsion.



Stress - strain graph for Mild steel under compression:



If structural steel is subjected to compression instead of tension - the stress - strain curve will be same through its straight line portion and through begining of curve portion corresponding through yield and strain hardening. For large value of strain, stress strain curve will diverged.

In compression no - necking occurs.

Modulus of elasticity in compression } = Modulus of elast in tensic

3/7/2015

STRENGTH OF MATERIALS

Creep - Time dependent deformation under sustained loading

* Creep is an important parameter that affects or influences.
* Influencing factor m, E,

Fatigue -

* Rate of creep decrease with time because as ~~stress~~ increases strain hardening ~~takes~~ place
* It depends on Temperature level, stress level, ~~time~~, type of loading (static (or) dynamic)
* Generally effect of creep becomes noticable at approx. 30% of melting point for metals.

Fatigue:

* Deterioration of material due to repeated cycles ~~loading~~ of stress or strain resulting in progressive cracking that eventually produces fracture is called fatigue.

Plasticity:

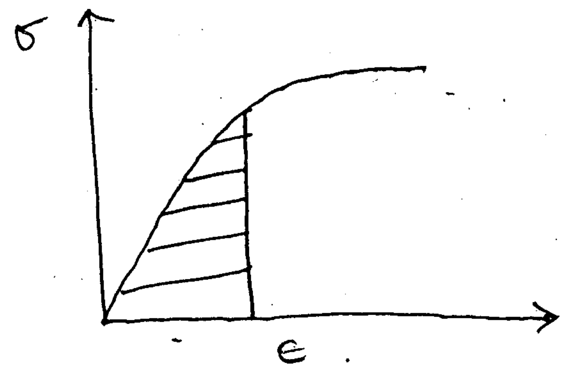
The characteristics of a material by which it undergoes inelastic strain beyond the strain at the elastic limit is known as plasticity

Resilience:

* It is the property of a material to absorb energy when it deformed elastically and then upon unloading to have this energy recovered.

Hence greater the resilience more desirable for spring action.

The area under stress strain curve within elastic limit is called Modulus of resiliance.



For a linearly elastic material strain energy stored per unit volum

$$= \frac{\sigma \times \epsilon}{2} \times \text{Volume} = \frac{\sigma \times \sigma}{2E}$$

$$= \frac{\sigma^2}{2E} \times V \text{ (Modulus of resilianc}$$

Area under load under deformation within elastic limit is called as resilience.

Endurance limit:

Endurance limit is the stress level below which even large no. of stress cycle cannot produce fatigue failure.

* For structural steel endurance limit = $\frac{1}{2}$ × ultimate strength

* For non-ferrous metal stress @ failure continous to decrease. hence we define fatigue limit as stress corresponding to failure after a specific no. of loading cycles

* Due to corrosion effect, endurance limit is reduced upto 50% of that under normal condition.

Relaxation:

* The decrease in stress in steel as a result of creep within steel under prolonged strain is called relaxation.

Stiffness:

* Stiffness may be defined as a ability of a material to with stand high load without major deformation

Deformable bodies:

~~Def~~ Undergoes deformation when external forces acted upon them.

Volumetric strain: $\left(\frac{dv}{v}\right)$

Ratio of change in volume to original volume.

$$\epsilon_v = \frac{dv}{v}$$

Factor Of Safety:

Ratio of ultimate tensile stress to the permissible stress is called factor of safety.

$$F.O.S = \frac{\text{Ultimate stress.}}{\text{Permissible stress.}}$$

There are 3 types of Elastic constant

E, G, K.

1.) Modulus of Elasticity (or) Young's Modulus. (E)

2.) Bulk Modulus. (K)

3.) Shear Modulus (or) Rigidity Modulus (or) Modulus of rigidity. (G)

Bulk modulus:

Bulk modulus of a substance measures the substances resistance to uniform compression.

It is defined as the ratio of direct stress to the resulting relative decrease of the volume.

$$K = \frac{\text{Direct stress}}{\text{Volumetric strain}}$$

Shear Modulus (or) Rigidity Modulus

[G, C, N]:

Ratio of shear stress to shear strain

$$C = \frac{\tau}{\phi}$$

Relationship b/w elastic constant.

$$\boxed{\begin{aligned} E &= 3K(1-2\mu) \\ E &= 2G(1+\mu) \end{aligned}}$$

* If $K = \mu$

$$E = 3\mu(1-2\mu)$$
$$= 3\mu - 6\mu^2$$
$$= 3\mu - 6\mu^2$$

* If $\cancel{k = G}$ $E = K$.

$$\cancel{E} = 3\cancel{G}\cancel{E}(1-2\mu)$$

$$\boxed{\mu = 1/3}$$

* If $E = G$

$$\cancel{G} = 2\cancel{G}(1+\mu)$$
$$= 2 + 2\mu.$$

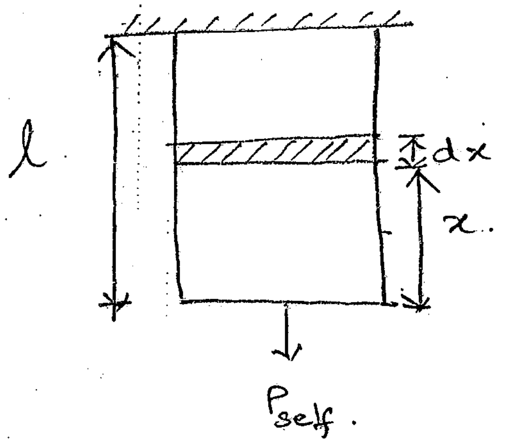
$$\boxed{\mu = -1/2}$$

* $$E = \frac{9KG}{3K+G}$$

12/8/2015

5

STRENGTH OF MATERIAL.



$$P_{self} = \gamma \times A \times l$$

$$\delta l_x = \frac{P_x \, l_x}{A_x \times E}$$

$$= \frac{\gamma \times A \times x \, dx}{AE}$$

$$= \int_0^l \frac{\gamma A x \, dx}{AE}$$

$$\delta l = \frac{\gamma A}{AE} \left[\frac{x^2}{2} \right]_0^l$$

$$= \frac{\gamma A \times l \times l}{2AE}$$

$$= \frac{(\gamma A l) l}{2AE}$$

$$\boxed{\delta l = \frac{W_{total} \times l}{2AE}}$$

$$\delta l = \frac{\left(\frac{0 + W_{total}}{2}\right) \times l}{AE}$$

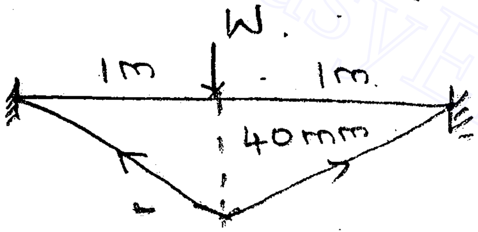
$$\boxed{\delta l = \frac{W_{total} \times l}{2AE}}$$

Note:

* Elongation of tappered circular dia rod D_1 = small dia D_2 = large dia due to load P.

$$\boxed{\delta l = \frac{Pl}{\frac{\pi}{4} D_1 D_2 E}}$$

i)



ϕ of wire = 1mm

length of deformed wire

$\delta l =$

$= \sqrt{1000^2 + 40^2}$

$= 1000.799$ mm

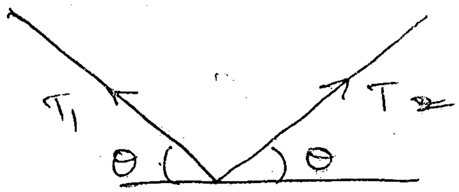
$\delta l = 1000.799 - 1000$

$$\boxed{\delta l = 0.799 \text{ mm}}$$

$$\delta l = \frac{Pl}{AE}$$

$$0.799 = \frac{P \times 1000}{\left(\frac{\pi \times 1^2}{4}\right) \times 2 \times 10^5}$$

$$\boxed{P = 125.61 \text{ N}}$$



$\theta = \tan^{-1}(40/1000)$

$\boxed{\theta = 2.29}$

$\Sigma H = 0$

$T_1 \cos\cancel{\theta} = T_2 \cos\cancel{\theta}$

$\boxed{T_1 = T_2}$

$\Sigma V = 0$

$T_1 \sin\theta + T_2 \sin\theta = W$

$W = 2(125.51 \sin 2.29)$

$\boxed{W = 10.03\ N}$

$\sigma = \delta . \epsilon E.$

$= \frac{\delta l}{l} \times E.$

$\boxed{\sigma = 159.8(}$ N

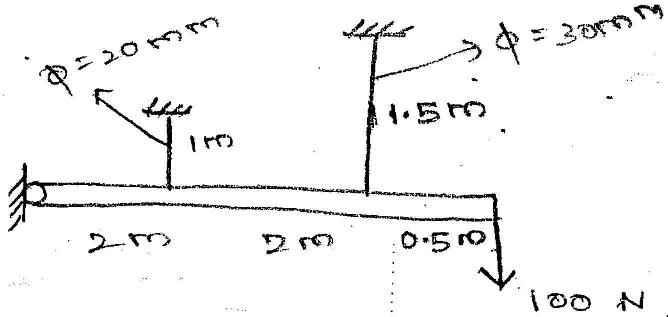
$T = \sigma \times A$

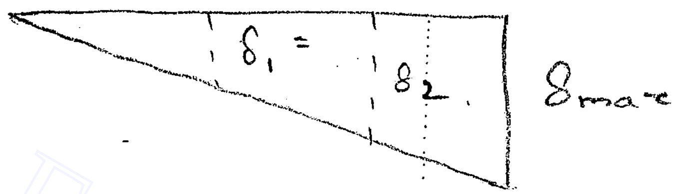
$= 159.808 \times \frac{\pi \times 1^2}{4}$

$\boxed{T = 125.51 k}$

2. A rigid beam shown in figure. Determine stresses developed in the rod supporting the rigid beam.

linear Vari





$\Sigma V = 0$.

$T_1 + T_2 + R_A = 100$ N —(1).

$\Sigma M_A = 0$.

$2T_1 + 4T_2 = 100 \times 4.5$

$2T_1 + 4T_2 = 450$ —(2)

By similar triangle property,

$\frac{\delta_1}{2} = \frac{\delta_2}{4}$.

$\boxed{\delta_2 = 2\delta_1}$

$\delta = \frac{Pl}{AE}$.

$2 \times \cancel{\delta} \frac{T_1 l_1}{A_1 E} = \frac{T_2 l_2}{A_2 E}$.

$\frac{2 \times T_1 \times 1000}{\frac{\pi \times 20^2}{4}} = \frac{T_2 \times 1500}{\frac{\pi \times 30^2}{4}}$.

$$T_2 = \frac{2 \times 1000}{20^2 \times 1500} \times 30^2 \, T_1$$

$$\boxed{T_2 = 3 T_1}$$

$$2T_1 + (4 \times 3) T_1 = 450.$$

$$14 T_1 = 450.$$

$$\boxed{T_1 = 32.14 \, N}$$

$$\boxed{T_2 = 96.43 \, N}$$

$$\sigma_1 = \frac{T_1}{A_1} = \frac{32.14}{\left(\frac{\pi \times 20^2}{4}\right)} = 0.102 \ N/m$$

$$\sigma_2 = \frac{T_2}{A_2} = \frac{96.43}{\left(\frac{\pi \times 30^2}{4}\right)} = 0.136 \ N/mm$$

$$R_A = 100 - 96.43 - 32.14.$$

$$= -28.57$$

$$\boxed{R_A = 28.57 \ N \ (\downarrow)}$$

CONCRETE STRUCTURES

→ Concrete Technology

* properties of concrete
* Basics of mix design.

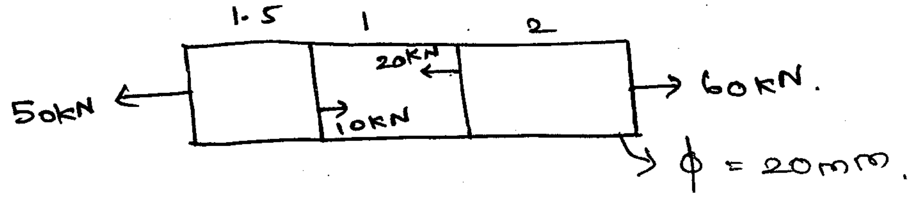
→ Concrete Design :

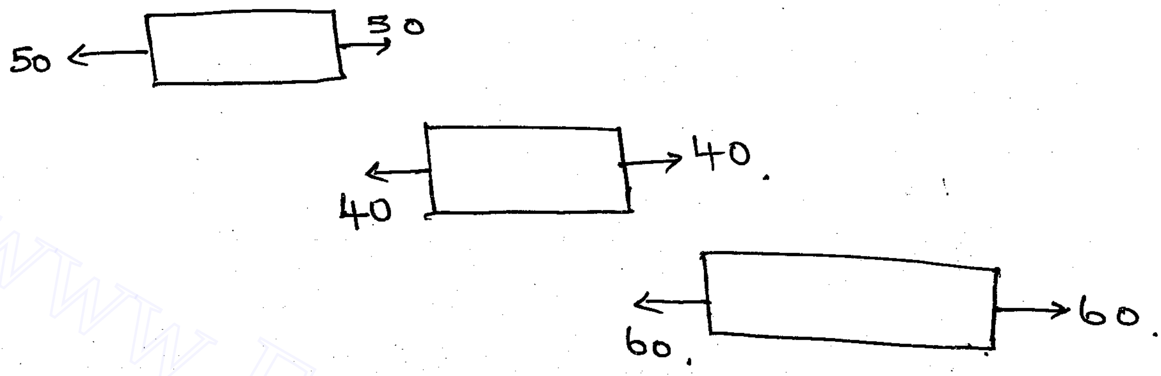
* Basic working stresses and limit state design concepts
* Analysis of ultimate load capacity
* Design of members subjected to flexure, shear, compression and torsion by limit state method.

→ Prestressed Concrete

* Basic elements of prestressed Concrete
* Analysis of beam sections at transfer and service load.

1. Determine the change in length of a loaded member





$$\delta l = \frac{P_1 l_1}{A_1 E_1} + \frac{P_2 l_2}{A_2 E_2} + \frac{P_3 l_3}{A_3 E_3}$$

$$= \frac{1}{AE}\left(50\times1.5 + 1\times40 + 60\times2\right)$$

$$\delta l = \frac{235\times10^3\times10^3}{AE}$$

$$= \frac{235\times10^6}{\left(\frac{\pi\times20^2}{4}\right)\times2\times10^5}$$

$$\boxed{\delta l = 3.74\ mm}$$

2. A metallic bar of $l = 200$ mm Axial tensile load = 160 kN. c/s = 40 × 40 mm. Elongation of bar is 0.2 mm. $\delta b = 0.005$ mm. Determine E, μ.

$$\delta l = \frac{Pl}{AE}.$$

$$E = \frac{Pl}{A\delta l} = \frac{160\times10^{3}\times200}{40\times40\times0.2}.$$

$$\boxed{E = 1\times10^{5}.}$$

$$\text{lateral strain} = \frac{0.005}{40} = 0.125\times10^{-3}$$

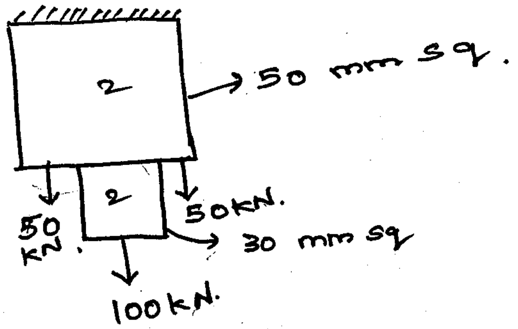
$$\text{longitudinal strain} = \frac{0.2}{200} = 1\times10^{-3}.$$

$$\mu = \frac{0.125\times10^{-3}}{1\times10^{-3}}.$$

$$\boxed{\mu = 0.125}$$

3/9/2015

1. Determine maximum stress developed in material as shown in.

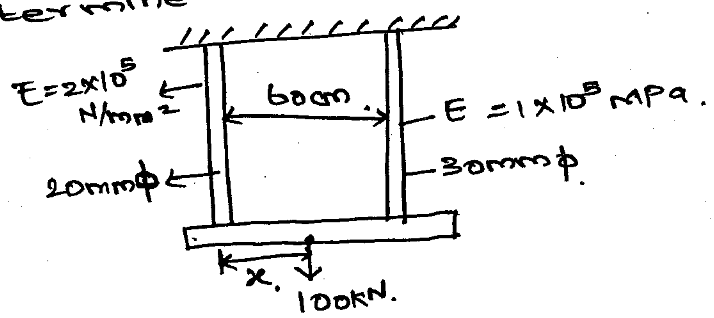


$$f_2 = \frac{P_2}{A_2} = \frac{100 \times 10^3}{30 \times 30} = 111.11 \ N/mm^2$$

$$f_1 = \frac{P_1}{A_1} = \frac{(50+50+100) \times 10^3}{50 \times 50} = 80 \ N/mm^2$$

∴ Maximum stress acting on the member is 80 N/mm².

2. 2 metallic bars are used to support a load as shown in figure. Determine the position of load such that the bottom supporting member remains horizontal. Also determine the stresses in the bars



$\Sigma V = 0$

$P_2 + P_1 = 100\ kN.$

$P_2 = 100 - P_1.$

$\Sigma M_{100kN} = 0.$

$P_1 \times x = P_2 \times (60 - x)$

$(100 - P_2)x = 60P_2 - P_2 x.$

$100x + P_2 x.$

By compatability equation.

$\delta l_1 = \delta l_2.$

$$\frac{P_1 \cancel{l_1}}{A_1 E_1} = \frac{P_2 \cancel{l_2}}{A_2 E_2}.$$

$$\frac{P_1}{\frac{\cancel{\pi} \times 20^2}{\cancel{4}} \times 2 \times \cancel{10^5}} = \frac{P_2}{\frac{\cancel{\pi} \times 30^2}{\cancel{4}} \times 1 \times \cancel{10^5}}$$

$$P_1 = P_2 \times \frac{20^2 \times 2}{30^2}.$$

$P_1 = 0.89 P_2.$

$(0.89 P_2) + P_2 = 100.$

$\boxed{P_2 = 52.94\ kN.}$

$\boxed{P_1 = 47.06\ kN.}$

$\Sigma M_A = 0.$

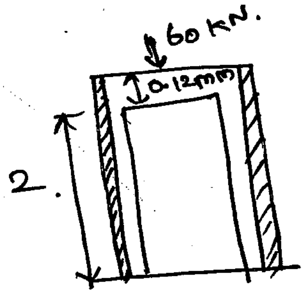
$(100 \times x) = (47.06 \times 60)$

$60.\cancel{x} = 28.235.$

$\boxed{x = 31.76\ cm.}$

$\sigma_1 = \frac{P_1}{A_1} = 149.8\ N/mm^2$ $\qquad \sigma_2 = 74.89\ N/mm^2$

3. A steel rod of 20mm dia is coaxially placed inside a brass ~~cylinder~~ of ~~inner dia 22m~~ $D_o = 30mm$. Brass is longer than steel by 0.12mm. the length of the steel is 2m. External coaxial compressive load is 60 kN. Determine the stresses developed in steel and brass: $E_s = 2\times10^5 N/mm^2$, $E_b = 1\times10^5 N/mm^2$



$$\delta l = \frac{P_b l}{AE}$$

$$P_b = \frac{0.12 \times \pi \times (30^2 - 20^2) \times 1\times10^5}{4 \times 2000}$$

$$\boxed{P_b = 1.96 \text{ kN}}$$

$$P_{total} = 60 - 1.96 = 58.04 \text{ kN}$$

$$P_b' + P_s = 58.04 \times 10^3$$

~~By~~ By compatability condition,

$$\delta l_{Pb'} = \delta l_{Ps}$$

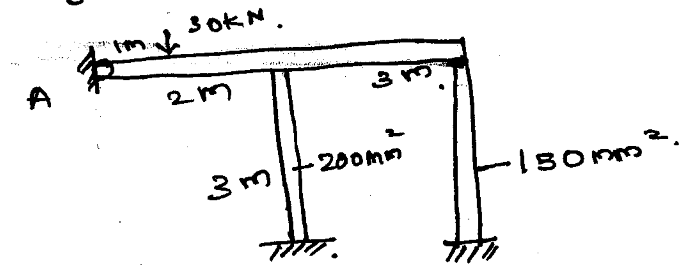
$$\frac{P_b' \cancel{l}}{A_b E_b} = \frac{P_s \cancel{l}}{A_s E_s}$$

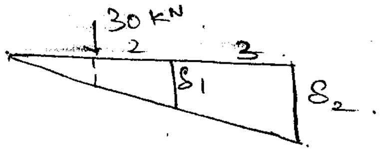
$$\frac{P_b'}{\frac{\pi}{\cancel{4}}(30^2 - 20^2) \times 1 \times 10^5} = \frac{P_s}{\frac{\cancel{\pi} \times 20^2}{\cancel{4}} \times 2 \times 10^5}$$

$$P_b' = \frac{P_s \times (30^2 - 20^2)}{20^2 \times 2}$$

$$\boxed{P_b' = 0.625 P_s}$$

4. A rigid beam as shown in figure.





$\Sigma M_A = 0$.

$30 \times 1 = (R_B \times 2) + (R_C \times 5)$

$\Sigma V = 0$.

$R_A + R_B + R_C = 30 \text{ kN}$ —(1).

$$\frac{\delta_1}{2} = \frac{\delta_2}{5}$$

$$\boxed{\delta_1 = \frac{2\delta_2}{5}}$$

$$\frac{P_1 l_1}{A_1 \cancel{E}} = \frac{2}{5} \frac{P_2 l_2}{A_2 \cancel{E}}$$

$$\frac{R_B \cancel{3}}{200} = \frac{2}{5} \times \frac{R_C \times \cancel{3}}{150}$$

$$R_B = \frac{2 \times 200}{5 \times 150} \times R_C$$

$$\boxed{R_B = 0.533 R_C}$$

$$(0.533 R_C \times 2) + 5 R_C = 30$$

$$\boxed{R_C = 4.945 \text{ kN}}$$

$$\boxed{R_B = 2.6357 \text{ kN}}$$

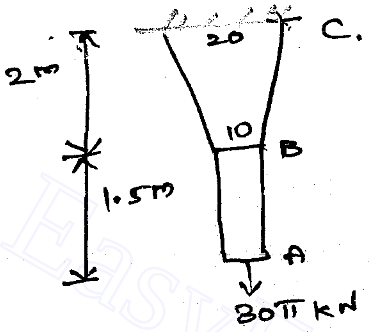
$$R_c = 30 - 4.945 - 2.6357$$

$$\boxed{R_A = 22.419 \text{ kN}}$$

1/9/2015

S.O.M.

1. A tappered circular rod dia varying from 20mm to 10mm. is connected to another circular rod 10mm as shown in figure both the bars are made of same material where $E = 2\times10^5$ MPa. when subjected to a load of 30π kN, the deflection @ point A is ____ mm.



$$\delta l = \delta l_1 + \delta_2.$$

$$= \frac{Pl}{\frac{\pi}{4}\times D_1 \times D_2 \times E} + \frac{Pl}{\frac{\pi}{4}D^2 \times E}.$$

$$= \frac{30\pi\times10^3}{2\times10^5}\left[\frac{4\times2000}{\pi\times20\times10} + \frac{1500\times4}{\pi\times10^2}\right].$$

$$\boxed{\delta l = 15\text{ mm.}}$$

2. For a beam of c/s width = 230mm d = 500mm. the no. of. rebars of 12mm ϕ required to satisfy minimum tension reinforcement as per IS 456 : 2000. Assuming Fe 500 is ____.

$$A_{st} = \frac{0.85 bd}{f_y}$$

$$= \frac{0.85 \times 230 \times 500}{500}$$

$$= 195.5 \text{ mm}^2$$

$$n = \frac{195.5}{\frac{\pi \times 12^2}{4}} = 1.72$$

$$n \simeq 2.$$

3. In a R.C.C section the stress @ the fiber in compression is 5.8MPa. x_a = 58mm. M_{25} Assuming linear elastic behaviour of concrete the effective curvature of the section is ____ /m

σ_{cbc} = 5.8MPa x_a = 58mm. f_{ck} = 25.

Bending equation

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}.$$

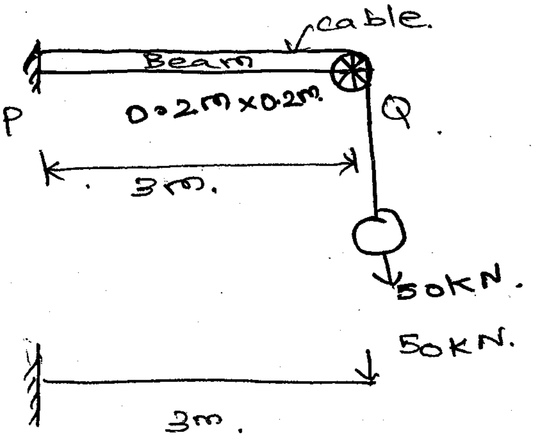
$$\frac{\sigma}{y} = \frac{E}{R}.$$

Radius of curvature $\frac{1}{R} = \frac{\sigma}{y \times E} = \frac{5.8}{58 \times 5000 \times \sqrt{25}}$

$$\boxed{\frac{1}{R} = 4 \times 10^{-6} \text{ /mm}}$$

4\. The values of axial stress in kN/m^2.

B.M in kN.m . S.F in kN. acting at point P. for the arrangement as shown in figure respectively.

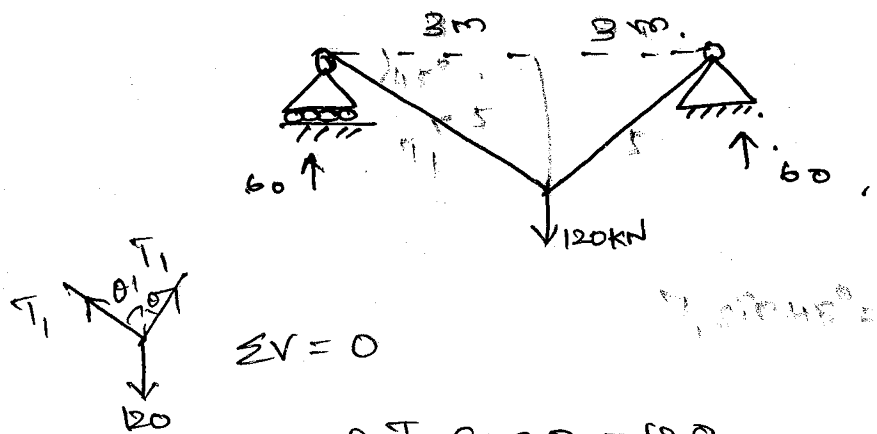


B.M = 50x3 = 150 kN.m.

$$\text{Axial stress} = \frac{\cancel{1.25}\,50}{0.2\times0.2} = 1250\ kN/m^2.$$

Axial force = 50kN.

5\. The tension in 10m long cable is shown in figure neglecting its self weight is.



$\Sigma V = 0$

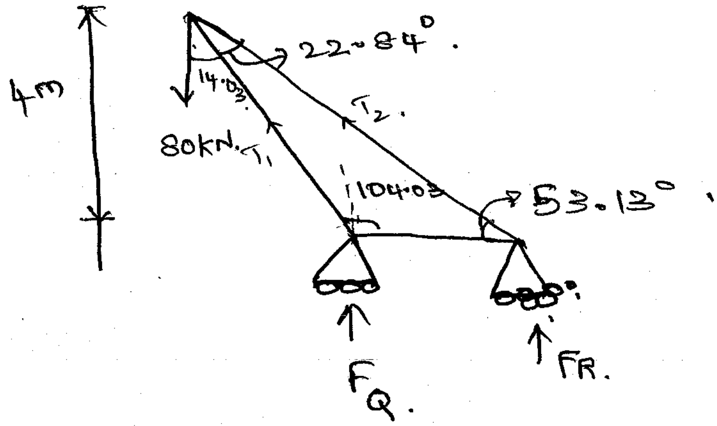
$2T_1 \cos\theta = 120$

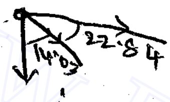
$2T_1 \left(\frac{4}{5}\right) = 120.$

$$T_1 = \frac{120\times5}{2\times4}$$

$$\boxed{T_1 = 75kN}$$

6. Mathematical idealization of crane has 3 bars ~~with vertices as shown in~~ figure with a load of 80kN hanging vertically the co-ordinates of the vertices are given in the diagram. The forces in PQ.





$\Sigma H = 0$

~~cos~~ $T_2 \sin 36.87 = T_1 \sin 14.03$

$$\boxed{T_2 = -0.404 T_1}$$

$\Sigma V = 0$

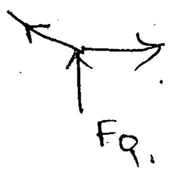
$80 = T_2 \cos 36.87 + T_1 \cos 14.03$

$= -0.404 T_1 \cos 36.887 + T_1 \cos 14.03$

$= -0.326 T_1 + 0.97 T_1$

$80 = T_1 \, 0.644$

$T_1 = 124.19 \text{ kN}$



$F_{QR} = T_1 \sin 14.03$

$$\boxed{F_{QR} = 30.107 \text{ kN}}$$

Euler column Load, $P_E = \frac{\pi^2 EI}{l_{eff}^2}$

<u>Thermal Stresses:</u>

change in length, $\delta l = l \alpha \Delta T$.

Thermal stress $\sigma_{thermal} = \varepsilon E$

$= \frac{\delta l}{l} \times E$

$= \frac{l \alpha \Delta T}{l} \times E$

$$\boxed{\sigma_{Thermal} = \alpha \Delta T E}$$

Compression member $\delta l = (l_c \times \Delta t) -$ net Elongation

Tension member $\delta l =$ net elongation $- (l_c \times \Delta t)$

23/9/2015

Thermal Stress:

1. Two copper plates and one steel plate are rigidly connected to each other and they are heated to a temperature to 315°C. The room temperature is 15°C. ~~Net~~ ~~the~~ elongation of assembly is 2mm. ~~of the~~ Determine the original length ~~of the~~ ~~the~~ material and stress developed in the material $\alpha_s = 12\times10^{-6}/°C$, $\alpha_c = 17.5\times10^{-6}/°C$

$E_s = 2\times10^5$ N/mm $\quad E_c = 1\times10^5$ N/mm².

~~Using Compa~~

Solution:

$$\delta l_c = l_c\,\alpha\,\Delta t - 2 \quad (\text{Compression})$$

$$\delta l_s = 2 - l_s\,\alpha\,\Delta t \quad (\text{tension})$$

Stress in steel

$$f_{steel} = \frac{\delta l_s}{l} \times E_s$$

$\in \times E_s$

$$= \frac{[2-(l_s\,\alpha\,\Delta T)]\times E_s}{l}$$

Stress in copper

$$f_{copper} = \frac{\delta l_c}{l} \times E_c$$

$$= \frac{(l_c\,\alpha\,\Delta T - 2)\times E_c}{l}$$

$$F_{steel} = F_{copper}$$

$$\frac{[2-(l_s\,\alpha\,\Delta T)]\times E_s \times \cancel{A}}{\cancel{l}} = \frac{(l_c\,\alpha\,\Delta T - 2)\times E_c \times 2\cancel{A}}{\cancel{l}}$$

$$2-(l\times 12\times10^{-6}\times300)\times \cancel{2}\times\cancel{10^5} = (l\times17.5\times10^{-6}\times300-2)\times \cancel{E_c}$$

$$4 = l\left[(12\times10^{-6}\times300)+(17.5\times10^{-6}\times300)\right]$$

$$\boxed{l = 451.98\ \text{mm}}$$

$\delta l = \alpha \Delta T l$

$2 = 12 \times 10^{-6} \times 300 \times$

* when two different bars are connected rigidly and have ~~two~~ different α value such that $\alpha_1 > \alpha_2$,

→ then material 1 is under compression

$\delta l_c = l\alpha_1 \Delta T$ - Net elongation

Material 2 is under Tension

δl_t = Net elongation $- l\alpha_2 \Delta T$

* when there is no yielding of support.

$\delta l = l \alpha \Delta T$

* when there is yielding of support

$\delta l = $ (yielding length) $=$

$$\delta l = \left[\frac{l\alpha \Delta T - \text{yielding length}}{l} \right]$$

* EI - Flexural rigidity
  AI - Axial rigidity
  GA - Shear rigidity
  GJ - Torsional rigidity

* $\frac{EI}{L}$ - Flexural stiffness.
  $\frac{AI}{L}$ - Axial stiffness
  $\frac{GA}{L}$ - Shear stiffness
  $\frac{GJ}{L}$ - Torsional stiffness.

$$f_{steel} = \frac{\left(2 - \left(451.8 \times 12\times10^{-6} \times 300\right)\right) \times 2\times10^5}{451.8}$$

$$= 165.35\ N/mm^2 \quad \text{(Tensile force)}$$

$$f_{copper} = \frac{\left(451.8 \times \cancel{18}\,17.5\times10^{-6} \times 300 - 2\right)\times 10^5}{451.8}$$

$$= +82.49\ N/mm^2. \quad \text{(compressive)}$$

2.) Determin $f_{thermal}$ of steel rod $l = 6m$ fixed @ the end:

T rises from 20°C – 120°C

(i) No yielding of support

(ii) 1mm yielding of support.

Solution:

(i) No yielding of support.

$$\sigma_{thermal} = \alpha \Delta T E.$$

$$= 12\times10^{-6} \times 2\times10^5 \times 100$$

$$= 240\ N/mm^2$$

(ii) Yielding of support by 1mm

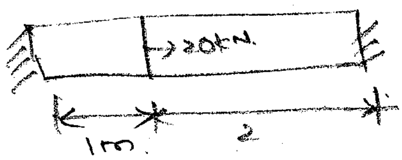
$$\sigma_{thermal} = \frac{\delta l}{l} \times E$$

$$= \frac{(l\alpha\Delta T - 1)}{l} \times E$$

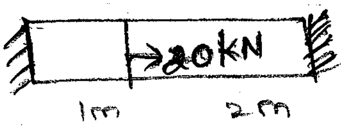
$$= \frac{\left[\left(6000\times 12\times10^{-6} \times 100\right) - 1\right] \times 2\times10^5}{6000}$$

$$= 206.67\ N/mm^2.$$

* stress will be more in non yielding when compared to stress induced due to yielding.



3.) Determine support reaction.



$\Sigma H_A = 0$.

$R_A + R_B = 20$

$\delta l_1 - \delta l_2 = 0$ (Due to ends are fixed)

$\delta l_1 = \delta l_2$.

$\frac{R_A l_1}{\cancel{AE}} = \frac{R_B l_2}{\cancel{AE}}$

$\boxed{R_A \times 1 = 2R_B.}$

$2R_B + R_B = 20$

$R_B = \frac{20}{3} = 6.67\,kN$.

$R_A = 20 - 6.67$

$\boxed{R_A = 13.33\,kN}$

4. A steel plate 10mm to be drilled to make a hole of 20mm ϕ Ultimate shear Stress 300 MPa. Determine Compressive stress required to make the hole.

solution:

Load = Shear stress $\times$ Area.

$= 300 \times \pi \times \frac{20}{4} \times 10$ $\pi \times d \times t$.

Load = 94.25 kN.

Load = 188.495 kN

Compressive stress $= \frac{188.495 \times 10^3}{\pi \times \frac{20^2}{4}}$ $\frac{\pi}{4} \times d^2$

Compressive stress = 300 N/mm²

5. A metal bar of $l = 100mm$ ~~inserted~~ b/w 2 rigid support and its ~~tem~~ $\Delta T = 10°C$. If $\alpha = 12\times10^{-6}/°C$. $E = 2\times10^5$ N/mm². what is the stresses in ~~th~~ the bar.

$$f_{thermal} = \alpha \Delta T E$$

$$= 12\times10^{-6}\times 10\times 2\times10^5$$

$$= 24 \text{ N/mm}^2$$

6. A mild steel specimen is under uniaxial tensile stress $E = 2\times10^5$ N/mm² $f_y = 250$ N/mm². Max. amount of strain energy / unit volume that can be stored in the material is.

$$\frac{\text{Strain energy}}{\text{unit volume}} = \frac{\sigma^2}{2E} = \frac{250^2}{2\times2\times10^5}$$

$$= 0.15625 \text{ Nmm/mm}^3$$

7. 2 steel column P and Q (length l, and $2l$), Fe 250 and 500 respectively have the same c/s and end condition the ratio of buckling load of the P to that colum load Q is $\underline{\underline{4}}$

0.5, 1, 2,

$$\frac{P_P = \frac{\pi^2 E I}{l_{eff}^2}}{P_Q = \frac{\pi^2 E I}{l_{eff}^2}} = \frac{\frac{1}{l^2}}{\frac{1}{(2l)^2}} = \underline{\underline{4}}$$

THERMAL STRESS:

α - co-efficient of thermal expansi

~~for~~ eg : for steel $\alpha_{steel} = 12 \times 10^{-6}$ m/m/c°

change in length

For 1mm in length and 1° rise in temperatu

$$\delta l = \alpha$$

For 1mm length Δt rise in temperature

$$\delta l = \alpha \Delta t$$

For $l$mm length Δt rise in temperatur

$$\boxed{\delta l = l \alpha \Delta t}$$

Thermal stress:

$$f_{thermal} = \frac{\delta l}{l} \times E = \frac{\cancel{l}\alpha\Delta t}{\cancel{l}} \times E$$

$$\boxed{f_{thermal} = \alpha \Delta t E}$$

Note:

when compound bars are used materials with higher α will try to expand untill the free expansion limit, but the material with lower alpha will not allow the higher α material till free expansion.

It will pull back the higher α material. As a result, material with higher α subjected compression.

Material will lower alpha will expand more than its free expansion since higher α material will pull it

As a result material with ~~higher~~ lower α subjected to tension.

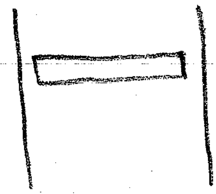
15/9/2015

S.O.M

1. Brass rod 2.4m long is placed b/w
1r' rigid walls, 2.43m apart. As shown in figure the temperature of rod is raised untill the rod is fixed b/w the wall and has a compressive stress 210 kg/cm². The rod is restrained from bending. what is the rise in temperation.

$E = 1.05 \times 10^{6}$ kg/cm². $\alpha = 11.8 \times 10^{-6}$ /°c.

solution:



$$\delta l = l \alpha \Delta T$$

$$0.03 = 2.4 \times 11.8 \times 10^{-6} \times \Delta T$$

$$\Delta T = 1059.32 \ ^{\circ}C.$$

$$\text{compressive stress} = \varepsilon E$$

$$21 = \cancel{\frac{\delta l}{l}} \times E \times \varepsilon.$$

$$\cancel{\delta l = 21 \times 2.4}. \qquad \varepsilon = \frac{21}{1.05 \times 10^{5}}.$$

$$\boxed{\varepsilon = 2 \times 10^{-4} m}$$

$$\varepsilon = \alpha \Delta T$$

$$\Delta T = \frac{2 \times 10^{-4}}{11.8 \times 10^{-6}} = 16.949 \ ^{\circ}C.$$

Total change in temp = 1059.32 + 16.949.

= 1076.269 °c.

2. A rod is composed of 3 segments as shown ~~in figure. The rod~~ is held b/w rigid supports. find the stress developed in each material when the ~~tem~~ $\Delta T = 50^\circ C$. under following 2 different condition

a) when the supports are perfectly rigid.

(b) when the right hand support yield by 0.2 mm

$E_s = 2\times10^6$ kg/cm²

$E_c = 1\times10^6$ kg/cm³.

$E_{al} = 0.7\times10^6$ kg/cm².

$\alpha_{steel} = 1.2\times10^{-5}$ /°c.

$\alpha_c = 1.8\times10^{-5}$ /°c

$\alpha_{alum} = 2.4\times10^{-5}$/°c

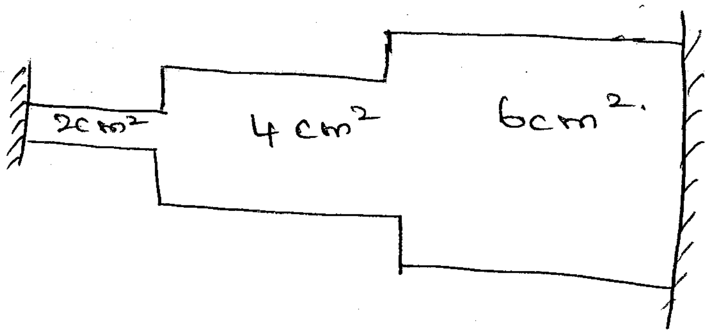
a) stress = $f_1 + f_2 + f_3$.

$$= \alpha_1 \Delta T E_1 + \alpha_2 E_2 \Delta T + \alpha_3 E_3 \times \Delta T.$$

$$= \left[\left[(1.2\times10^{-5} \times 2\times10^5) + (1.8\times10^{-5} \times 1\times10^5) + (2.4\times10^{-5} \times 0.7\times10^5)\right]\right] \times 5$$

$$= (2.4 + 1.8 + 1.68) \times 50$$

$$= 294 \text{ N/mm}^2.$$



$$\delta l = \delta l_1 + \delta l_2 + \delta l_3 .$$

$$= \left[\left(0.15 \times 1.2 \times 10^{-5} \times 2 \times \cancel{10^5}\right) + \left(0.2 \times 1.8 \times 10^{-5} \times 1 \times \cancel{10^5}\right) + \left(0.15 \times 2.4 \times 10^{-5} \times 0.7 \times \cancel{10^5}\right) \right] \times 50$$

$$\boxed{\delta l = 0.045}$$

$$f_{st} = f_{cu} = F_{Al}.$$

$$\frac{f_{st}}{2} = \frac{f_c}{4} = \frac{f_{Al}}{6}.$$

$$\boxed{f_{st} = \frac{f_c}{2}} \qquad \boxed{f_{Al} = \frac{3}{2} f_c}$$

$$\delta l = \frac{f_s}{E_s} l_s + \frac{f_c}{E_c} l_c + \frac{f_{Al}}{E_{Al}} l_{Al}.$$

$$0.045 = \frac{f_c \times 0.15}{2 \times 2 \times 10^5} + \frac{f_c \times 0.2}{1 \times 10^5} + \frac{3 f_c}{0.7 \times 10^5 \times 2}.$$

$$= f_c \{$$

$$f_c = 913.1 \ kg/cm^2$$

$$f_{st} = \cancel{\frac{913.1}{2}} \qquad f_{st} = \cancel{913.1} \ 1826.2 \ kg/cm^2.$$

$$f_{Al} = 608.7 \ kg/cm^2.$$

Case (b) :

$$\delta l - 2 = \frac{f_s}{E_s} \times l_s + \frac{f_{ca} \times l_{ca}}{E_c} + \frac{f_{Al} \times l_{Al}}{E_{al}}$$

$f_s = 1014.4 \text{ kg/cm}^2$

$f_c = 507.2 \text{ kg/cm}^2$.

$f_{Al} = 338.1 \text{ kg/cm}^2$.

3. A surveyor steel tape nominally 30m. is 1.25 wide and 1mm thick. It's length is correct when used @ a temperature of 16°C. and under a pull of 10kg. By how much it will be in error when used at a temperature 50°C under a pull of 5kg. $E = 2\times10^6 \text{ kg/cm}^2$

$\alpha = 11\times10^{-6} /°C$.

30m Wd = 1.25mm
t = 1mm
16°c
P = 10kg

$$\delta l = l \alpha \Delta T$$

$$= 30\times11\times10^{-6}\times34.$$

$$\delta l = 0.0112 \text{ m}.$$

$$= 1.12 \text{ cm}$$

$\frac{5}{1.25\times0.1} = \varepsilon E$

$\frac{\text{load}}{\text{Area}} = \sigma \Rightarrow \frac{P}{A} = \varepsilon E$; Strain × Young's Modulus

@ 50°
P = 5kg

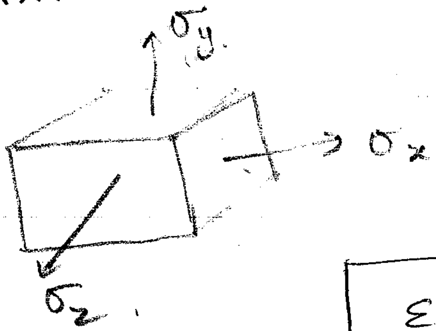
$$\frac{\delta l}{l} = \frac{5}{1.25\times0.1\times2\times10^6 \text{ kg/cm}^2}$$

$$\delta l = \frac{5 \text{ kg} \times 30\times100 \text{ cm}}{1.25\times0.1 \text{ sq cm}\times2\times10^6 \text{ kg/cm}^2}$$

$$= 0.06 \text{ cm}.$$

Error in length = 1.06 cm. = (1.12 − 0.06)

ELASTIC CONSTANT :



$$\varepsilon_v = \varepsilon_x + \varepsilon_y + \varepsilon_z$$

$$= \frac{\sigma_x}{E} + \frac{\sigma_y}{E} + \frac{\sigma_z}{E} - 2\mu\left[\frac{\sigma_x+\sigma_y+\sigma_z}{E}\right]$$

$$\varepsilon_x = \frac{\sigma_x}{E} - \frac{\mu}{\cancel{E}}\left[\frac{\sigma_y}{E}\right] - \mu\left(\frac{\sigma_z}{E}\right)$$

$$\varepsilon_y = \frac{\sigma_y}{E} - \frac{\mu\sigma_z}{E} - \frac{\mu\sigma}{E}$$

$$\varepsilon_z = \frac{\sigma_z}{E} - \frac{\mu\sigma_x}{E} - \ldots$$

$$\varepsilon_v = \frac{\sigma_x+\sigma_y+\sigma_z}{E}(1-2\mu)$$

$$E = \frac{\sigma_x+\sigma_y+\sigma_z}{\varepsilon_v}(1-2\mu)$$

If $\sigma_x = \sigma_y = \sigma_z = \sigma$.

$$\frac{\sigma}{\varepsilon_v} = K$$

$$E = \underbrace{\frac{3\sigma}{\varepsilon_v}}_{k}(1-2\mu)$$

$$\boxed{E = 3K(1-2\mu)}$$

$$\boxed{E = 2G(1+\mu)}$$

$$E = 3K(1-2\mu) \Rightarrow \mu = -\left(\frac{E}{6K} - \frac{1}{2}\right)$$

$$E = 2G\left(1 - \frac{E}{6K} + \frac{1}{2}\right)$$

$$E = \frac{2G}{\cancel{2}}\left(\frac{3}{2} + \frac{E}{6K}\right)$$

$$= 2G\left(\frac{9K+E}{6K}\right)$$

$$6KE = 18KG + 2GE \Rightarrow \boxed{E = \frac{9KG}{3K+G}}$$

EI - flexural rigidity. $\frac{N}{mm^2} \times mm^4 = Nmm^2$.

GA - Shear rigidity. $\frac{kgm}{s^2} \times m^2$

GJ - Torsional rigidity. ML^3T^{-2}

AE - Axial rigidity.

$\frac{EI}{L}$ - flexural stiffness.

$\frac{GA}{L}$ - Shear stiffness.

$\frac{GJ}{L}$ - Torsional stiffness

$\frac{AE}{L}$ - Axial stiffness.

Effect due to Impact Load = 2X static load.

21/10/2015

STRENGTH OF MATERIALS

Shear Force Diagram & Bending Moment Diagram

* Beam is a structural member subjected to load ⊥[r] to the axis of the member. and it ~~transferes~~ the load to the support through bending only.

* Beam is a bending member which is designed on the basis of max. B.M and max S.F.

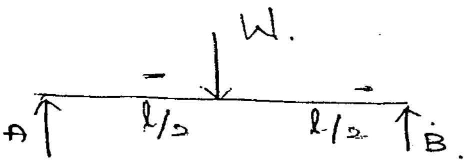
* The algebaric sum of all moment considered from extreme end to any section of beam is called B.M @ that section.

* The graphical representation of B.M along with its nature is called B.M diagram. which is very essential to find out the position of steel bars in R.C ~~Sect~~ structures.

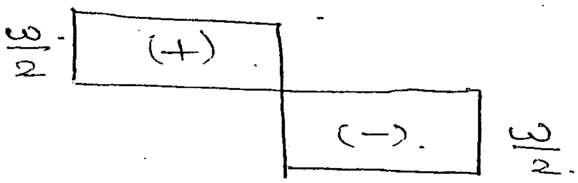
* The algebaric sum of all forces considered from extreme end to any section of beam is called S.F at that section.

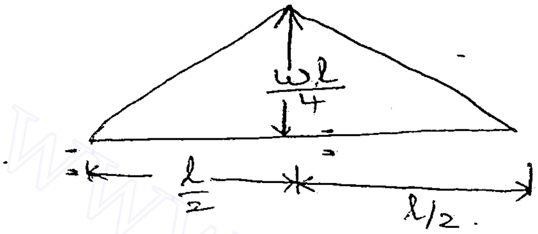
Standard Cases :

1.)



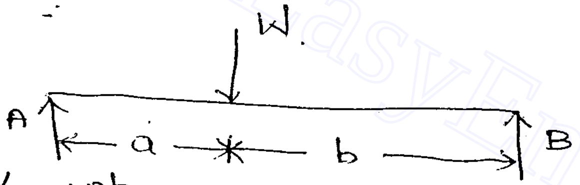
$V_A = V_B = \frac{W}{2}$.





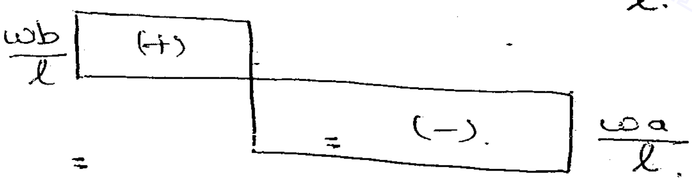
$\frac{wl}{4}$

2.)

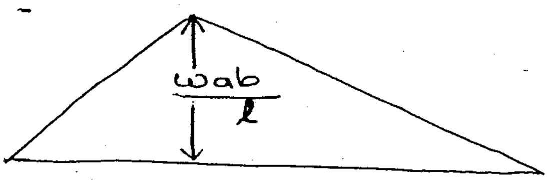


$V_A = \frac{wb}{l}$ $\qquad$ $V_B = \frac{wa}{l}$.

$\frac{wb}{l}$

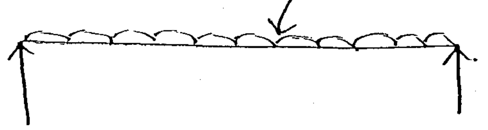


$\frac{wa}{l}$



$\frac{wab}{l}$

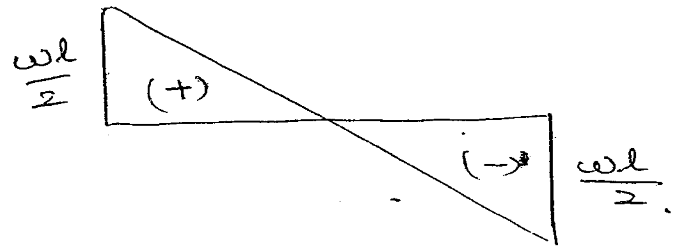
3.)

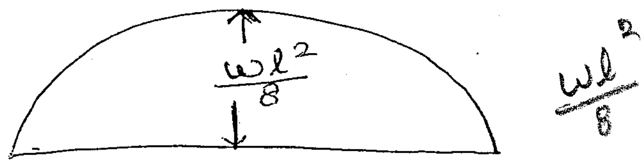


$V_A = \frac{\omega l}{2}$ $\qquad V_B = \frac{\omega l}{2}$.

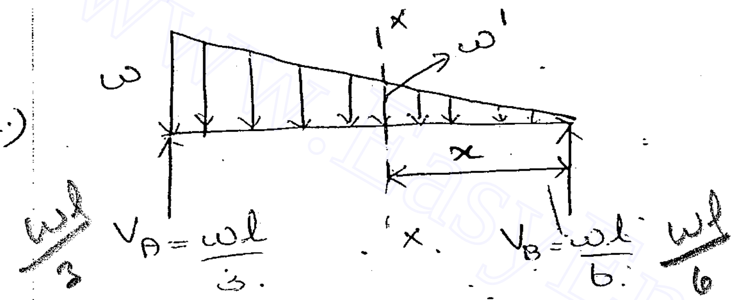
Load intensity = Area of loadin Diagram

$W = \omega \times l$.





4.)



$M_A = 0$.

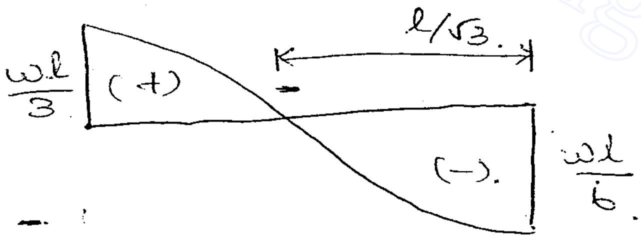
$R_B \times l = \frac{1}{2} \times \omega \times l \times \frac{l}{3}$

$\boxed{R_B = \frac{\omega l}{6}}$

$R_A = \frac{\omega l}{2} - \frac{\omega l}{6}$

$R_A = \frac{2\omega l}{6} = \frac{\omega l}{3}$

$\boxed{R_A = \frac{\omega l}{3}}$



S.F = 0

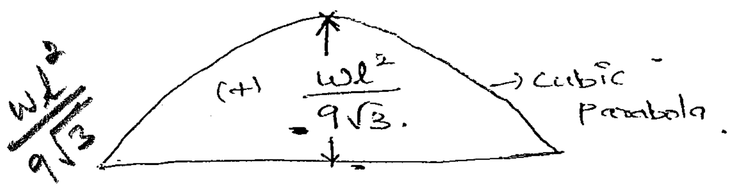
$Y_{xx} = 0$.

$R_B - \frac{1}{2} \times \omega' \times x = 0$.

By similar triangle property

$\frac{\omega'}{\omega} = \frac{x}{l}$.

$\boxed{\omega' = \frac{\omega x}{l}}$



$\frac{\omega l}{6} = \frac{1}{2} \frac{\omega x}{l} \times x$.

$\frac{2\omega l^2}{6} = \omega x^2$.

$\boxed{x = \frac{l}{\sqrt{3}}}$ S.F = 0.

B.M @ $\boxed{x = \frac{l}{\sqrt{3}}}$

$= R_B \times \frac{l}{\sqrt{3}} - \frac{1}{2} \times \omega' \times \frac{l}{\sqrt{3}} \times \left(\frac{l}{\sqrt{3}}\right) \times \frac{1}{3}$.

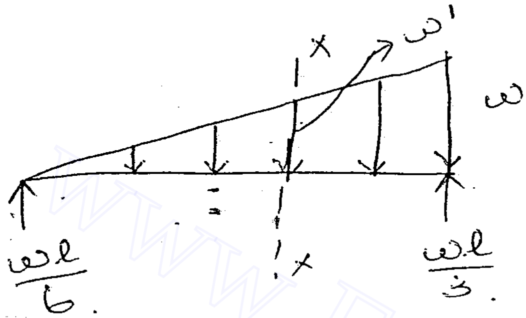
$= \left(\frac{\omega l}{6} \times \frac{l}{\sqrt{3}}\right) - \frac{1}{2} \cdot \frac{\omega \times l}{l\sqrt{3}} \times \frac{l}{\sqrt{3}} \times \frac{l}{3\sqrt{3}}$

$= \frac{\omega l^2}{6\sqrt{3}} - \frac{\omega l^2}{18\sqrt{3}}$.

$$= \frac{1}{\sqrt{3}}\left(\frac{wl^2}{6} - \frac{wl^2}{18}\right)$$

$$= \frac{1}{\sqrt{3}}\left(\frac{\cancel{2}wl^2}{\cancel{18}}\right)$$

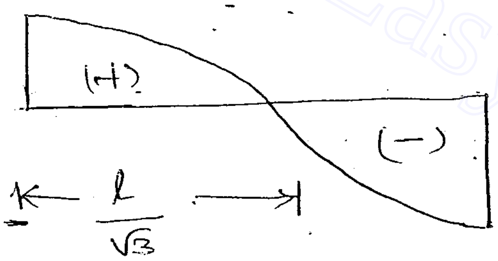
$$\boxed{\left.\begin{array}{l}\text{B.M @}\\ x = \frac{l}{\sqrt{3}}\end{array}\right\} = \frac{wl^2}{9\sqrt{3}}}$$

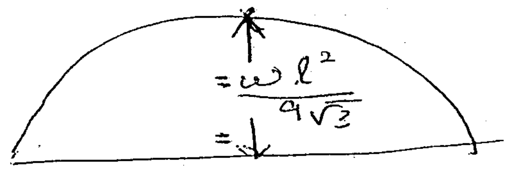


$V_{xx} = 0$

$R_A - \frac{1}{2} \times w' \times x = 0$

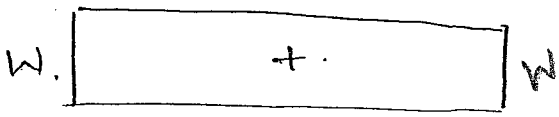
$$\boxed{x = \frac{l}{\sqrt{3}}}$$

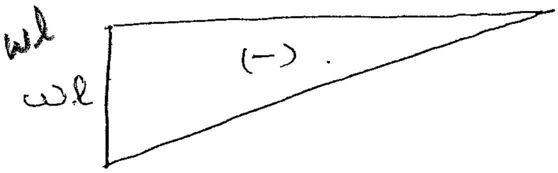




5.)

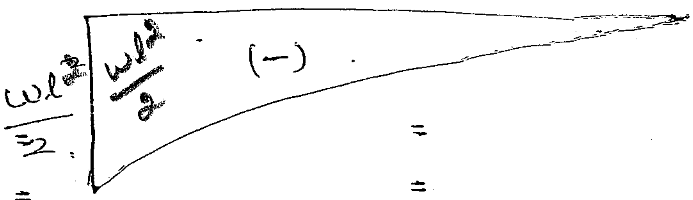
$R_A = W$



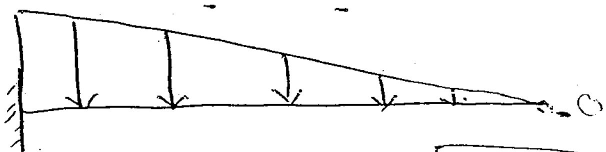


6.)

$R_A = wl$.

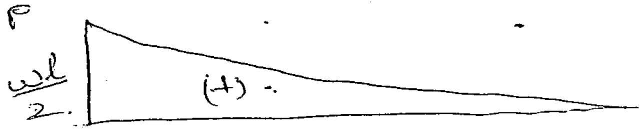


7.)



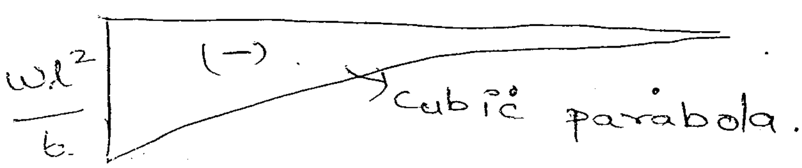
$R_A = \frac{1}{2} \times w \times l$.

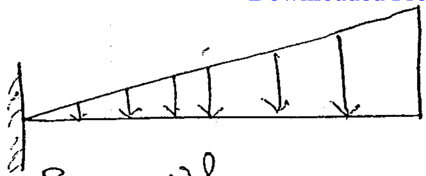
$\boxed{R_A = \frac{wl}{2}}$



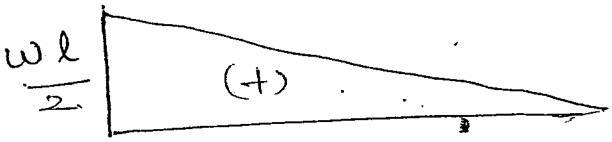
$$B.M = \frac{wl}{2} \times \frac{l}{3}$$

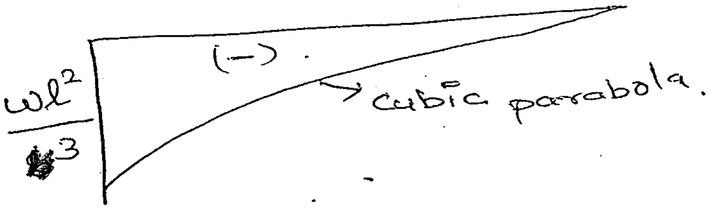
$$= \frac{wl^2}{6}$$



8.) 

$R_A = \frac{wl}{2}$.





B.M @ support

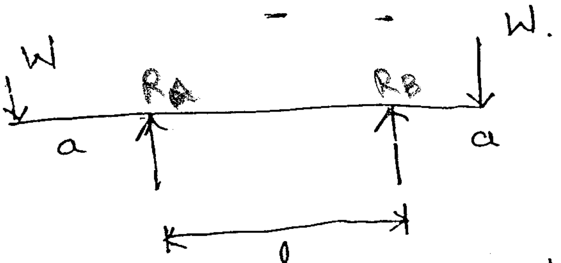
$$= \frac{w \times l}{\cancel{2}} \times \frac{\cancel{2}l}{3}$$

$$= \frac{wl^2}{3}.$$

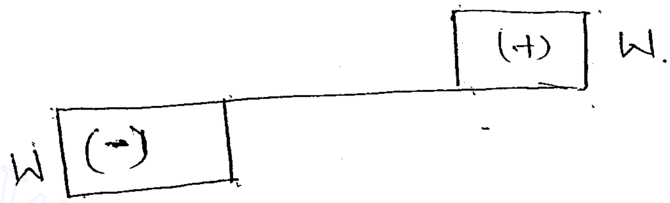
21/10/2015

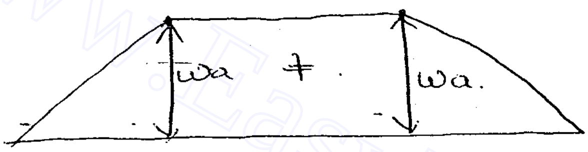
SFD and BMD.

1.)

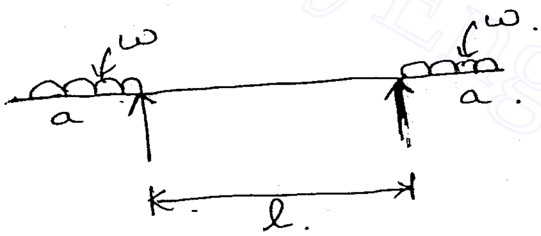


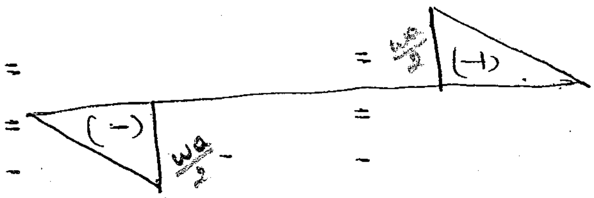
$R_A = W$ $R_B = W$.

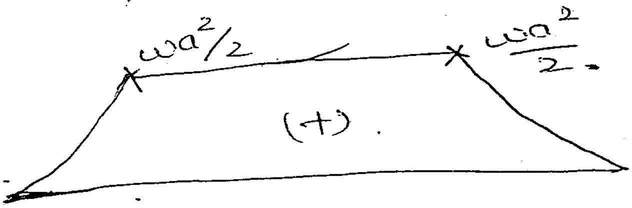




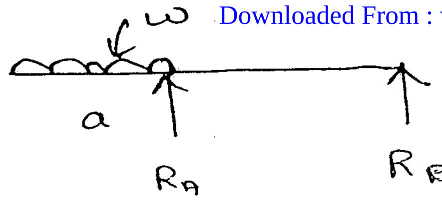
2.)







3.)



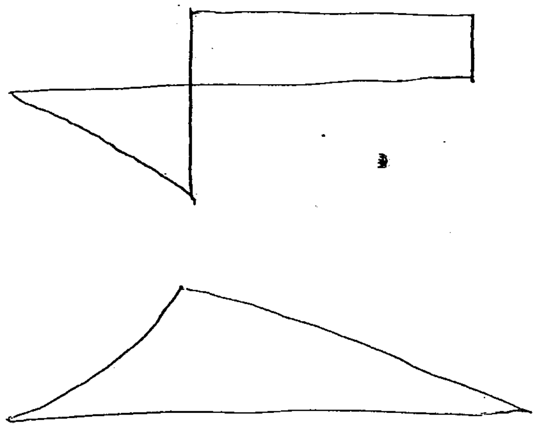
$R_A + R_B = \omega a$.

$\Sigma M_A = 0$.

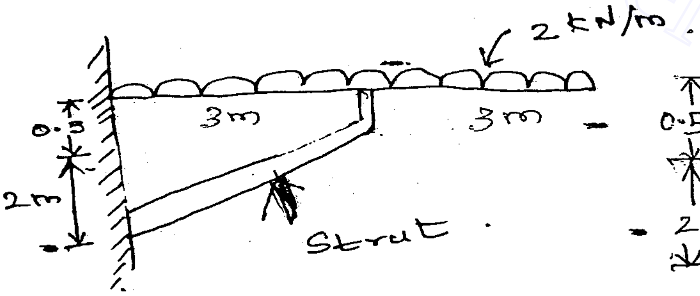
$R_B \, l = -\frac{\omega a^2}{2}$

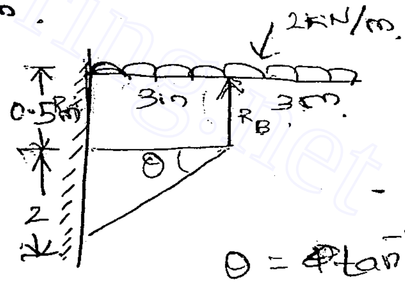
$R_B = -\frac{\omega a^2}{2l}$.

$R_A = \omega a + \frac{\omega a^2}{2l}$.



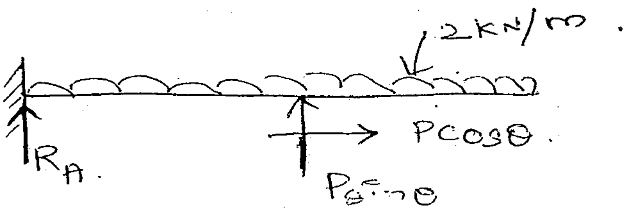
4.) A beam is supported by a strut. Determine force in the strut. thrust in the beam. Max. B.M in the beam. Max. S.F in the beam and S.FD and B.M.D.





$\theta = \tan^{-1}\left(\frac{2}{3}\right)$

$\theta = 33.69$.



$R_A + P\cos\theta = 2 \times 6$.

$\Sigma M_A = 0$,

$\left(P\sin\theta \times 3\right) + \left(P\cos\theta \times 0.5\right) - \frac{2 \times 6^2}{2} = 0$.

$3P \sin 33.69 + 0.5P \cos 33.69 = 36$

$$\boxed{P = 17.31 \text{ kN}}$$

$R_A = 12 - 17.31 \sin 33.69$

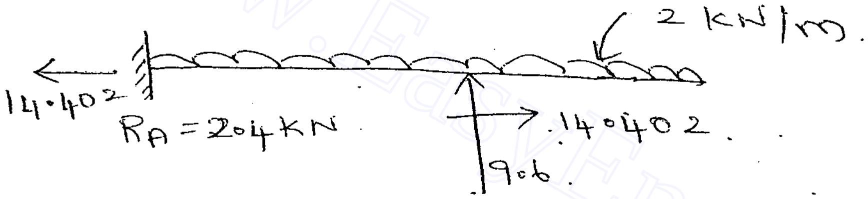
$$\boxed{R_A = 2.4 \text{ kN}}$$

$P \sin\theta = 17.31 \sin 33.69$

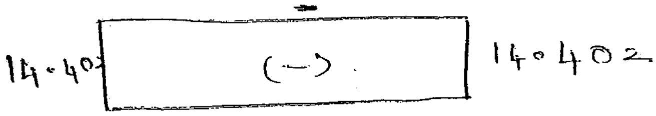
$= 9.6$

$P \cos\theta = 17.31 \cos 33.69$

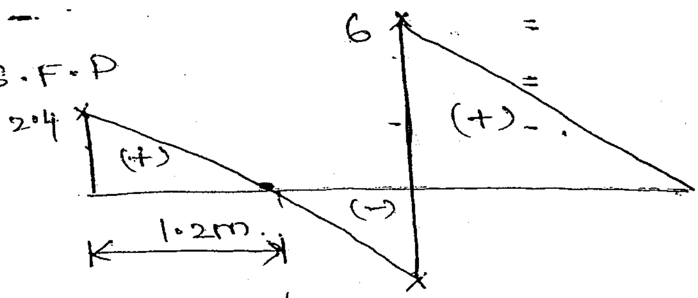
$= 14.402$ kN



Thrust -



S.F.D

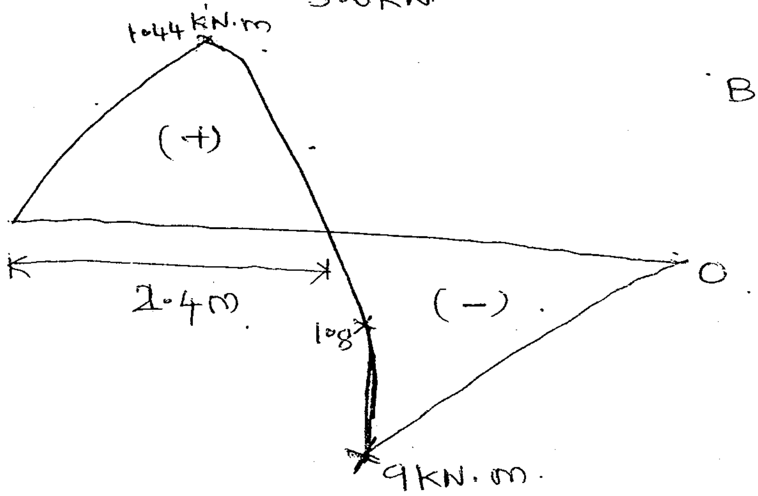


$V_{xx} = 0$

$R_A - 2x = 0$

$2.4 = 2x$

$$\boxed{x = 1.2 \text{ m}}$$

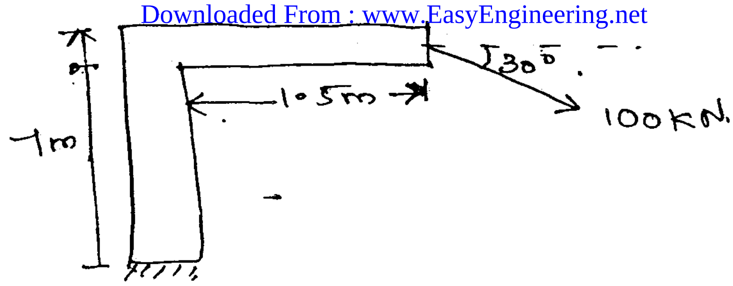


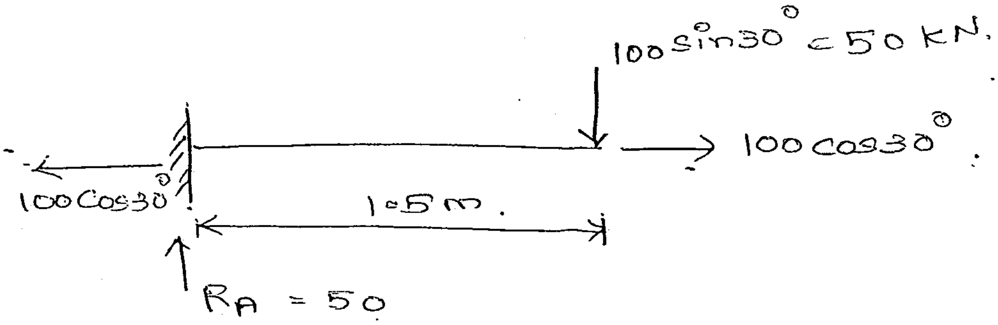
B.M @ $x = 1.2$

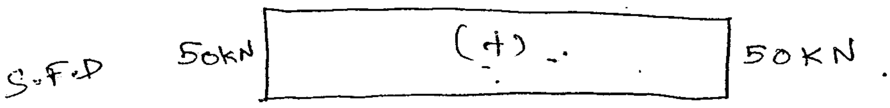
$2.4 \times 1.2 - \frac{2 \times 1.2^2}{2}$

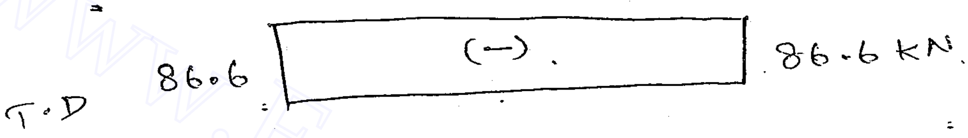
$= 1.44$ kN.m

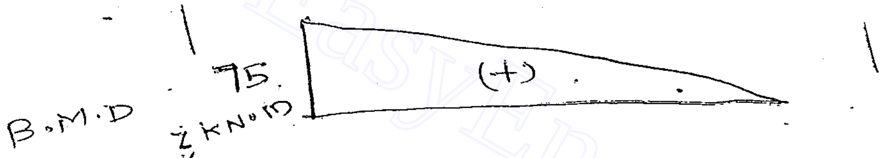
5.)

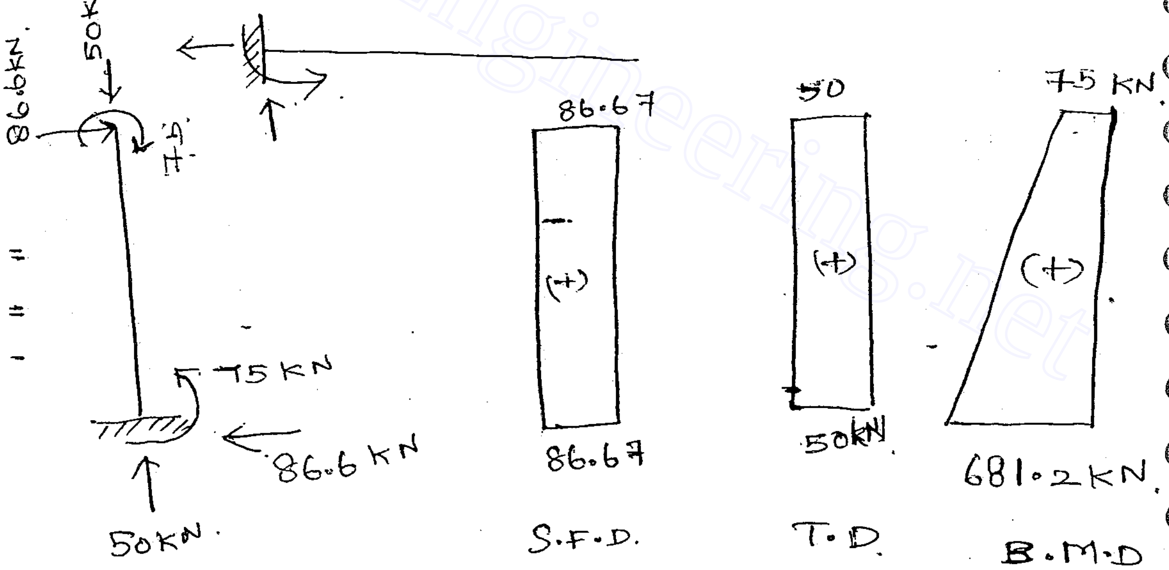

1m
1.5m
30°
100KN.


100 sin30° = 50 KN.
100 Cos30°
100 Cos30°
1.5m.
RA = 50


S.F.D
50kN
(+)
50KN


T.D
86.6
(−)
86.6 KN.


B.M.D
75
KN.m
(+)

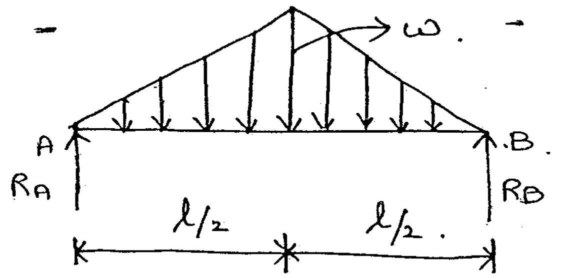

86.6kN
50KN
75KN
86.6KN
50KN.
86.67
(+)
86.67
S.F.D.
50
(+)
50KN
T.D.
75 KN
(+)
681.2KN.
B.M.D

3/10/2015

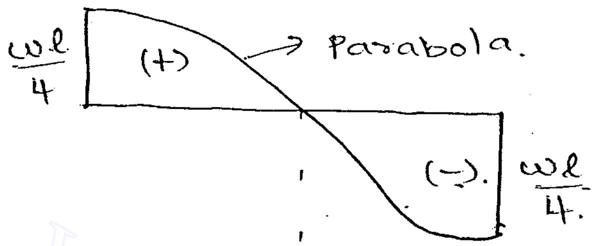
STRENGTH OF MATERIALS.

S.F.D and B.M.D.

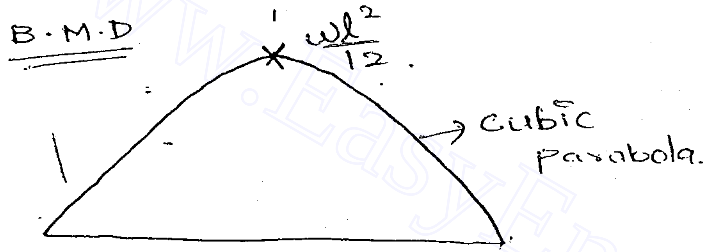
1.)



S.F.D



B.M.D



$$R_A + R_B = \frac{1}{2}\times w \times l.$$

$$\Sigma M_A = 0.$$

$$R_B \times l - \frac{1}{2}\times w \times l \times \frac{l}{2} = 0.$$

$$R_B \cancel{l} = \frac{1}{2}\times w \times \cancel{l} \times \frac{l}{2}$$

$$\boxed{R_B = \frac{wl}{4}}$$

$$\boxed{R_A = \frac{wl}{4}.}$$

Taking moment about centre.

$$= R_A \times \frac{l}{2} - \left(\frac{1}{2}\times w \times \frac{l}{2} \times \ldots\right.$$

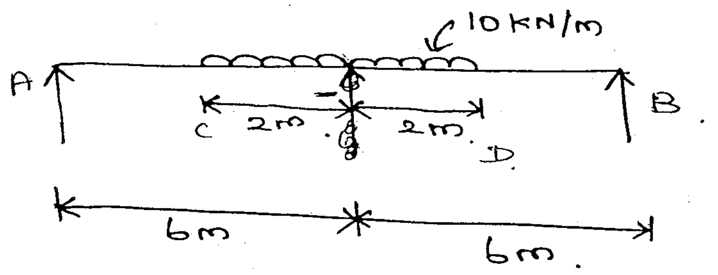
$$= \frac{wl}{4}\times\frac{l}{2} - \left[\frac{wl^2}{24}\right]$$

$$= \frac{wl^2}{8} - \frac{wl^2}{24}.$$

$$= \frac{2wl^2}{24}.$$

$$\boxed{M_{max} = \frac{wl^2}{12}.}$$

2) Draw S.F.D and B.M.D for the beam.



$M_A = 0.$

$R_B \times 12 - 10 \times 4 \times (2+4) = 0.$

$R_B = \frac{240}{12} = 20\,kN.$

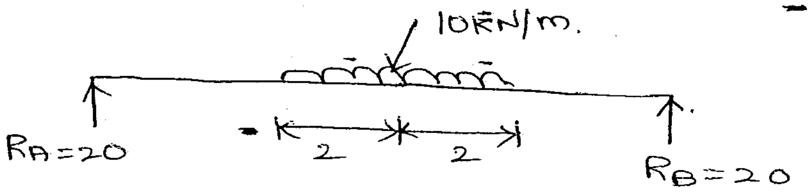
$R_A = 40 - 20.$

$\boxed{R_A = 20\,kN}$

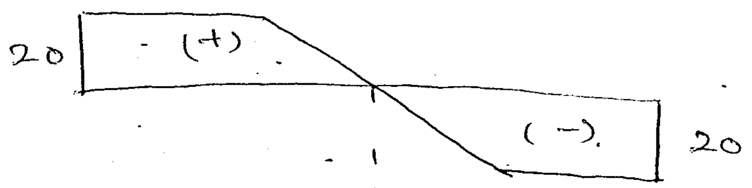
$M_C = R_A \times 4.$
$= 20 \times 4$
$= 80\,kN.m.$

$M_D = R_B \times 4.$
$= 20 \times 4$
$= 80\,kN.m.$

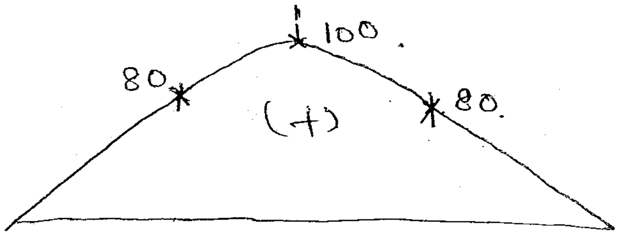
$M_{max} = R_A \times 6 - 10 \times 2 \times 1.$
$= (20 \times 6) - 20.$
$= 120 - 20$
$= 100\,kN.m.$



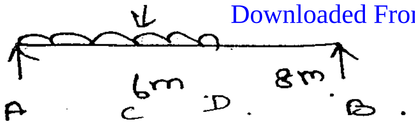
S.F.D



B.M.D



3.)

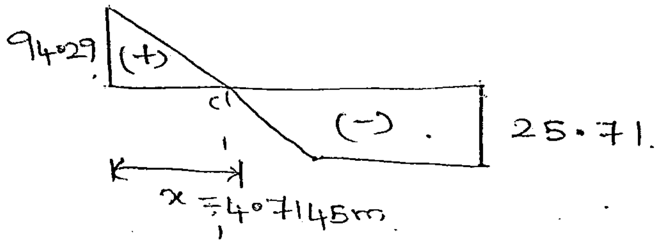


$\Sigma M_A = 0$

$R_B \times 14 = \frac{20 \times 6^2}{2}$

$R_B = 25.71\ kN$

$R_A = 94.29\ kN$

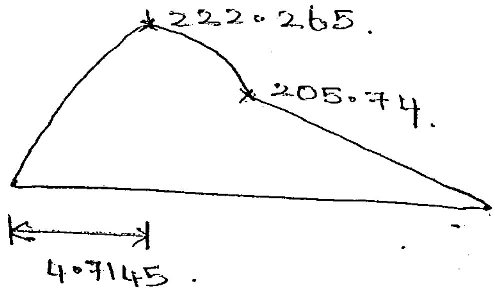


$V_{xx} = 0$

$R_A - 20x = 0$

$94.29 = 20x$

$x = 4.7145m$



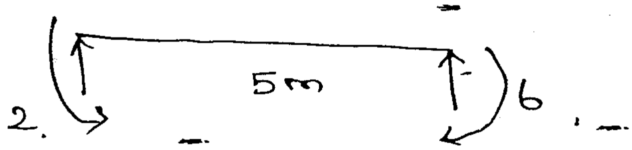
$M_{max} = R_A \times x - \left(\frac{20x^2}{2}\right)$

$= 222.265\ kN.m$

$M_D = (94.29 \times 6) - \frac{20 \times 6^2}{2}$

$= 205.74\ kN.m$

4.)



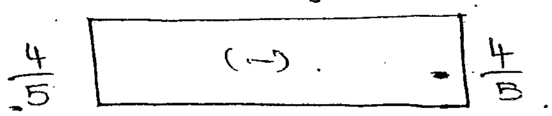
$R_A + R_B = 0$

$\Sigma M_A = 0$

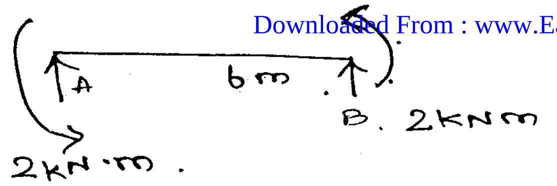
$R_B \times 5 - 6 + 2 = 0$

$\boxed{R_B = \frac{4}{5}}$

$\boxed{R_A = -\frac{4}{5}}$



5.)



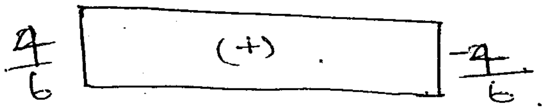
$R_A + R_B = 0$.

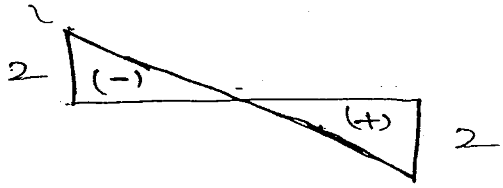
$\Sigma M_A = 0$.

$R_B \times 6 + 2 + 2 = 0$.

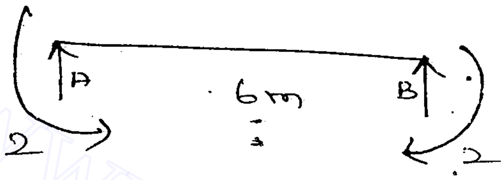
$\boxed{R_B = -\frac{4}{6}}$

$\boxed{R_A = \frac{4}{6}}$





6.)



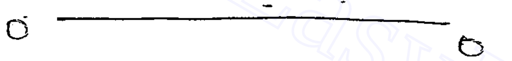
$\Sigma M_A = 0$.

$R_B \times 6m - 2 + 2 = 0$

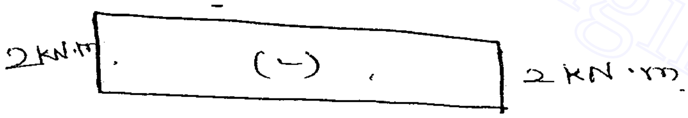
$\boxed{R_B = 0}$

$R_A = 0$

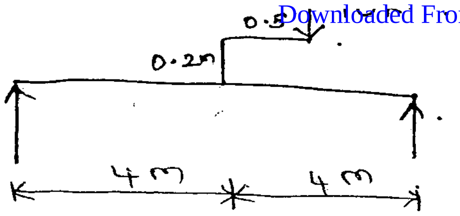
S.F.D.

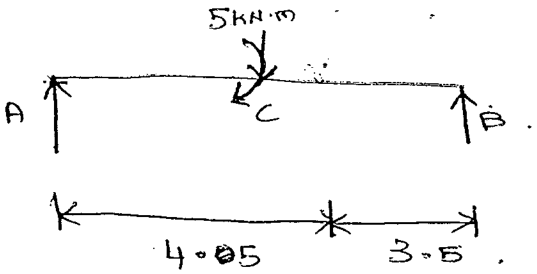


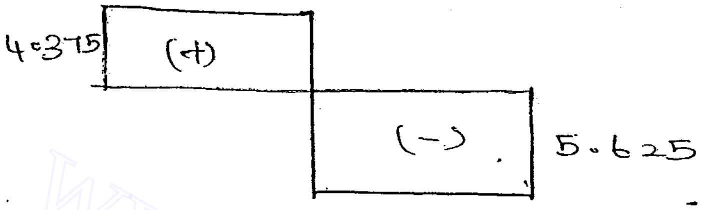
B.M.D

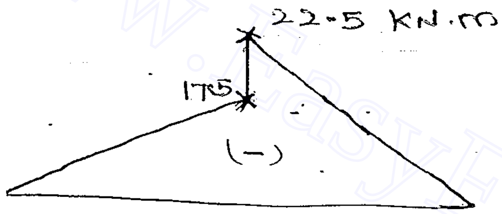


7)









$\Sigma M_A = 0$

$R_B \times 8 = (10 \times 4.05) + 5$

$R_B = \frac{45}{8}$

$R_B = 5.625$ kN

$R_A = 10 - 5.625$

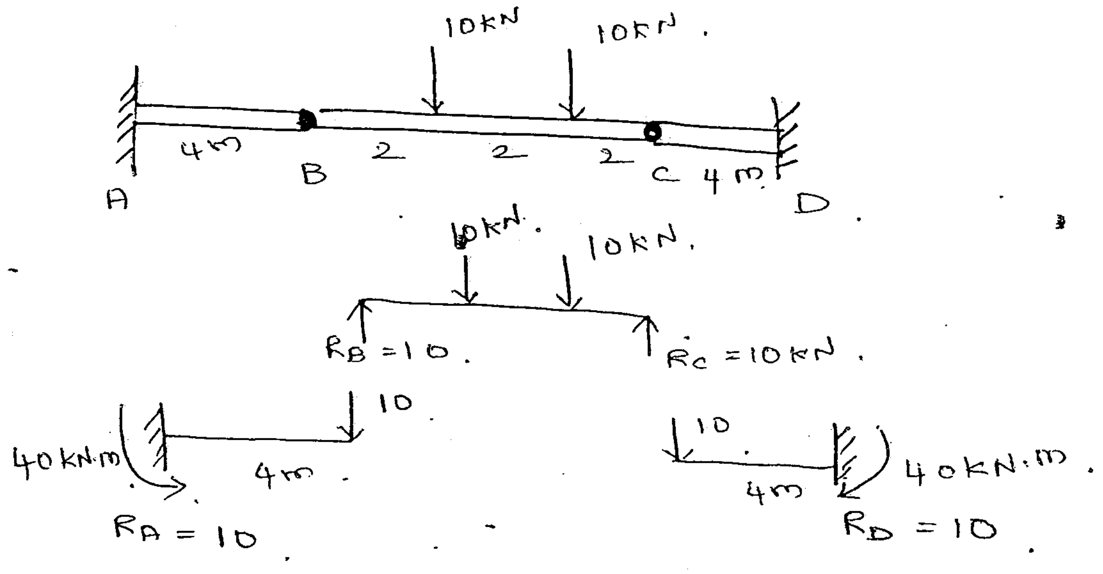
$= 4.375$ kN

$M_C = R_A \times 4 + 5$

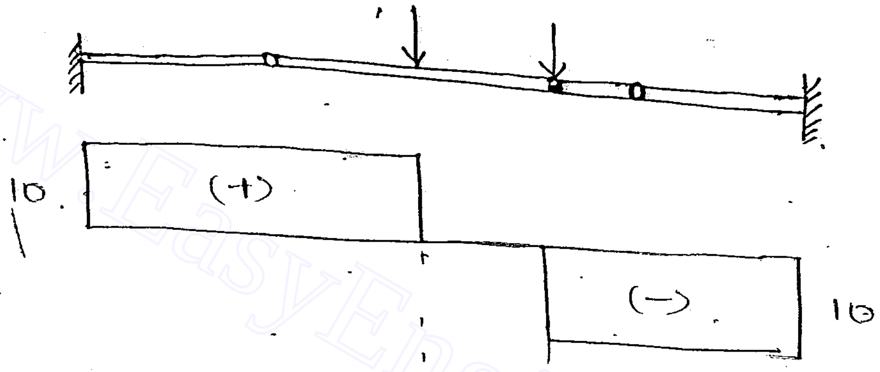
$= (4.375 \times 4) + 5$

$= 22.5$ kN·m

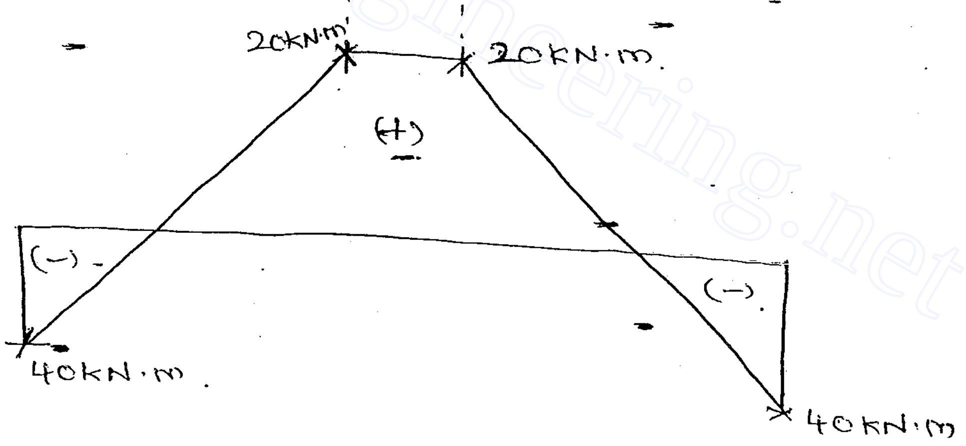
8.) Determine Max S.F and B.M for the compound beam given below.



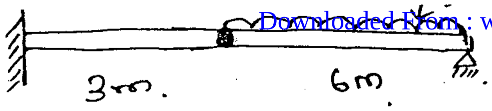
S.F.D.

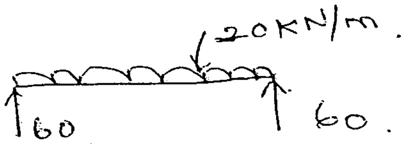


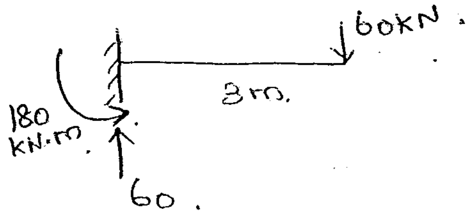
B.M.D.

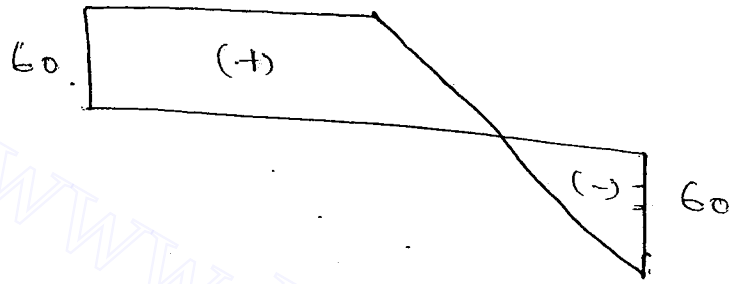


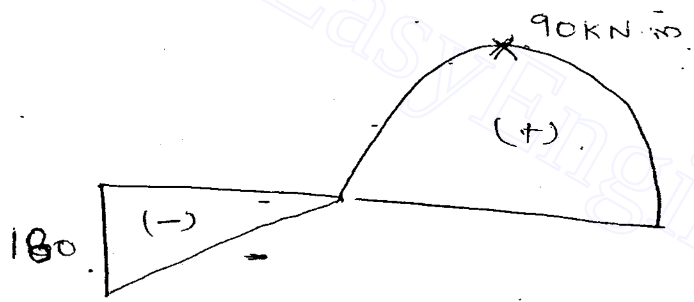
9.)



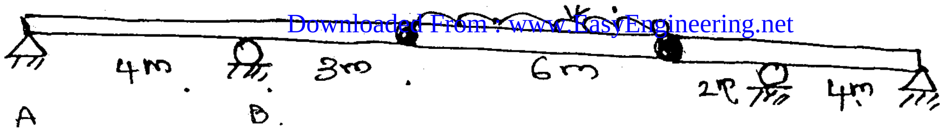


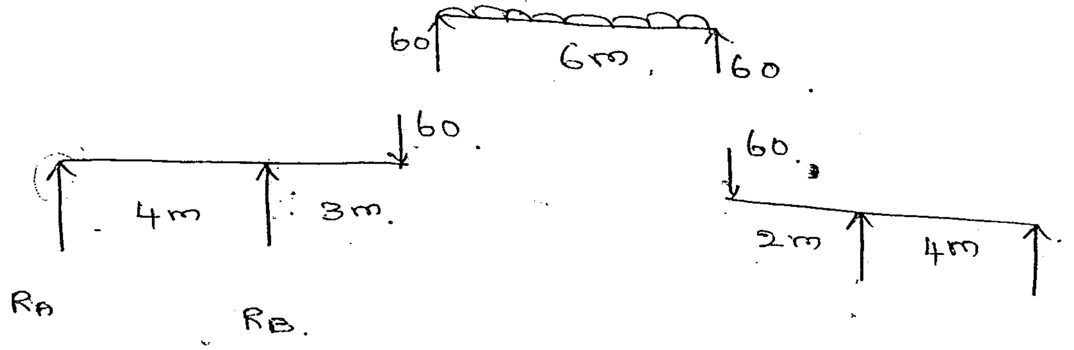






10)





$\Sigma M_B = 0$

$R_A \times 4 = 60 \times 3$

$\boxed{R_A = 45\,kN}$

$\boxed{R_B = 105\,kN}$

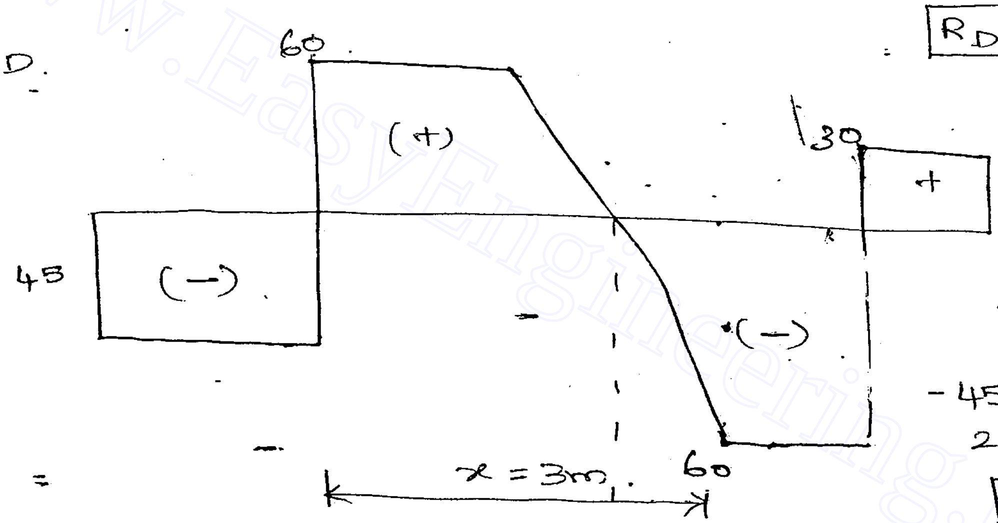
$\Sigma M_D = 0$

$R_C \times 4 = 60 \times 6$

$R_C = 90\,kN$

$\boxed{R_D = -30\,kN}$

S.F.D.



$V_{xx} = 0$

$-45+105-20x = 0$

$20x = 60$

$\boxed{x = 3m}$

~~M@3m~~

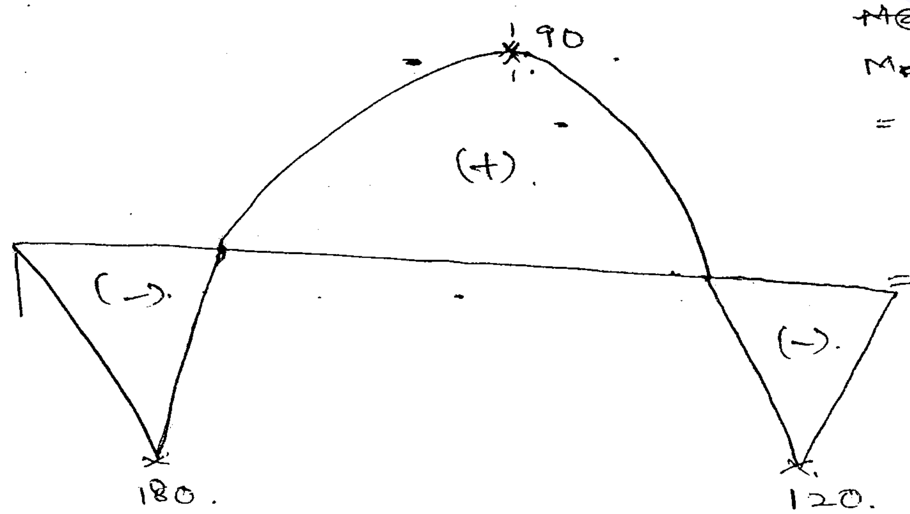
M_{max}

$= (-45\times10)$

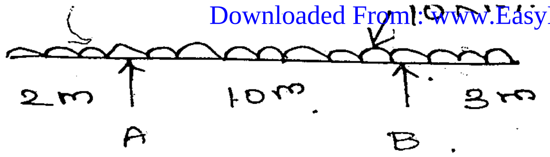
$+(105\times6)$

$-(20\times\frac{3^2}{2})$

$= 90\,kN\cdot m$



11.)



$R_A + R_B = 150$.

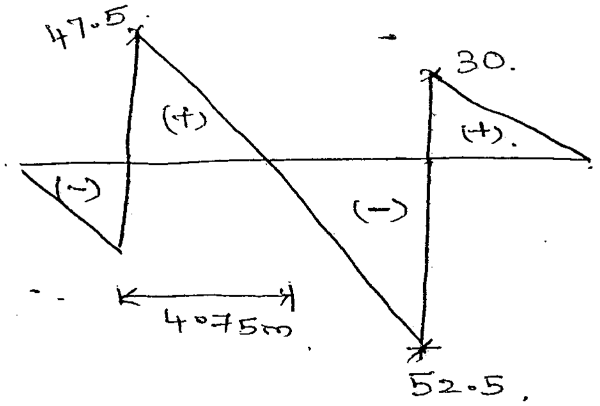
$\Sigma M_A = 0$.

$$R_B \times 10 - \left(\frac{10\times 13^2}{2}\right) + \left(\frac{10\times 2^2}{2}\right) = 0$$

$10 R_B = 825$.

$\boxed{R_B = 82.5\ \text{kN}}$

$\boxed{R_A = 67.5\ \text{kN}}$

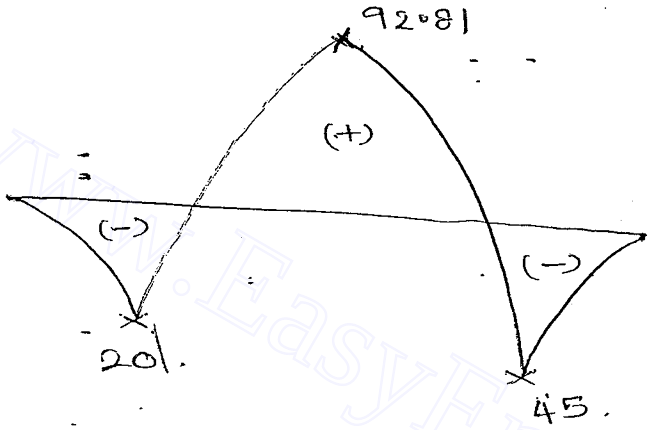


$V_{xx} = 0$.

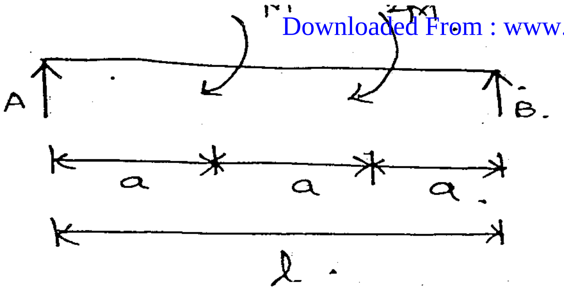
$R_A - 10\times(2+x) = 0$

$67.5 - 20 = 10x$.

$\boxed{x = 4.75\text{m}}$



12.)

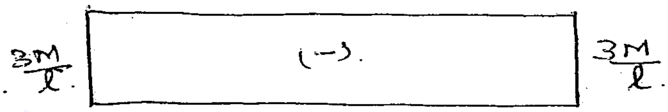


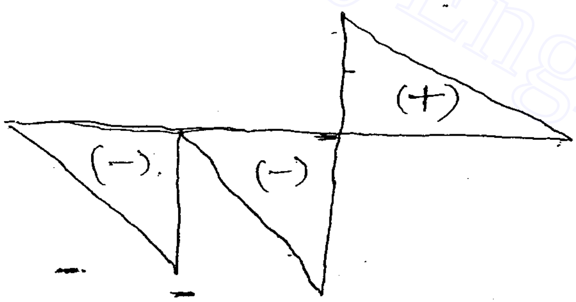
$\Sigma M_A = 0.$

$R_B \, 3a = 2M + M.$

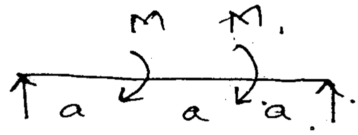
$R_B = \frac{3M}{l}.$

$R_A = -\frac{3M}{l}.$





(13.)

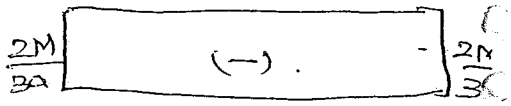


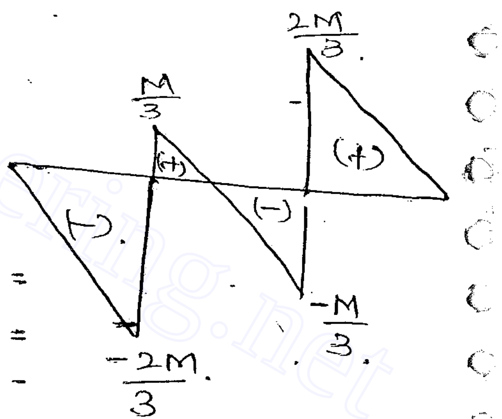
$\Sigma M_A = 0.$

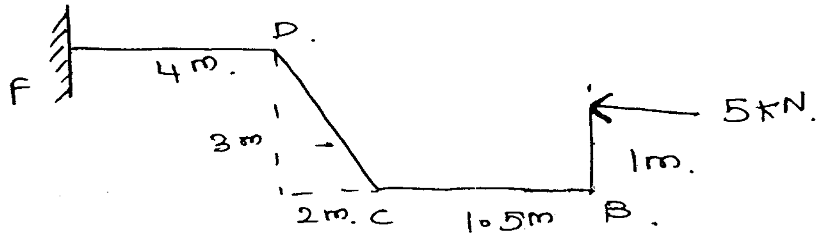
$3 R_B a = 2M.$

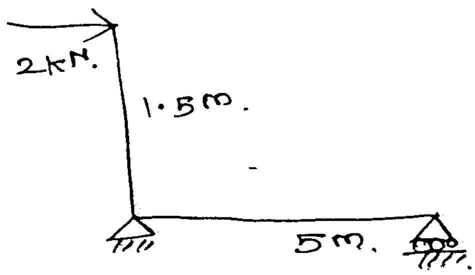
$R_B = \frac{2M}{3a.}$

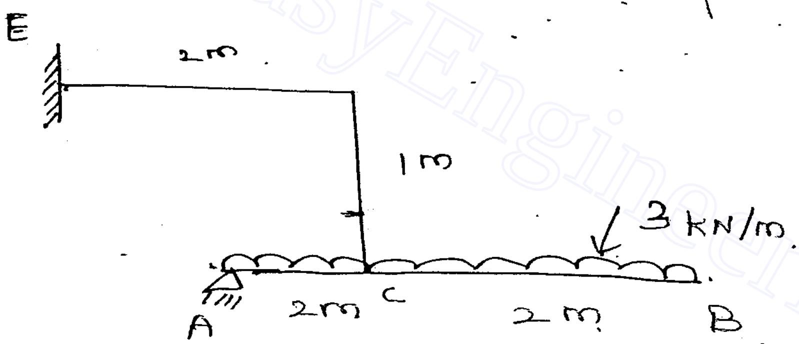
$R_A = -\frac{2M}{3a}.$






D.
F
4m.
3m
5kN.
1m.
2m. C
1.5m
B.


2kN.
1.5m.
5m.


E
2m.
1m
3 kN/m.
A
2m
C
2 m.
B.

28/10/2015

SOM

COMPLEX STRESSES.

* To ensure the safety of the structural component, we not only have to ensure that the structural component is under equilibrium due to external forces. But also each and every point inside the volume of the structural component must be in equilibrium and must must have stresses less than the maximum permissible stress.

Thus we need to know on which plane maximum normal stress will act, on which maximum shear stress will act, what is the magnitude of normal and shear stress.

As the magnitude of normal and shear stress varies with inclination of plane. stress on a plane can be calculated from stress on other plane using method of transformation of stresses.

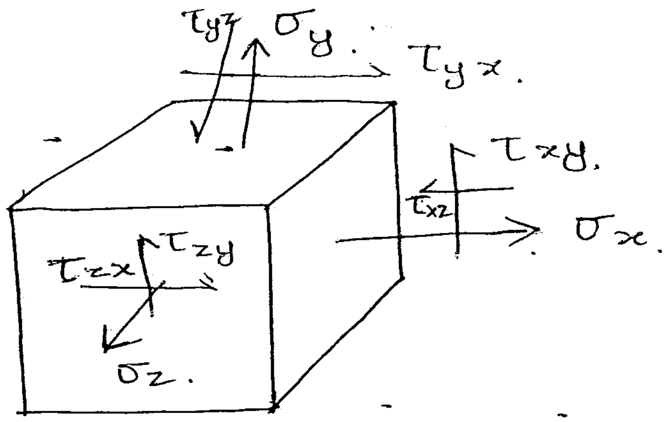
At any point most general state of stress represented by 6 components

$\sigma_x, \sigma_y, \sigma_z, \tau_{xy}, \tau_{yz}, \tau_{zx}$

But in particular case not all of these 6 stresses act simultaneous.

Here we will consider only 3 stress components $(\sigma_x, \sigma_y, \tau_{xy})$.

PLANE STRESS :



when two faces of cubic elements are free from any element the stress condition is called plane stress.

condition :

For eg: if Z-axis is choosen on which no. $\perp^{r}$ to the face on which (i.e) stress is acting then

$$\sigma_{zx} = \tau_{yz} = \tau_{zx} = 0$$

remaing component σ_x, σ_y, τ_{xy}.

Examples of plane stress condition are

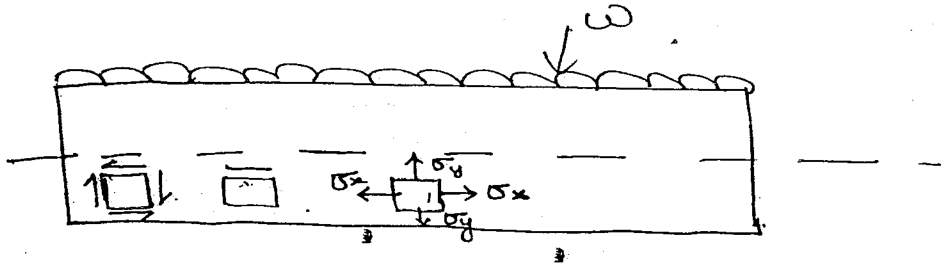
(i) Bar in tension and compression.

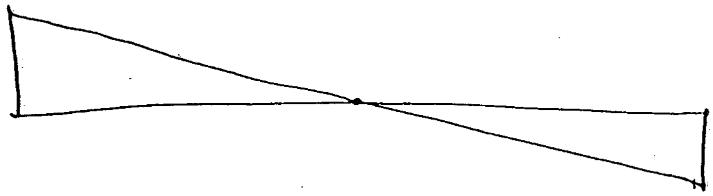
(ii) shaft in torsion.

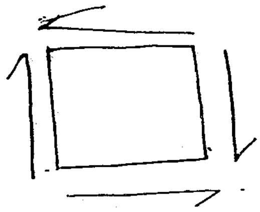
(iii) Beam in bending

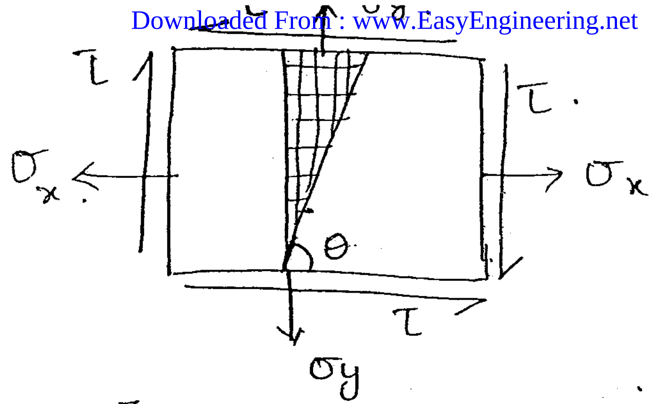
(iv) plates subjected to forces acting in the plane of the plate.

(v) stress on the surface of structural element that is not subjected to external forces etc.,

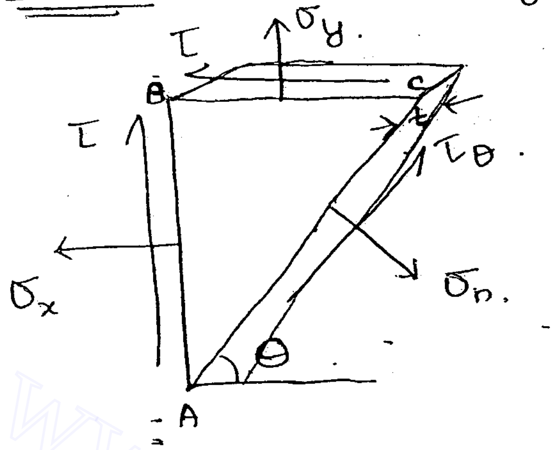

ω
σy
σx
σx
σy







Stress



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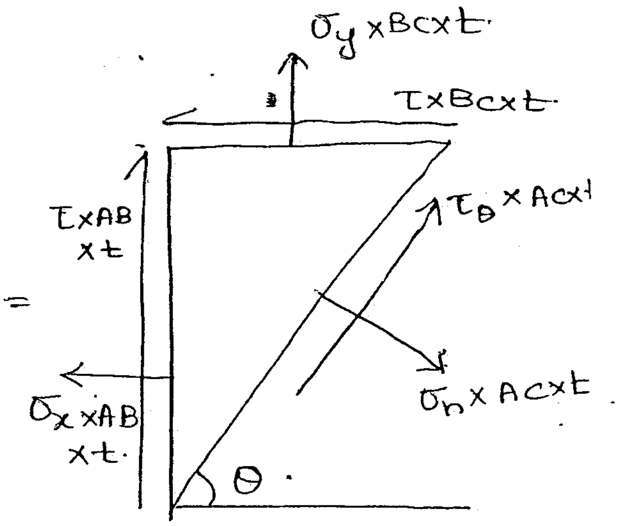
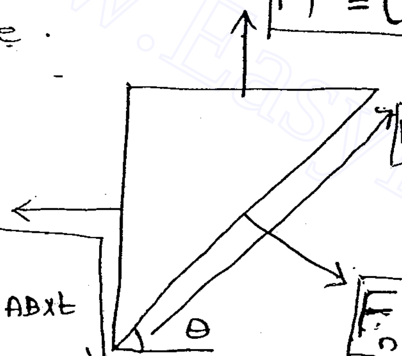


Force

$\boxed{F_1 = (\sigma_y \times BC \times t + \tau \times AB \times t)}$



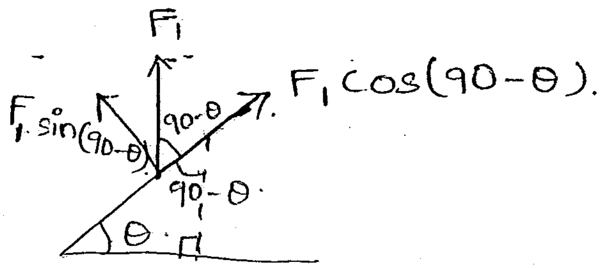
$\boxed{F_s = \tau_\theta \times AC \times t}$

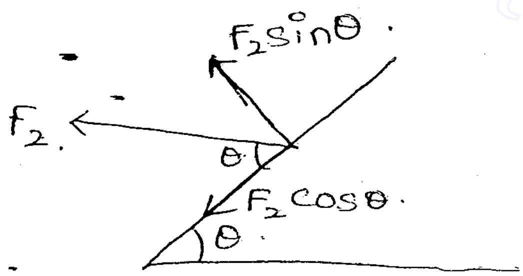
$\boxed{F_2 = (\sigma_x \times AB \times t + \tau \times BC \times t)}$

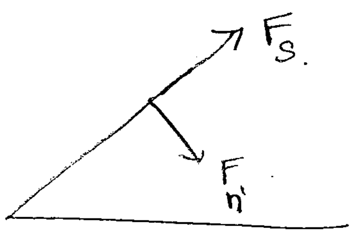
$\boxed{F_n = \sigma_n \times AC \times t}$

$AB = AC \sin\theta$

$BC = AC \cos\theta$







ΣF along the plane $= 0$.

$$F_1 \cos(90-\theta) - F_2 \cos\theta + F_s = 0.$$

$$F_1 \sin\theta - F_2 \cos\theta + F_s = 0.$$

$$(\sigma_y \times BC \times t + \tau \times AB \times t)\sin\theta - (\sigma_x \times AB \times t + \tau \times BC \times t)\cos\theta + \tau_\theta \times A_c \times t = 0.$$

$$[\sigma_y \times A_c \cos\theta \times t + \tau \times A_c \sin\theta \times t]\sin\theta - [\sigma_x \times A_c \sin\theta \times t + \tau \times A_c \cos\theta \times t]\cos\theta + (\tau_\theta \times A_c \times t) = 0.$$

$$(\sigma_y \cos\theta + \tau \sin\theta)\sin\theta - (\sigma_x \sin\theta + \tau \cos\theta)\cos\theta + \tau_\theta = \frac{0}{A_c \times t}$$

$$\sigma_y \cos\theta \sin\theta + \tau \sin^2\theta - \sigma_x \sin\theta \cos\theta - \tau \cos^2\theta + \tau_\theta = 0$$

$$\tau_\theta = \sigma_x \sin\theta \cos\theta + \sigma_y \sin\theta \cos\theta + \tau(\cos^2\theta - \sin^2\theta)$$

$$\boxed{\tau_\theta = \frac{(\sigma_x \mp \sigma_y)}{2}\sin 2\theta + \tau \cos 2\theta}$$

ΣF across plane $= 0$.

$$F_n - F_2 \sin\theta - F_1 \sin(90-\theta) = 0.$$

$$F_n - F_2 \sin\theta - F_1 \cos\theta = 0.$$

$$(\sigma_n \times AC \times t) - (\sigma_x \times AB \times t + \tau \times BC \times t)\sin\theta$$
$$- (\sigma_y \times BC \times t + \tau \times AB \times t)\cos\theta = 0$$

$$(\sigma_n \times Ac \times t) - (\sigma_x \times Ac\sin\theta \times t + \tau \times AC\cos\theta \times t)\sin\theta$$
$$- (\sigma_y \times Ac\cos\theta \times t + \tau \times AC\sin\theta \times t)\cos\theta = 0$$

$$\sigma_n - \sigma_x \sin^2\theta - \tau \cos\theta\sin\theta - \sigma_y \cos^2\theta$$
$$- \tau \sin\theta\cos\theta = \frac{0}{Ac \times t}$$

$$\sigma_n - \sigma_x \sin^2\theta - \sigma_y \cos^2\theta - 2\tau \sin\theta\cos\theta = 0$$

$$\boxed{\sigma_n = \sigma_x \sin^2\theta + \sigma_y \cos^2\theta + \tau \sin 2\theta}$$

when $\theta = 45^\circ$ $\tau_\theta = \tau_{max}$.

τ_{max} is

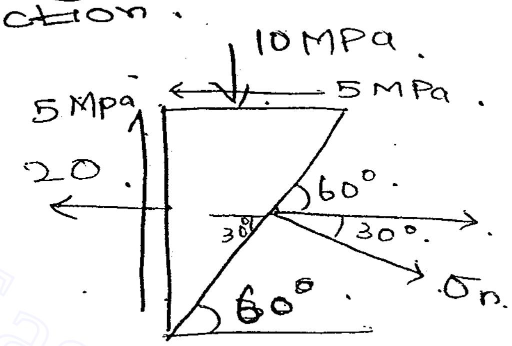
$$\tau_\theta = \frac{\sigma_x - \sigma_y}{2} \sin 2(45^\circ) + \tau\cos 2(45^\circ)$$

$$= \frac{\sigma_x - \sigma_y}{2}(1) + (\tau \times 0)$$

$$\boxed{\tau_{max} = \frac{\sigma_x - \sigma_y}{2}}$$

$$\boxed{\sigma_r \text{, Resultant stress} = \sqrt{\tau_\theta^2 + \sigma_n^2}}$$

1. Determine normal and tangetial stress which is subjected to two $\perp^{r}$ stress 20 M.Pa tensile along x direction and 10 MPa compressive along y direction shear stress 5 MPa. The normal stress on the inclined plane makes an angle 30° with the direction of tensile stress. Determine σ_r. and direction.



$$\tau_\theta = \frac{\sigma_x - \sigma_y}{2} \sin 2\theta + \tau \cos 2\theta$$

$$= \frac{20 - 10}{2} \sin 2(60) + 5 \cos 2(60)$$

$$\boxed{\tau_\theta = 10.49 \text{ MPa}}$$

$$\sigma_n = \frac{\sigma_x + \sigma_y}{2}$$

$$\sigma_n = \sigma_x \cos^2\theta + \sigma_y \sin^2\theta + \tau \sin 2\theta$$

$$= 20 \sin^2 60 - 10 \sin^2 60 + 5 \sin 2(60)$$

$$\boxed{\sigma_n = 1.88 \text{ MPa}}$$

$$\sigma_n = \sigma_x \sin^2\theta + \sigma_y \cos^2\theta + \tau \sin 2\theta$$

$$= 20 \sin^2 60 + -10 \cos^2 60 + 5 \sin 2(60)$$

$$\boxed{\sigma_n = 16.83 \text{ MPa}}$$

$$\sigma_x = \sqrt{\sigma_n^2 + \tau_\theta^2}$$

$$= \sqrt{16.83^2 + 10.49^2}$$

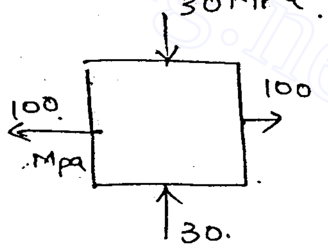
$$\sigma_x = 19.83 \text{ MPa}.$$

$$\alpha = \tan^{-1}\left(\frac{\sigma_n}{\tau_\theta}\right)$$

$$= \tan^{-1}\left(\frac{16.83}{10.49}\right)$$

$\boxed{\alpha = 58.06^\circ}$ w.r.t σ_n.

2.) Determine max shear stress and its plane of load.



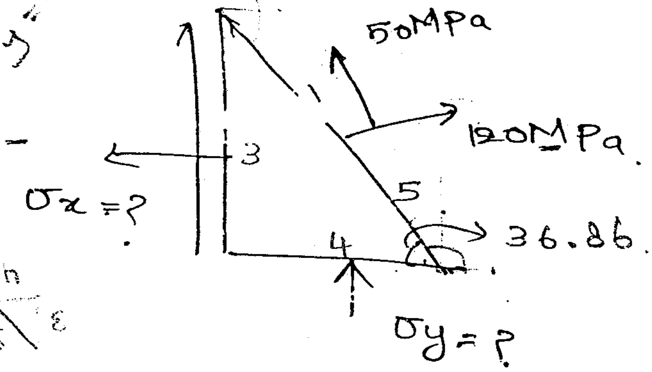
$$\tau_{max} = \frac{\sigma_x - \sigma_y}{2}$$

$$= \frac{100 + 30}{2}$$

$$\boxed{\tau_{max} = 65 \text{ MPa}}$$

Plane is @ 45°.

3.) 53.3
$\sigma_y = 15.7$



$\sigma_x = 186.67$
$\sigma_y = 82.5$

$\frac{50}{0.96}$ $= \sigma_x - \sigma_y$

$$\tau_\theta = \frac{\sigma_x - \sigma_y}{2} \cdot \sin 2\theta.$$

$\sigma_x - \sigma_y = 104.1$

$$\sigma_n = \sigma_x \sin^2\theta + \sigma_y \cos^2\theta$$

$\theta = 143.13$

$$-120 = 0.36\,\sigma_x + 0.64\,\sigma_y.$$

$$50 = \left(\frac{\sigma_x - \sigma_y}{2}\right) \sin 2(36.87)$$

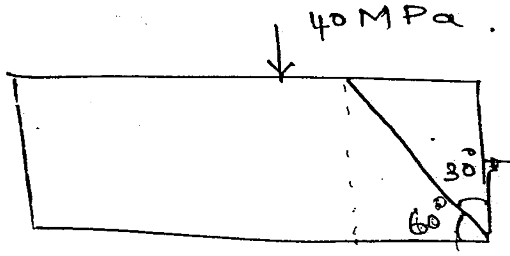
$$\sigma_x - \sigma_y = 104.17$$

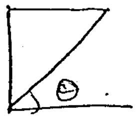
$$0.36\,\sigma_x + 0.64\,\sigma_y = -120$$

$$\sigma_x - \sigma_y = 104.17$$

$$0.36\,\sigma_x + 0.64\,\sigma_y = -120.$$

4.) Determin σ_r for given plane.





$$\tau_\theta = \frac{\sigma_x - \sigma_y}{2} \cancel{\cos} \sin 2(60^\circ)$$

$$= \frac{100+40}{2} \sin 120$$

$$= 60.62 \text{ MPa}$$

$$\sigma_n = \sigma_x \sin^2\theta + \sigma_y \cos^2\theta$$

$$= 100(\sin 60)^2 \cancel{+} - 40(\cos 60)^2$$

$$\boxed{\sigma_n = 65 \text{ MPa}}$$

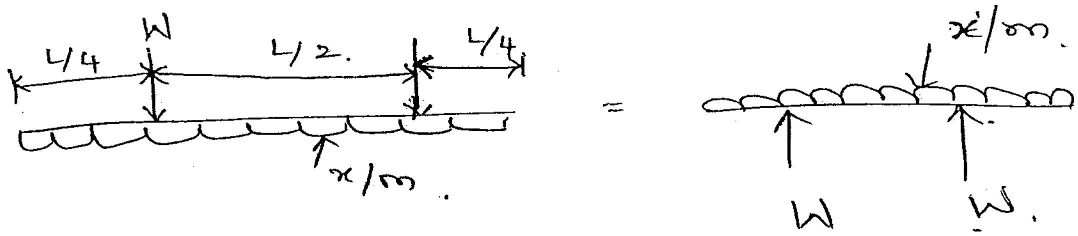
$$\sigma_r = \sqrt{\tau_\theta^2 + \sigma_n^2}$$

$$= \sqrt{(60.62)^2 + 65^2}$$

$$\boxed{\sigma_r = 88.88}$$

28/10/2015

1.) Determine the B.M @ centre of plank floating in water with 2 person of weight W. standing @ distance $\frac{L}{4}$ from the free end. Length of plank L



$\Sigma V = 0$.

$W + W - x \times L = 0$.

$x \times L = 2W$

$$\boxed{x = \frac{2W}{L}}$$

B.M @ centre.

$$= W \times \frac{L}{4} - \left(\frac{\cancel{2}W}{\cancel{L}} \times \frac{\cancel{L}}{\cancel{2}} \times \frac{L}{4}\right)$$

$$= \frac{WL}{4} - \frac{WL}{4}$$

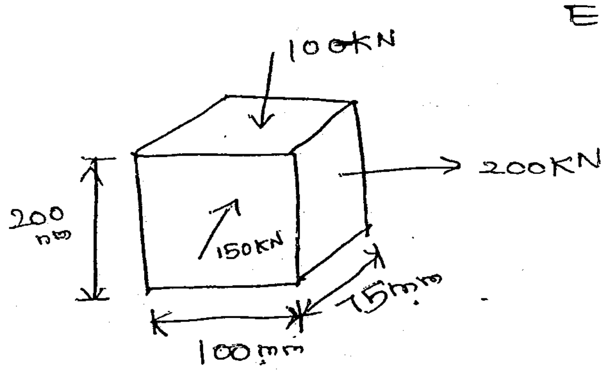
$$\boxed{B \cdot M \text{ @ centre} = 0.}$$

2) Determine change in volume of an object as shown in figure below.

$E = 200 \times 10^9$ N/m

$\mu = 0.3$

$E = 2 \times 10^5$ N/mm²



$$\boxed{\epsilon_v = \frac{\delta v}{v}}$$

$$\delta v = \epsilon_v \times V$$
$$= -0.1 \times 100 \times 75 \times 200$$
$$= -150 \times 10^{+3} \text{ mm}^3$$
$$= -1.5 \times 10^{-4} \text{ m}^3$$

$$\boxed{\epsilon_v = \frac{\sigma_x + \sigma_y + \sigma_z}{E}(1-2\mu)}$$

$$\epsilon_v = \frac{(200 - 100 - 150) \times 10^3}{2 \times 10^5}[1 - 2(0.3)]$$

$$= -0.1$$

$$\sigma_y = \frac{-100 \times 10^3}{100 \times 75} = -13.33 \text{ N/mm}^2$$

$$\sigma_x = \frac{200 \times 10^3}{75 \times 200} = 13.33 \text{ N/mm}^2$$

$$\sigma_z = \frac{-150 \times 10^3}{100 \times 200} = -7.5 \text{ N/mm}^2$$

$$\epsilon_v = \frac{13.33 - 13.33 - 7.5}{2 \times 10^5}(1 - 2(0.3))$$

$$\boxed{\epsilon_v = -1.5 \times 10^{-5}}$$

$$\delta v = \epsilon_v \times V$$
$$= -1.5 \times 10^{-5} \times 100 \times 75 \times 200$$

$$\boxed{\delta v = -22.5 \text{ mm}^3}$$

3\. A metallic bar of size $l = 300mm$, $40 \times 40mm$ C/S. Subjected to axial load 160kN. $\delta l = \cancel{4}\,0.12\,mm$. $\delta b = 0.005mm$. Determine E, μ Bulk ~~bar~~ modulus $K, G, \epsilon_v, \delta_v$.

$l = 300mm$
$A = 40mm$
$P - 160kN$

$$\boxed{\delta l = \frac{Pl}{\cancel{2}AE}}$$

$$0.12 = \frac{160 \times 10^3 \times 300}{40 \times 40 \times E}$$

$$E = 2.5 \times 10^5 \ N/mm^2$$

$$\epsilon_v = \frac{\sigma_x + \sigma_y + \sigma_z}{E}(1-2\mu)$$

$$\mu = \frac{(\delta b/b)}{\left(\frac{\delta l}{l}\right)} = \frac{(0.005/40)}{\left(\frac{0.12}{300}\right)} = \cancel{0.0042}\ 0.3125$$

$$\cancel{K=} \quad E = 3K(1 - 2\mu)$$

$$K = \frac{E}{3(1-2\mu)}$$

$$= \frac{2.5 \times 10^5}{3(1-2(0.3125))}$$

$$\boxed{K = 22.22 \times 10^3 \ N/mm^2}$$

$$G = \frac{E}{2(1+\mu)} = \frac{2.5 \times 10^5}{2(1+0.3125)}$$

$$\boxed{G = 95.238 \times 10^3 \ N/mm^2}$$

$$\epsilon_v = \sigma_x \qquad K = \frac{\text{direct stress}}{\cancel{\text{Volum}}\ \epsilon_v}$$

$$\epsilon_v = \frac{\text{Direct stress}}{K}$$

$$= \frac{100}{22.22 \times 10^2}$$

$$\cancel{\epsilon_v = 0.045} \quad \boxed{\epsilon_v = 4.5 \times 10^{-4}}$$

$$\varepsilon_v = \frac{\delta v}{v}.$$

$$\delta v = \varepsilon_v \times v.$$

$$= 4.5 \times 10^{-4} \times 300 \times 40 \times 40.$$

$$\boxed{\delta v = 216 \text{ mm}}$$

11/11/2015

STRENGTH OF MATERIALS

PRINCIPLE STRESSES AND THEORIES OF FAILURE

* Principle stress is the normal stress acting on any plane where shear stress is zero ($\tau = 0$)
* The plane where only normal stress acts (shear stress or tangential stress is zero) is called principal plane.
* There are 2 principle plane.
  * Major principal plane
  * Minor principal plane.
* Principal planes must be always @ 90° to each other.
* Max. shear stress $\tau_{max} = \frac{\sigma_1 - \sigma_2}{2}$
* Plane along which maximum shear stress acts make ang angle 45° from the principle plane.
* To determine the location of principal plane, the tangential stress (or) shear stress is made equal to zero from where θ can be determined.

$$\tan(180 - 2\theta) = \frac{2\tau}{\sigma_x - \sigma_y}$$

* The major and minor principle stresses are given by the formula:

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2}$$

$$= \frac{\sigma_x + \sigma_y}{2} \pm \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2}$$

* Location of principal plane:

$$\tau_\theta = 0.$$

$$\frac{\sigma_x - \sigma_y}{2}\sin 2\theta + \tau\cos 2\theta = 0.$$

$$\frac{\sin 2\theta}{\cos 2\theta} = \frac{-2\tau}{\sigma_x - \sigma_y}.$$

$$\tan 2\theta = \frac{-2\tau}{\sigma_x - \sigma_y}.$$

$$\tan(180 - 2\theta) = \frac{2\tau}{\sigma_x - \sigma_y}.$$

$$180 - 2\theta = \tan^{-1}\left(\frac{2\tau}{\sigma_x - \sigma_y}\right)$$

$$\theta_{P.P} = 90 - \frac{1}{2}\tan^{-1}\left(\frac{2\tau}{\sigma_x - \sigma_y}\right).$$

→ If $\sigma_x > \sigma_y$. $\theta_{P.P} = 45^\circ - 90^\circ$ from σ_x

→ If $\sigma_x = \sigma_y$. $\theta_{P.P} = 45^\circ$. from σ_x.

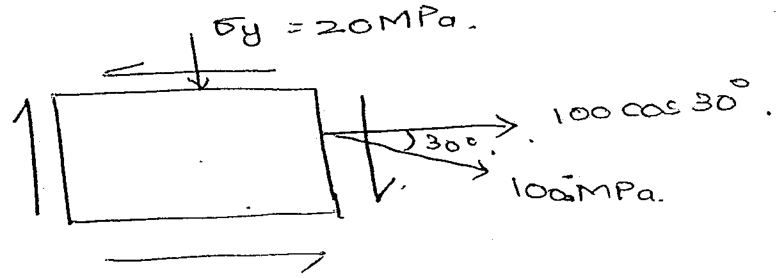
→ If. $\sigma_x < \sigma_y$ $\theta_{P.P} = 0^\circ - 45^\circ$. from σ_x.

$$\tan 2\theta = \frac{-2\tau}{\sigma_x - \sigma_y} = \frac{-2(-\tau)}{\sigma_x - \sigma_y}.$$

$$\tan 2\theta = \frac{2\tau}{\sigma_x - \sigma_y}.$$

$$\boxed{\theta = \frac{1}{2}\tan\left(\frac{2\tau}{\sigma_x - \sigma_y}\right)}$$

1. A steel material is subjected to direct stress @ an angle 30° with the horizontal having magnitude of 100 KPa and compressive stress of 20 MPa in I^{rl} direction. Determine principal stress, principal plane, max. shear stress.



$$\tau = 100 \sin 30 = 50 \text{ MPa}.$$

$$\sigma_x = 100\cos 30 = 86.6 \text{ MPa}.$$

$$\sigma_y = 20.$$

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2}$$

$$= 106.4 \text{ MPa}$$

$$\sigma_2 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2}$$

$$= 39.75 \text{ MPa}.$$

$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2} = \frac{106.4 - 39.75}{2} = 73.075 \text{ MPa}$$

$$\theta_{p.p} = 90 - \frac{1}{2}\tan^{-1}\left(\frac{2\tau}{\sigma_x - \sigma_y}\right).$$

$$\boxed{\theta_{p.p} = 68.46^\circ}$$

CONCRETE STRUCTURES

→ Concrete Technology

* properties of concrete
* Basics of mix design.

→ Concrete Design :

* Basic working stresses and limit state design concepts
* Analysis of ultimate load capacity
* Design of members subjected to flexure, shear, compression and torsion by limit state method.

→ Prestressed Concrete -

* Basic elements of prestressed concrete
* Analysis of beam sections at transfer and service load.

26/3/2015

(4)

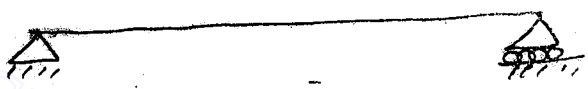
STRUCTURAL ANALYSIS

Structure:

A structure refers to a system of connected parts used to support a load.

When any elastic body is subjected to system of loads and deformation takes place and the resistance is set up against the deformation. Then the elastic body is known as structure.

eg: simply supported beam is a stable structure.



Mechanism:

If no resistance is set up in the body against the deformation then it is known as an unstable structure or mechanism.

The simply supported beam with an internal hinge is an unstable structure.

Classification of Structure:

Skeletal structure:

Structures which can be idealized to a series of straight or curved lines

eg: Roof trusses (or) building frame

Surface structure:

Structure which can be idealized to plane or curved surface

eg: slabs (or) and shells.

Solid structure:

structure which can neither be idealized to a skeleton nor to a plane or curved surface

eg: Massive foundation.

Classification of Skeletal Structure:

(i) Based on type of Joint:

→ Pin jointed frames.

when members are connected by means of pin-joint. These frames support the load by developing only axial forces, which the external load acts at the joints and members are straight.

(ii) Rigid Jointed Frames:

Assumptions:

→ The joints of rigid jointed frame are assumed to be rigid, so, that the angles between the members meeting at a joint remain unchanged.

→ These frames resist external forces by developing bending moments, shear forces, axial forces and twisting moment in the members of the fram

Based on Dimension:

(i) Plane Frames.

- All members of the plane fram as well as the external loads are assumed to be in one plane.

→ Pinjointed plane frame:

Members carrying only axic forces.

→ Rigid jointed plane frame:

Members carry Axial force, Bending moment and shear force.

Note: If loaded in its own plane any cross section of member is subjected to 3 internal forces. (Axial force, Shear force, Bending moment).

If loading is away from the plane torsional moment also acts on the member.

(ii) Space Frame:

All members ~~to~~ donot lie in one plane.

Every often it is also a combination of series of frames.

→ Pin jointed space frame

→ Rigid jointed space frame.

Note: Any cross section of a member of a skeletal space structure there are six internal force component (Axial force, Biaxial shear force (V_x, V_y), Twisting moment (T) and biaxial moment (M_x, M_y)].

Equations of static Equilibrium:

* For plane Frame: subjected to inplane external force in xy plane.

$$\Sigma F_x = 0 \qquad \text{(or)} \qquad \Sigma H = 0$$
$$\Sigma F_y = 0 \qquad\qquad\quad \Sigma V = 0$$
$$\Sigma M_z = 0 \qquad\qquad\quad \Sigma M = 0$$

* For space frame

$$\Sigma F_x = 0$$
$$\Sigma F_y = 0$$
$$\Sigma F_z = 0$$
$$\Sigma M_x = \Sigma M_y = \Sigma M_z = 0$$

10/8/2015 (1)

STRUCTURAL ANALYSIS.

(4).

Statically Determinate Structure:

* Structure that can be analysed with the help of equations of static equilibrium ~~equat~~ alone.

* It undergoes finite equilibrium deformation before the condition of equilibrium satisfied.

eg: A cantilever beam, a simply supported beam, a 3 hinged arch, a suspension cable.

Statically Indeterminate Structure:

* Any structure whose reaction components (or) Internal stresses cannot be established by using the equation of static equilibrium alone.

* The no. of forces > greater than no. of equilibrium equations.

* For complete analysis additional equations based on conditions of compatability (or) consistent displacement can be used.

* D

Degree of static indeterminancy (or)

Redundancy:

* Equations in addition to static equilibrium equation necessary to complete analysis statically indeterminate structure.

D_S = No. of. unknowns - No. of. static equilibrium equation

* Formulation of static indeterminancy

$D_s = D_{se} + D_{si}$

where,

D_{se} - External indeterminancy.

$D_{se} = r - 6$ (space frame)

$D_{se} = r - 3$ (plane frame)

D_{si} = Internal indeterminancy.

$D_{si} = m - (2j - 3)$ (pin jointed plane frame)

$D_{si} = m - (3j - 6)$ (Pin jointed space frame)

$D_{si} = 3C$ (Rigid jointed plane frame)

$D_{si} = 6C$ (Rigid jointed space frame)

j - No. of. joints.

C - No. of. Cuts required for obtaining an open configuration.

Simplified Formula:

$D_s = m + r - 2j$ (Pinjointed Plane fram

$= (m+r) - 3j$. (Pinjointed space frame

$= (3m+r) - 3j$. (Rigid jointed plane from

$= (6m+r) - 6j$. (Rigid jointed space fram

m – No. of. member forces.

r – No. of. reaction.

Internal pin (or) Internal Hinge.

* A pin provided anywhere in the structure cannot transmit the moment from one part to another part of the structure and thus provides one additional condition equation.

$\Sigma M = 0$.

Internal Link:

* A link (consisting of a short bar with a pin at each end) provided anywh in the structure is incapable of transferi a moment as well as horizontal force from one part to another part of the structure and thus provides to addition equation condition.

$\Sigma M = 0$, $\Sigma H = 0$.

Statically Indeterminate beams:

Eg: Propped cantilever, Continous beam, Fixed beam.

* General loading (which has both horizontal or vertical component. $\Sigma H = \Sigma V = \Sigma M = 0$).

No. of. equilibrium equation = 3.

<u>Vertical</u> <u>loading</u>:

$$\left.\begin{array}{l}\text{No. of. equilibrium} \\ \text{equation}\end{array}\right\} = 2.$$

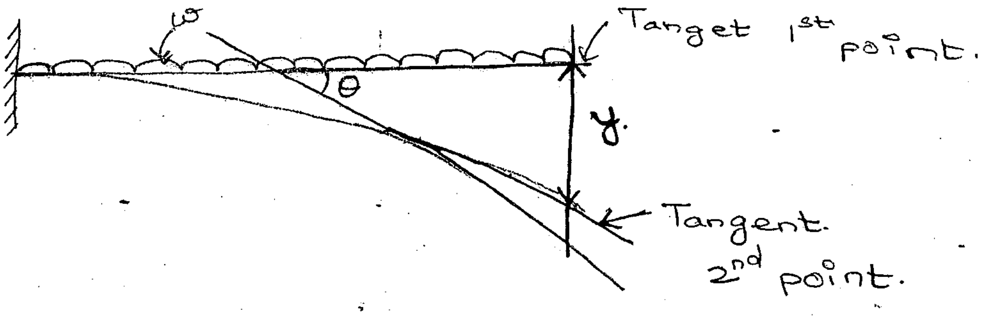
$$\Sigma V = \Sigma M = 0.$$

Only for beams not for frames.

1/9/2015

STRUCTURAL ANALYSIS

MOMENT AREA THEOREM:



Theorem 1:

The angle made by intersection of two tangents drawn at ~~two~~ places of elastic curve. (deflected shape of beam) is equal to area of B.M diagram b/w two point of tangence divided by EI

(or)

The difference of angle b/w ~~at~~ two point of tangency of elastic curve is equal to Area of B.M divided by EI.

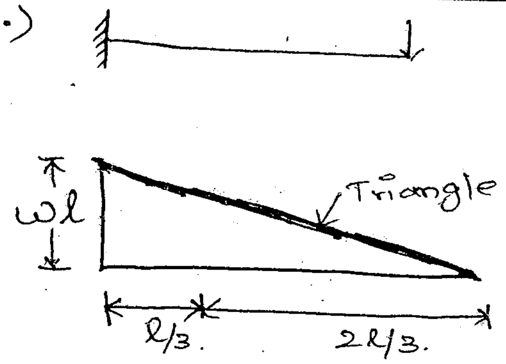
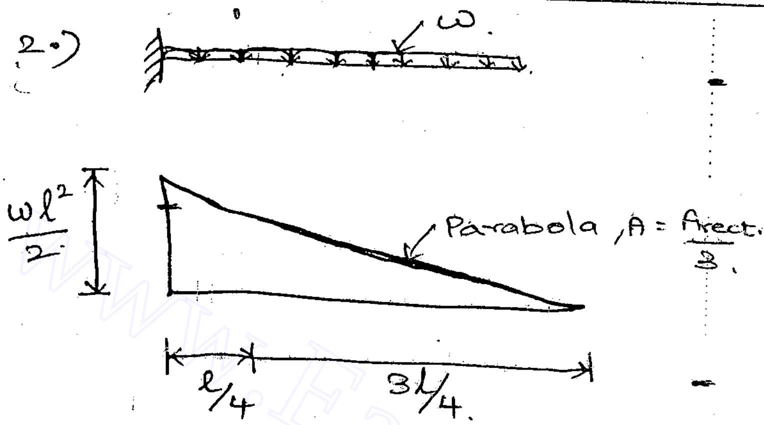
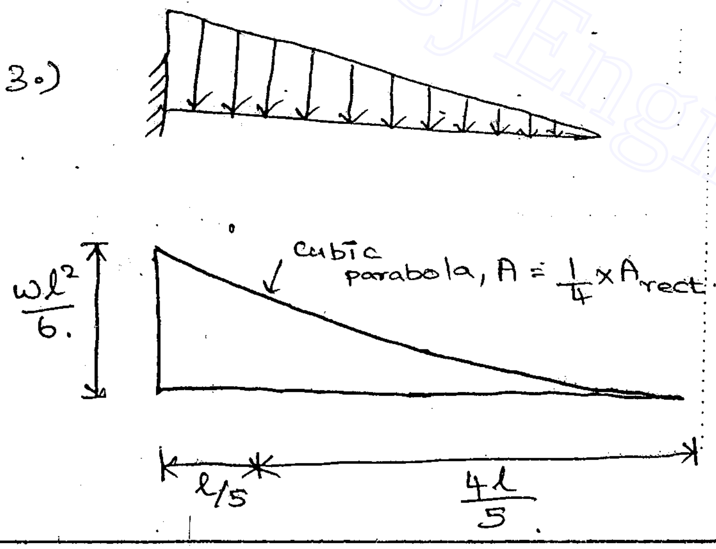
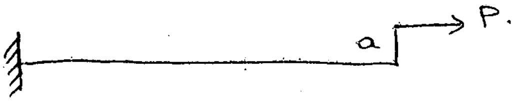
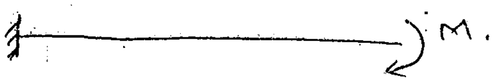
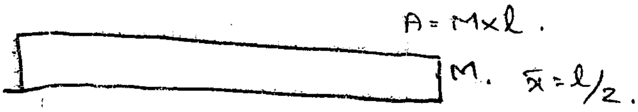
Theorem 2: $\boxed{y = \frac{A\bar{x}}{EI}}$

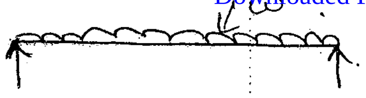
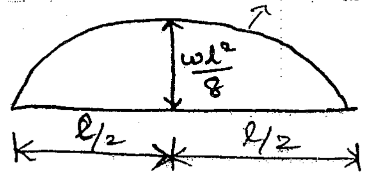
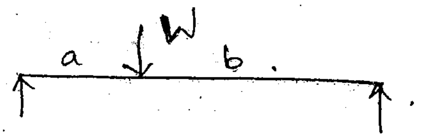
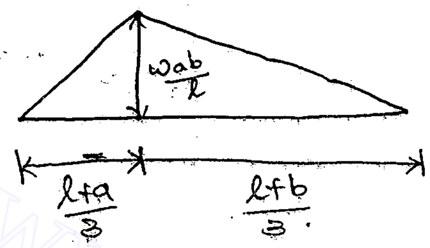
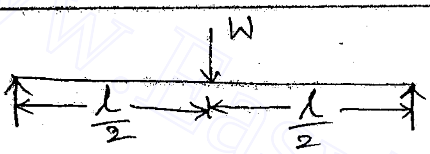
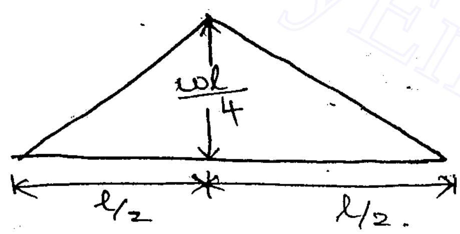
The intercept made by 2 tangents drawn on two points of elastic curve on a vertical reference line is equal to Moment of Area of B.M diagram b/w 2 ~~two~~ points of tangency about the vertical reference line divided by EI.

(or)

The vertical distance of second point with respect to first point is equal to the moment of Area of M/EI diagram about the second point.

Standard Cases:

Case	A	$\bar{x}$	θ	y
1.) 	$\frac{A_{rect}}{2}$	$\frac{2l}{3}$ (from free end)	$\frac{wl^2}{2EI}$	$\frac{wl^3}{3EI}$
2.) 	$\frac{A_{rect}}{3}$	$\frac{3l}{4}$ (from free end)	$\frac{wl^3}{6EI}$	$\frac{wl^4}{8EI}$
3.) 	$\frac{A_{rect}}{4}$	$\frac{4l}{5}$ (from free end)	$\frac{wl^3}{24EI}$	$\frac{wl^3}{30EI}$
4.)   (Pure bending member) 	A_{rect}	$\frac{l}{2}$ (from free end)	$\frac{Ml}{EI}$	$\frac{Ml^2}{2EI}$

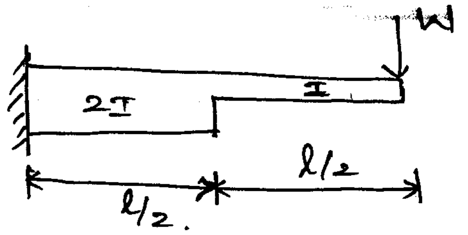
	A	$\bar{x}$	θ	y
5.)  consider half area due to symmetricity. Parabola $= \frac{2}{3} \times A_{rect}$. 	$\frac{2}{3} \times A_{rect}$	$\frac{5}{8} \times \frac{l}{2}$ from s.s end	$\frac{wl^3}{24EI}$	$\frac{5\,wl^4}{384EI}$
6.)  	$\frac{A_{rect}}{2}$	$\frac{2}{3}\left(\frac{l+a}{3}\right)$ (or) $\frac{2}{3}\left(\frac{l+b}{3}\right)$ from s.s end.	$\frac{wab(l+a)}{6EIl}$ (or) $\frac{wab(l+b)}{6EIl}$	$\frac{wab(l+a)}{27EIl}$ (or) $\frac{wab(l+b)}{27EIl}$
7.)  	$\frac{A_{rect}}{2}$	$\frac{l}{3}$ from s.s end	$\frac{wl^2}{16EI}$	$\frac{wl^3}{48EI}$

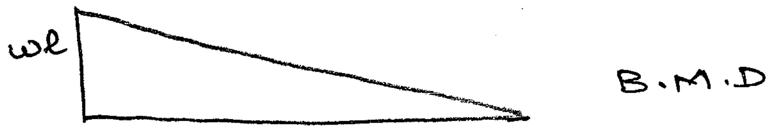
GEOTECHINAL ENGINEERING

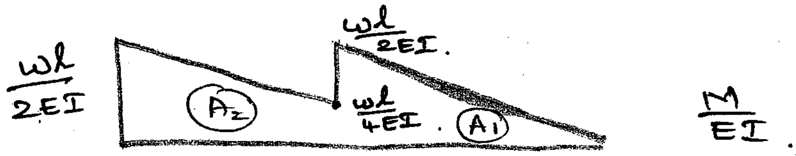
→ SOIL MECHANICS

→ FOUNDATION ENGINEERING

2.) Determine θ and y for the following beam :







θ = Area of B.M. diagram.

$= A_1 + A_2$.

$$= \left(\frac{1}{2} \times \frac{l}{2} \times \frac{wl}{2EI}\right) + \left[\left(\frac{l/2}{2}\right) \times \left(\frac{wl}{4EI} + \frac{wl}{2EI}\right)\right]$$

$$= \frac{wl^2}{8EI} + \left(\frac{l}{4} \times \frac{3wl}{4EI}\right)$$

$$= \frac{wl^2}{8EI} + \frac{3wl^2}{16EI}.$$

$$\boxed{\theta = \frac{5wl^2}{16EI}}$$

$y = A_1 \times \bar{x}_1 + A_2 \bar{x}_2$.

$$= \left[\frac{wl^2}{8EI} \times \frac{l}{3}\right] + \left[\frac{3wl^2}{16EI} \times \frac{7l}{9 \cdot 3}\right].$$

$$= \frac{wl^3}{24EI} + \frac{wl^3}{48EI}.$$

$$= \frac{3wl^3}{48EI}$$

$$\boxed{y = \frac{wl^3}{16EI}}$$

$$\bar{x}_1 = \frac{2(l/2)}{3} = \frac{l}{3}.$$

$$\boxed{\bar{x}_1 = \frac{l}{3}}$$

$$\bar{x}_2 = \left(\left(\frac{a+2b}{a+b}\right)\frac{h}{3}\right)$$

$$= \frac{l}{2} + \left[\frac{\frac{wl}{4EI} + \frac{2wl}{2EI}}{\frac{wl}{4EI} + \frac{wl}{2EI}} \times \right.$$

$$= \frac{l}{2} + \left[\frac{\frac{5wl}{4EI}}{\frac{3wl}{4EI}} \times \frac{l}{6}\right.$$

$$= \frac{l}{2} + \frac{5l}{18}.$$

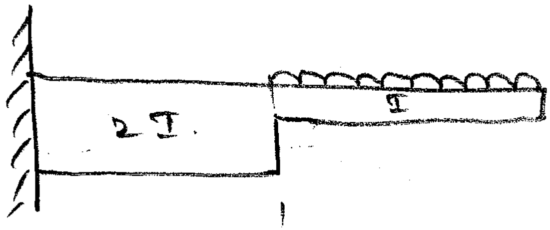
$$\boxed{\bar{x}_2 = \frac{14l}{18}} = \frac{7l}{9}.$$

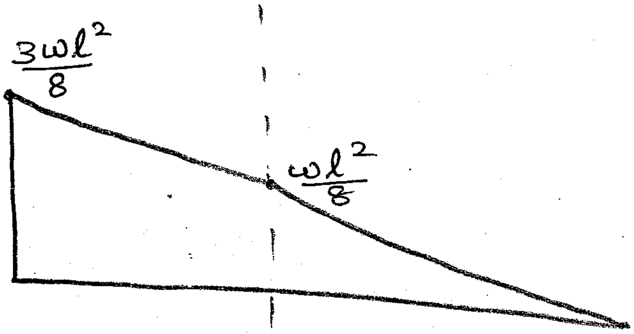
25/9/16

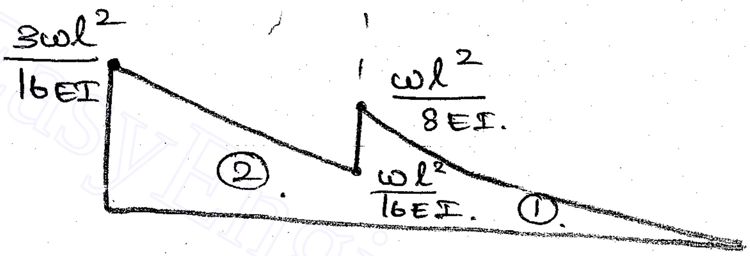
STRUCTURAL ANALYSIS.

1. Find θ_{max} and y_{max} for the beam:

(Mohr's Method)







$$\theta = A_1 + A_2.$$

$$= \frac{wl^3}{48EI} + \frac{wl^3}{16EI}.$$

$$= \frac{4wl^3}{48EI}$$

$$\boxed{\theta = \frac{wl^3}{12EI}}$$

$$A_1 = \frac{1}{3}\ \text{Arect.}$$

$$= \frac{1}{3} \times \frac{l}{2} \times \frac{wl^2}{8EI}$$

$$= \frac{wL^3}{48EI}.$$

$$A_2 = \frac{4wl^2}{16EI} \times \frac{l}{4}.$$

$$= \frac{wl^3}{16EI}.$$

$$\bar{x}_1 = \frac{3l}{4\times2} = \frac{3l}{8}.$$

$$\bar{x}_2 = \frac{l}{2} + \left(\frac{\frac{wl^2}{16EI} + \frac{6wl^2}{16EI}}{\frac{4wl^2}{16EI}}\right)$$

$$= \frac{l}{2} + \frac{7}{4} \times \frac{l}{6}.$$

$$= \frac{l}{2} + \frac{7}{24}l.$$

$$\boxed{\bar{x}_2 = \frac{19}{24}l.}$$

$$y = A_1\bar{x}_1 + A_2\bar{x}_2$$

$$= \left[\frac{wl^3}{\cancel{48}EI_{16}} \times \frac{3l}{8}\right] + \left[\frac{wl^3}{16EI} \times \frac{19l}{24}\right]$$

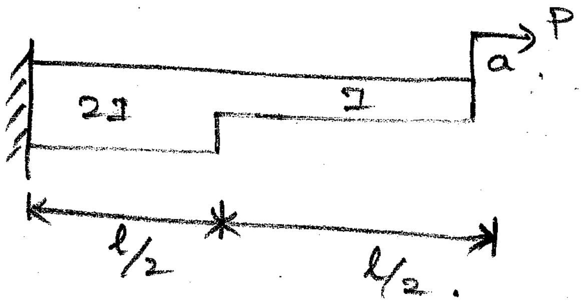
$$= \frac{wl^4}{128EI} + \frac{19wl^4}{384EI}$$

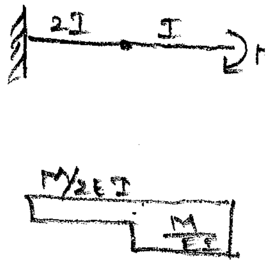
$$= \frac{3wl^4 + 19wl^4}{384EI}$$

$$\boxed{y = \frac{22wl^4}{384EI}} \qquad \boxed{y = \frac{11wl^4}{192EI}}$$

4/9/2015

1.) Determine slope and Deflection of the beam given below.





θ:

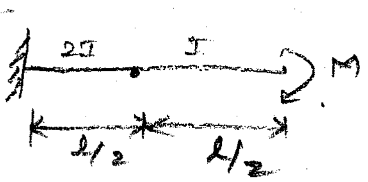
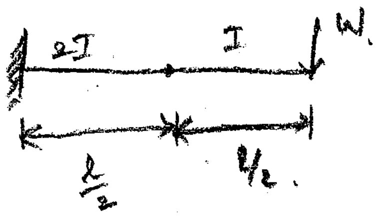
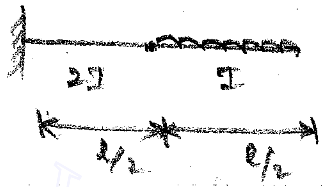
Area of B.M diagram $= \dfrac{M \times l}{4EI} + \dfrac{M \times l}{2EI}$.

$$\boxed{\theta = \frac{3Ml}{4EI}}$$

y:

Slope of beam $= \left[\dfrac{Ml}{4EI} \times \left(\dfrac{l}{4} + \dfrac{l}{2}\right)\right] + \left[\dfrac{Ml}{2EI} \times \dfrac{l}{4}\right]$.

$$\boxed{y = \frac{5Ml^2}{16EI}.}$$

		θ	y
1.)		$\frac{3ML}{4EI}$	$\frac{5ML^2}{16EI}$
2.)		$\frac{5wl^2}{16EI}$	$\frac{wl^3}{16EI}$
3.)		$\frac{wl^3}{12EI}$	$\frac{11wl^4}{192EI}$

STRUCTURAL ANALYSIS

→ Analysis of statically determinate trusses, arches, beams, cables and frames

→ Displacement in statically determinate structures.

→ Analysis of statically indeterminate structures by force / energy method.

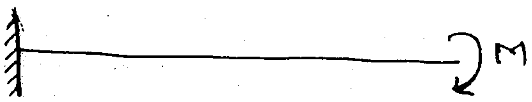
→ Analysis by displacement method (slope deflection and moment distribution method)

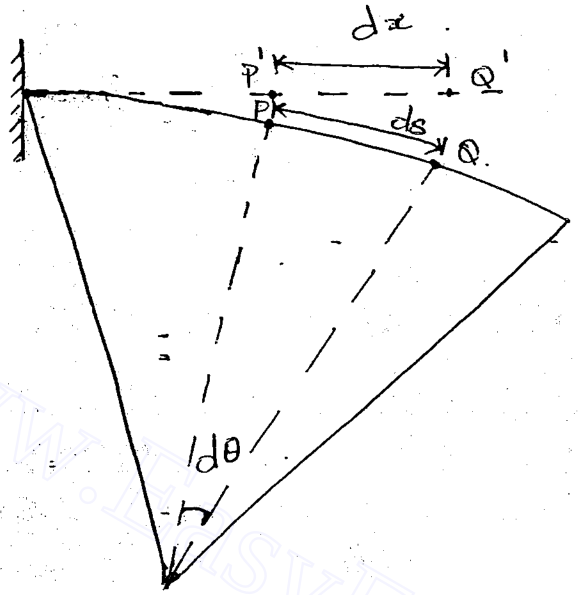
→ Influence lines for determinate and indeterminate structures.

→ Basic concept of matrix method of structural analysis.

2|10|2015

DOUBLE INTEGRATION:





$$\theta = \frac{\text{Arc length}}{\text{Radius of Curvature}}.$$

$$d\theta = \frac{ds}{R}. \qquad \boxed{ds \simeq dx.}$$

$$d\theta = \frac{dx}{R} \qquad \boxed{\theta = \frac{dy}{dx}}$$

$$d\left(\frac{dy}{dx}\right) = \frac{dx}{R}.$$

$$\frac{d^2y}{dx^2} = \frac{1}{R} \quad \text{--- (1)}.$$

According to Bending Theory.

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}.$$

$$\frac{1}{R} = \frac{M_{xx}}{EI}. \quad \text{--- (2)}$$

Substitute (1) in (2)

$$\frac{d^2y}{dx^2} = \frac{M_{xx}}{EI}.$$

$$\boxed{EI\frac{d^2y}{dx^2} = M_{xx}}$$

Integrating above equation.

$$EI\frac{dy}{dx} = \int M_{xx}$$

$$\boxed{\theta = \frac{dy}{dx} = \int \frac{M_{xx}}{EI}}.$$

Integrating above equation.

$$\boxed{y = \iint \frac{M_{xx}}{EI}}$$

$$\boxed{EI\frac{d^2y}{dx^2} = M_{xx}}$$

Differentiating above equation.

$$\boxed{E.I\frac{d^3y}{dx^3} = \frac{dM_{xx}}{dx} = \text{Shear Force.}}$$

Differentiating above equation.

$$\boxed{EI\frac{d^4y}{dx^4} = \frac{dv}{dx} = \text{Load}}$$

Note:

* If B.M is -ve (i.e) hogging then the curvature is +ve. but if B.M is +ve. (sagging) then the curvature is -ve. And hence correction is applied to above equation.

$$\boxed{EI \frac{d^2y}{dx^2} = -M_{xx}}$$

Note:

* At fully rigid end the product of EI is ~~inf~~ infinity.

$$\therefore \left| \frac{1}{EI} = \frac{1}{\infty} = 0 \right|$$

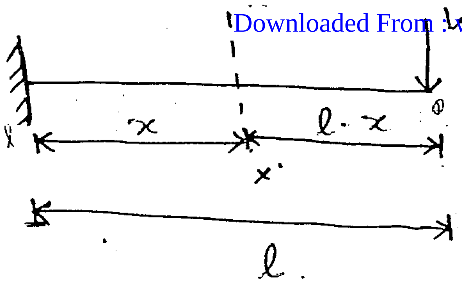
and hence θ and y is zero @ fixed end.

* At simple support. δ is maximum @ mid span but $\theta = 0$ @ mid span where as @ support $\theta = \theta_{max}$, $\delta = 0$.

* Cantilever with one point load @ free end max. deflection will be at the free end.

* cantilever with one point load any where in the beam δ_{max} of elastic curve will be under point load. Beyond point load deflection with vary linearly.

1.)



$$EI\frac{d^2y}{dx^2} = -M_{xx}.$$

$$\boxed{M_{xx} = -w(l-x)}$$

$$EI\frac{d^2y}{dx^2} = -[-w(l-x)]$$

$$EI\frac{d^2y}{dx^2} = w(l-x) \quad \text{--- (1)}$$

Integrating equation (1).

$$\int EI\frac{d^2y}{dx^2} = \int w(l-x)$$

$$EI\frac{dy}{dx} = w\left(lx - \frac{x^2}{2}\right) + C_1.$$

@ when $x = 0$ $\frac{dy}{dx} = 0$.

$$\boxed{C_1 = 0.}$$

$$\therefore \frac{dy}{dx} = \frac{w}{EI}\left(lx - \frac{x^2}{2}\right)$$

For max, slope. $x = l$.

$$\frac{dy}{dx} = \frac{w}{EI}\left(l^2 - \frac{l^2}{2}\right).$$

$$\boxed{\theta_{max} = \frac{dy}{dx} = \frac{wl^2}{2EI}}$$

$$EI \frac{dy}{dx} = \omega\left(xl - \frac{x^2}{2}\right) \quad — (2)$$

Integrating equ (2).

$$\int EI \frac{dy}{dx} = \int \omega \left(lx - \frac{x^2}{2}\right)$$

$$EI\, y = \omega\left(\frac{lx^2}{2} - \frac{x^3}{6}\right) + C_2$$

when $x = 0$ $y = 0$.

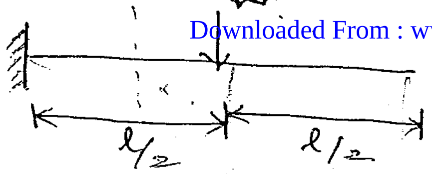
$$\cancel{y} \cancel{EI} = \frac{\cancel{\omega}}{EI} \qquad \boxed{C_2 = 0}$$

For y_{max} put $x = l$.

$$y = \frac{\omega}{EI}\left(\frac{l^3}{2} - \frac{l^3}{6}\right)$$

$$\boxed{y_{max} = \frac{\omega l^3}{3EI}}$$

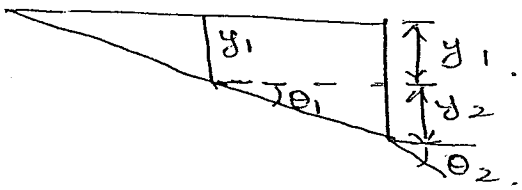
2.)

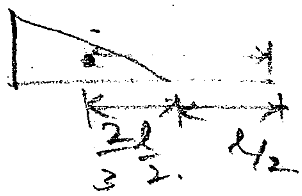


$$\frac{1}{2}\times\frac{wl}{2}\times\frac{l}{2}\times\left[\frac{2l}{3\times 2}+\frac{l}{2}\right]$$

$$\frac{wl^2}{8EI}\left[\frac{5l}{6}\right]$$

$$\frac{5\,wl^3}{48\,EI}$$





$$\theta_1 = \frac{wa^2}{2EI}$$

$$= \frac{w\,(l/2)^2}{2EI}$$

$$\boxed{\theta_1 = \frac{wl^2}{8EI}}$$

$$\boxed{\theta_1 = \theta_2 = \frac{wl^2}{8EI}}$$

$$y_1 = \frac{wa^3}{3EI} = \frac{w\,(l/2)^3}{3EI} = \frac{wl^3}{24EI}$$

$$\theta_1 = \frac{y_2}{l/2}$$

$$\frac{wl^2}{8EI} = \frac{y_2}{l/2}$$

$$\boxed{y_2 = \frac{wl^3}{16EI}}$$

4,2,4
2 | 6,4
3,2

$$y_{max} = y_1 + y_2 = \frac{wl^3}{24EI} + \frac{wl^3}{16EI}$$

$$= \frac{2wl^3 + 3wl^3}{48EI}$$

$$\boxed{y_{max} = \frac{5wl^3}{48EI}}$$

3.) A cantilever of span "l" subjected to UDL "ω" un yielding support is provided at free end. Determine

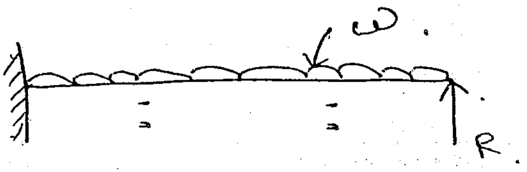
(i) Prop reaction.

(ii) Max. hogging. B.M.

(iii) Max. sagging B.M.

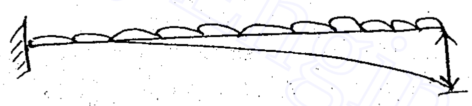
(iv) Position of sagging B.M.

(v) Point of contraflexure.



$$\cancel{R} \times \cancel{l} = \frac{\cancel{\omega} l^{\cancel{2}}}{2}$$

$$\boxed{\cancel{R} = \frac{\cancel{\omega} l}{2}}$$



$$y_1 = \frac{\omega l^4}{8EI}$$

$$y_2 = \frac{Rl^3}{3EI}$$

$$y_1 - y_2 = 0$$

$$\frac{Rl^3}{3EI} - \frac{\omega l^4}{8EI} = 0$$

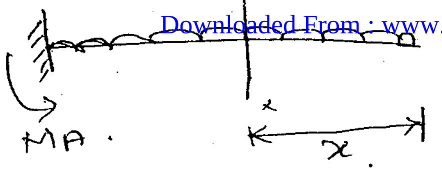
$$\frac{Rl^3}{3EI} = \frac{\omega l^4}{8EI}$$

$$\boxed{R = \frac{3\omega l}{8}}$$

Max. hogging B.M.

$$= R \times l - \frac{\omega l^2}{2}$$

$$= \frac{3\omega l^2}{8} - \frac{\omega l^2}{2} = -\frac{\omega l^2}{2}$$



$R_A \times x - M_A - \frac{wx^2}{2} = 0$

$\frac{5wl}{8}x - \frac{wl^2}{2} - \frac{wx}{2}$

Point of Contra flexure.

$$R \times x - \frac{wx^2}{2} = 0.$$

$$\frac{3wl}{8}x = \frac{wx^2}{2}.$$

$$\boxed{x = \frac{3l}{4}}$$

Max. saggin B.M.

B.M di

$$V_{xx} = 0.$$

$$R - wx = 0.$$

$$\frac{3wl}{8} = wx.$$

$$\boxed{x = \frac{3l}{8}}$$

Max. sagging B.M @ $x = \frac{3l}{8}$.

$$M = R \times x - \frac{wx^2}{2}$$

$$= \frac{3wl}{8}x - \frac{wx^2}{2}$$

$$= \left(\frac{3wl}{8} \times \frac{3l}{8}\right) - \frac{w}{2}\left(\frac{3l}{8}\right)^2.$$

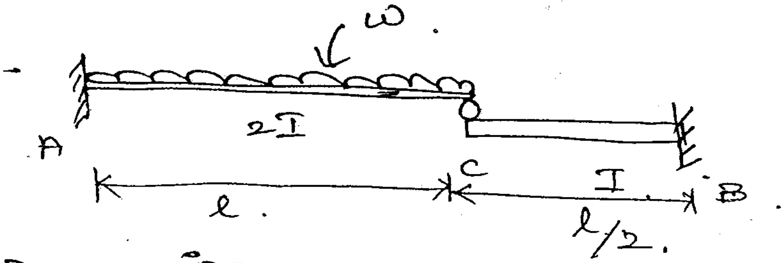
$$= \frac{9wl^2}{64} - \frac{9wl^2}{128}.$$

Max. Sagging B.M. $= \frac{9wl^2}{128}.$

29/10/2015

STRUCTURAL ANALYSIS.

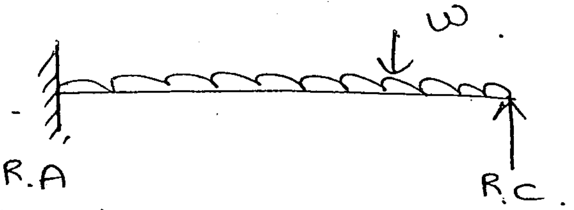
10).

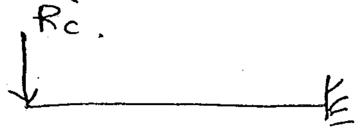


$R_c = \frac{3}{10} \omega l$

$R_A = \frac{7}{10}$

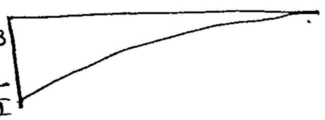
Determine S.F @ hinge Deflection @ hinge support reaction and support B.M.





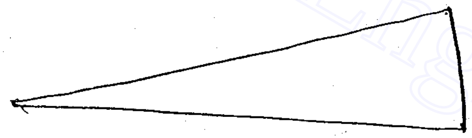
Due to U.D.L.

$\frac{\omega l^4}{16EI}$



$\frac{R_c l^3}{3EI}$

Due to point load.



$\frac{R_c l^3}{6EI}$

$\delta_{net} = \frac{R_c l^3}{3EI}$

$$\delta_{net} = \frac{\omega l^4}{16EI} - \frac{R_c l^3}{6EI} \quad (1) \qquad = \frac{R_c l^3}{24EI} \quad (2)$$

Equating (1) and (2)

$$\frac{\omega l^4}{16EI} - \frac{R_c l^3}{3EI} = \frac{R_c l^3}{3EI}$$

3ω

$$\frac{\omega l^4}{16EI} = \frac{2 R_c l^3}{3EI}$$

$\frac{2 R_c l^3}{6EI}$ $\frac{3}{16 \times 2}$

$$\boxed{R_c = \frac{\omega l}{8}}$$

$\frac{R_c l^3}{3EI}$ $\frac{\omega l}{8} - \omega l$

$\frac{7\omega l}{8}$

$$\frac{wl^4}{16EI} - \frac{R_c l^3}{6EI} = \frac{R_c l^3}{24EI}.$$

$$\frac{wl^4}{16EI} = \frac{R_c l^3}{24EI} + \frac{R_c l^3}{6EI}.$$

$$\frac{wl^{\cancel{4}}}{16\cancel{EI}} = \frac{5R_c \cancel{l^3}}{24\cancel{EI}}.$$

$$R_c = \frac{\cancel{24}\,wl}{16\times 5}.$$

$$= \frac{6\,wl}{4\times 5}.$$

$$\boxed{R_c = \frac{3wl}{10}}$$

$\Sigma V = 0$.

$$R_A + R_c = wl.$$

$$R_A = wl - \frac{3wl}{10}.$$

$$\boxed{R_A = \frac{7wl}{10}}$$

$$M_A = \frac{3wl}{10}\times l - \frac{w\times l^2}{2}.$$

$$= \frac{3wl^2}{10} - \frac{wl^2}{2}.$$

$$= -\frac{2wl^2}{10}.$$

$$\boxed{M_A = \frac{-wl^2}{5}}$$

$$M_B = R_c \times l/2.$$

$$= \frac{3wl}{10}\times\frac{l}{2}.$$

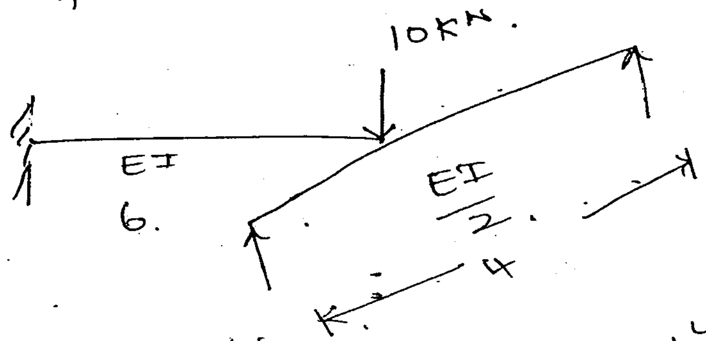
$$\boxed{M_B = \frac{3wl^2}{20}}$$

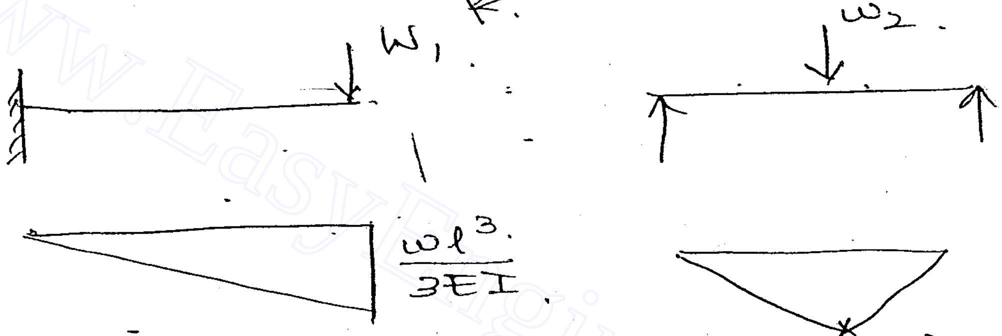
$$\delta = \frac{R_c l^3}{24EI}.$$

$$= \frac{3wl \times l^3}{10\times \cancel{24}_{8} EI}$$

$$\boxed{\delta_{net} = \frac{3wl^4}{80EI}}$$

2.) A cantilever beam. $l = 6m$ ~~ss~~ is supported by s.s Beam in transverse direction @. free end is @ the mid span. of s.s beam. Point load of 10kN is applied at the junction point. Determine support reaction. s.s span = 4m. Assume EI of ~~Co~~ s.s is half of EI of cantilever.





$$= \frac{\omega l^3}{48 \cdot \frac{EI}{2}}$$

$$= \frac{\omega l^3}{24EI}$$

$$\cancel{\delta_1 = \delta_2}$$

$$\cancel{\frac{\omega l^3}{24EI} = \frac{\omega l^3}{3EI}}$$

$$\cancel{\omega_1 = \frac{3\omega_2}{24}}$$

$$\delta_1 = \delta_2$$

$$\frac{\omega_1 l^3}{3EI} = \frac{\omega_2 l^3}{24EI}$$

$$\frac{\omega_1 \times 6^3}{3} = \frac{\omega_2 (4)^3}{24}$$

$$\omega_1 = \frac{3}{6^3} \times \frac{4^3}{24} \times \omega_2$$

$$\omega_1 = \frac{\cancel{64}^{8} \times \cancel{8}}{\cancel{24}_{8} \times 216} \omega_2$$

$$\boxed{\omega_1 = \frac{\omega_2}{27}}$$

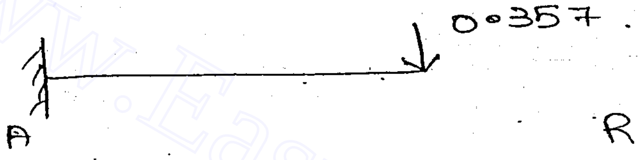
$$\omega_1 + \omega_2 = 10$$

$$\frac{\omega_2}{27} + \omega_2 = 10$$

$$\frac{28\,\omega_2}{27} = 10$$

$$\omega_2 = 9.643 \text{ kN}$$

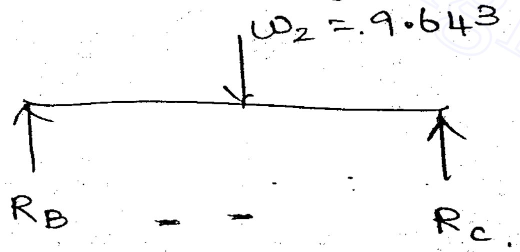
$$\omega_1 = 0.357 \text{ kN}$$



$$\Sigma V = 0$$

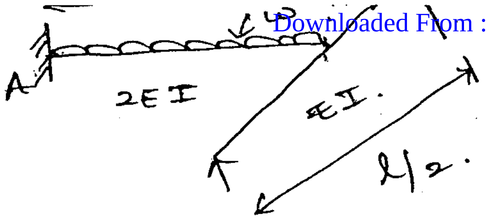
$$R_A - 0.357 = 0$$

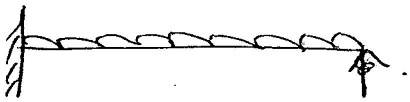
$$\boxed{R_A = 0.357}$$

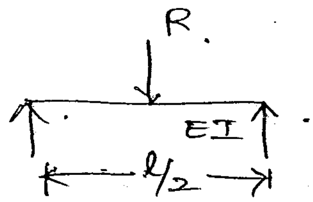


$$R_B = R_C = \frac{9.643}{2}$$

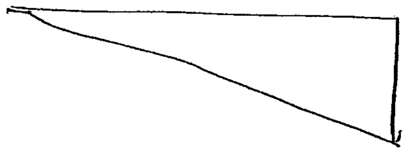
$$\boxed{R_B = R_C = 4.8215 \text{ kN}}$$

3.)  Determine support reaction and support B.M.

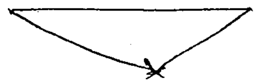




Due to UDL.

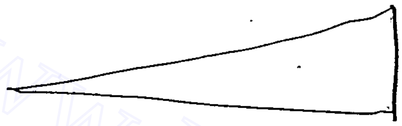


$\frac{wl^4}{8\times 2EI}$.



$y = \frac{R\left(\frac{l}{2}\right)^3}{48EI}$

Due to point load.



$\frac{Rl^3}{3\times 2EI}$.

$y = \frac{Rl^3}{384EI}$.

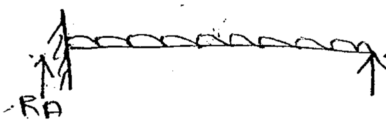
$$\frac{wl^4}{16EI} - \frac{Rl^3}{6EI} = \frac{Rl^3}{384EI}.$$

$$\frac{wl^4}{16EI} = \frac{Rl^3}{384EI} + \frac{Rl^3}{6EI}.$$

$$\frac{wl^{\cancel{4}}}{16\cancel{EI}} = \frac{65R\cancel{l^3}}{384\cancel{EI}}$$

$$R = \frac{wl\times 384}{16\times 65}.$$

$$\boxed{R = \frac{24wl}{65}}$$



R_A $\quad R = \frac{24wl}{65}$.

$\Sigma V = 0$.

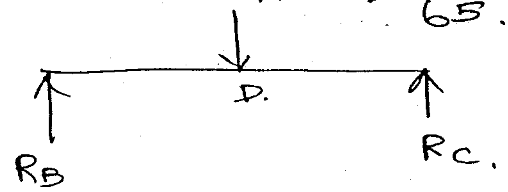
$R_A = wl - \frac{24wl}{65}$.

$R_A = \frac{41wl}{65}$

$M_A = \frac{24wl}{65}\times l - \frac{wl^2}{2}$.

$= \frac{24wl^2}{65} - \frac{wl^2}{2}$.

$$\boxed{M_A = \frac{-17wl^2}{130EI}}$$



$$R_B = R_C = \frac{24\omega l}{65\times 2} = \frac{24\omega l}{130}.$$

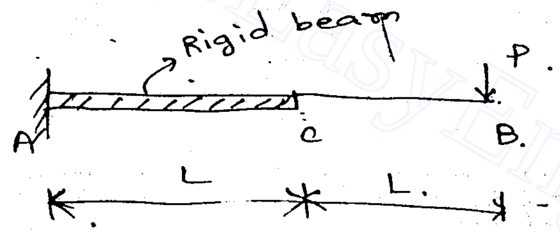
$$M_D = R_B \times \frac{l}{4}.$$

$$= \frac{\cancel{24}^{6}\,\omega l}{130} \times \frac{l}{\cancel{4}}.$$

$$= \frac{6\omega l^2}{130}.$$

$$\boxed{M_D = \frac{3\omega l^2}{65}}$$

4)



In rigid beam deflection is infinity.

$$y = \frac{Pl^3}{3}.$$

$$B.M. @ A = P\times(L+L)$$

$$= P\times 2L$$

$$B.M @ A = 2PL.$$

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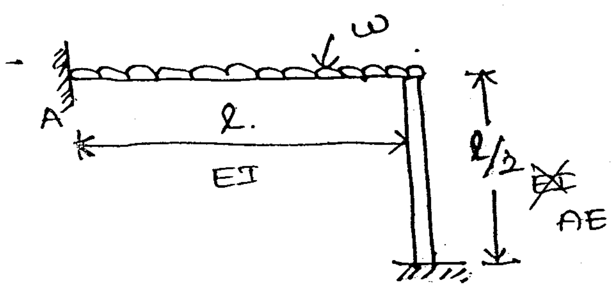
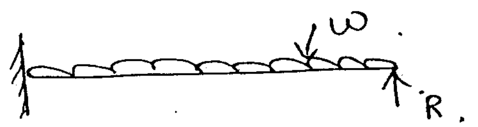
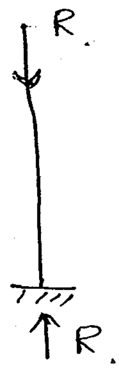
STRUCTURAL ANALYSIS.

1.) Determine support reactions and support moment.



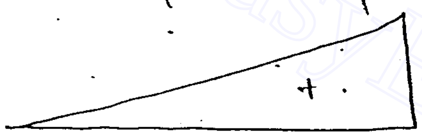




Due to U.D.L

$\delta_1 = \dfrac{wl^4}{8EI}$

Due to point load



$\delta_2 = \dfrac{Rl^3}{3EI}$

$\delta_{net} = \dfrac{Rl}{AE}$

$= \dfrac{R(l/2)}{AE}$

$\delta_{net} = \dfrac{Rl}{2AE}$

$$\delta_{net} = \delta_2 - \delta_1$$

$$= -\frac{Rl^3}{3EI} + \frac{wl^4}{8EI}$$

$$-\frac{Rl^3}{3EI} + \frac{wl^4}{8EI} = \frac{Rl}{2AE}$$

$$\frac{Rl^3}{3EI} + \frac{Rl}{2AE} = \frac{wl^4}{8EI}$$

$$\frac{2ARl^3 + 3IRl}{6EAI} = \frac{wl^4}{8EI}$$

$$R\left(\frac{2Al^3 + 3Il}{6EAI}\right) = \frac{wl^4}{8EI}$$

$$\therefore R = \frac{wl^4}{8\cancel{E}\cancel{I}} \times \frac{6\cancel{E}A\cancel{I}}{(2Al^3 + 3I)l}$$

$$\boxed{R = \frac{3Aw l^3}{4(2Al^2 + 3I)}}$$

$$M_A = R \times l - \frac{wl^2}{2}$$

$$= \frac{3Awl^3 \times l}{4(2Al^2+3I)} - \frac{wl^2}{2}$$

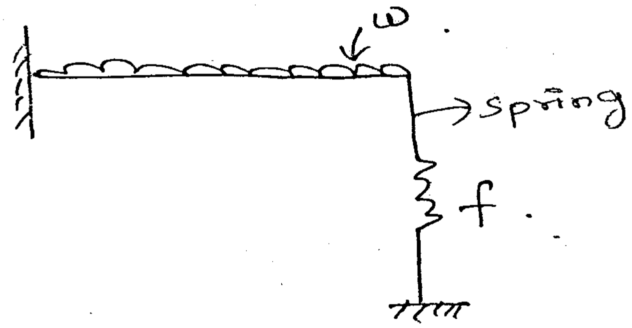
$$= \frac{3Awl^4}{4(2Al^2+3I)} - \frac{wl^2(2Al^2+3I)2}{4(2Al^2+3I)}$$

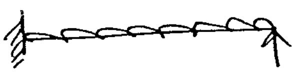
$$= \frac{3Awl^4 - 4Awl^4 - 6wl^2 I}{4(2Al^2+3I)}$$

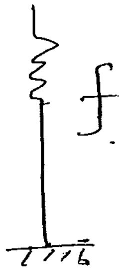
$$= \frac{-Awl^4 - 6wl^2 I}{4(2Al^2+3I)}$$

$$\boxed{M_A = -\left[\frac{Awl^4 + 6wl^2 I}{\cancel{4(2}\,8Al^2 + 12I}\right]}$$

2.) Analysis a cantilever beam supported by spring @ free end. Flexibility f, stiffness K. Determine reaction:





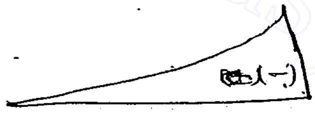


Due to U.D.L

$$\delta_1 = \frac{\omega l^4}{8EI}$$

$$f = \frac{\text{Displacement}}{\text{force}}$$

Due to point load



$$\delta_2 = \frac{Rl^3}{3EI}$$

Displacement $= f \times$

$\delta_{net} = f \times R$

$$\delta_{net} = \delta_1 - \delta_2$$

$$= \frac{\omega l^4}{8EI} - \frac{Rl^3}{3EI}$$

$$\therefore \frac{\omega l^4}{8EI} - \frac{Rl^3}{3EI} = f \times R$$

$$R\left[\frac{l^3}{3EI} + f\right] = \frac{\omega l^4}{8EI}$$

$$R\left[\frac{l^3 + 3fEI}{3EI}\right] = \frac{\omega l^4}{8EI}$$

$$R = \frac{\omega l^4}{8EI} \times \frac{3EI}{(l^3 + 3fEI)}$$

$$\boxed{R = \frac{\frac{\omega l^4}{8EI}}{f + \frac{l^3}{3EI}}}$$

$$K = \frac{R}{\delta_{net}}$$

$$\boxed{\delta_{net} = \frac{R}{K}}$$

$$\frac{wl^4}{8EI} - \frac{Rl^3}{3EI} = \frac{R}{K}$$

$$\frac{R}{k} + \frac{Rl^3}{3EI} = \frac{wl^4}{8EI}$$

$$R\left[\frac{1}{k} + \frac{l^3}{3EI}\right] = \frac{wl^4}{8EI}$$

$$R\left[\frac{3EI + kl^3}{3kEI}\right] = \frac{wl^4}{8EI}$$

$$R = \frac{wl^4}{8EI} \times \frac{3kEI}{(3EI + kl^3)}$$

$$\boxed{R = \frac{3kwl^4}{8(3EI + kl^3)}}$$

CONJUGATE BEAM METHOD

Conjugate beam is an imaginary beam has same span as the original beam subjected to load equal to $\frac{M}{EI}$ diagram. The support condition of conjugate beam may or may not same as that of the original beam.

There are 2 theorems:

Theorem 1:

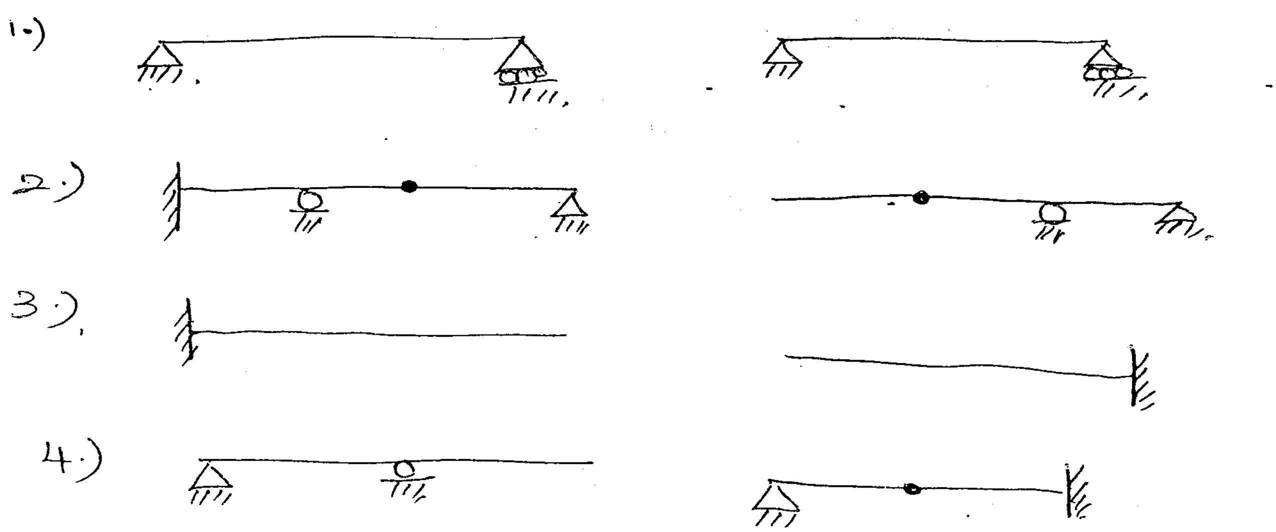
The slope @ any section of original beam is equal to the S.F of its conjugate beam @ the corresponding section. ($\theta_{xx} = V_{xx}$)

Theorem 2:

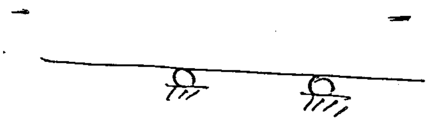
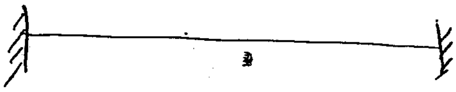
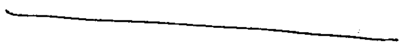
The deflection @ any section of an original beam is equal to the B.M of its conjugate beam @ the corresponding section. ($\delta_{xx} = M_{xx}$)

Support conditions:

Real beam — Conjugate Beam.



Support condition

	Real beam	Conjugate Beam.
5.)		
6.)		

21/11/2015

STRAIN ENERGY METHOD (Castigilliano's Theorem).

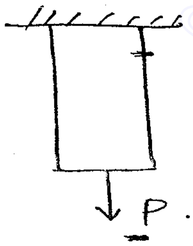
* Energy theorem is based on law of conservation of energy, which states that "External workdone on a structure due to displacement = Internal Energy stored"

Strain energy = External work done

External work done = Average Load x displacement

Strain Energy = $\frac{P}{2} \times \delta l$.

Strain Energy Due to Axially applied Load:



$U = EWD.$

$EWD = \begin{pmatrix}\text{Average} \\ \text{Load}\end{pmatrix} \times \text{displacement}$

$= \left(\frac{0+P}{2}\right) \times \delta l.$

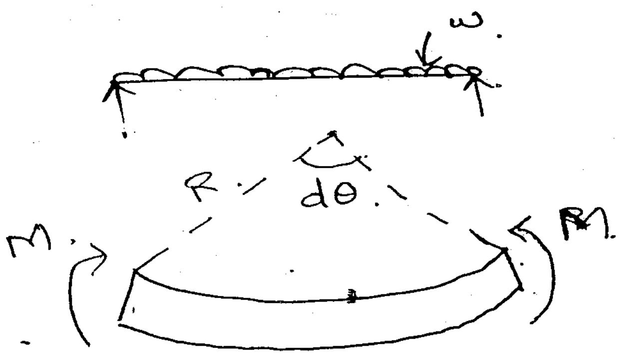
$= \frac{P}{2} \times \frac{Pl}{AE}.$

$= \frac{P^2 l}{2AE} \times \frac{A}{A}.$

$= \left(\frac{P}{A}\right)^2 \times \left(\frac{Al}{2E}\right)$

$$\boxed{\text{Strain Energy} = f^2 \times \frac{V}{2E}.}$$

Strain Energy Due to Moment.



$U = EWD.$

$= \text{Moment} \times \text{Rotation}$

$$du = \left(\frac{0+M}{2}\right) \times d\theta.$$

$$\boxed{du = \frac{M}{2}\, d\theta} \quad \text{——— (A)}.$$

Bending Theory.

$$\frac{M}{I} = \frac{E}{R}.$$

$$\boxed{\frac{1}{R} = \frac{M}{EI}} \quad \text{——— (1)}$$

$$d\theta = \frac{\text{Arc length}}{\text{Radius}} = \frac{ds}{R}.$$

$$\boxed{\frac{1}{R} = \frac{d\theta}{ds}} \quad \text{——— (2)}.$$

Equating (1) and (2).

$$\boxed{d\theta = \frac{M}{EI}\, ds.} \quad \text{——— (3)}.$$

Substitute (3) in (A).

$$du = \frac{M}{2} \times \frac{M}{EI}\, ds.$$

Strain energy due to Bending.

$$\boxed{U = \int \frac{M^2}{2EI}\, ds.}$$

Strain energy due to Shear,

$$U = \frac{\tau^2}{2GA} ds.$$

Strain energy due to Torsion.

$$U = \frac{T^2}{2GJ} ds.$$

Theorem 1:

The partial derivative of total strain energy of a loaded structure with respect to applied Load is equal to the deflection. δ under that Load.

where,

$$\delta = \frac{\partial U}{\partial P}.$$

The partial derivative of total strain energy with respect to applied moment is equal to the slope. in the direction of applied moment

$$\theta = \frac{\partial U}{\partial M}.$$

Theorem 2: [Least Energy Theorem]-

Total strain energy of a loaded Indeterminate structure is the minimum.

The partial derivative of total strain Energy of a loaded indeterminate structure with respect to reduntant force is zero.

$$\frac{\partial U}{\partial R} = 0.$$

$$\frac{\partial U}{\partial R_B} = 0 .$$

For a bending member G, C and N is given.

$$U_{total} = \int \frac{M^2}{2EI} . ds + \int \frac{F_s^2 \, ds}{2GA}$$

If G is not given.

$$U_{total} = \int \frac{M^2}{2EI} ds .$$

If I is very small and I is neglected.

$$U_{total} = \int \frac{M^2}{2EI} ds .$$

Strain Energy Method

* External workdone of a structure due to displacement } = Internal Energy Stored.

* Strain energy due to axially applied load

$$U = \frac{f^2}{2E} \times V.$$

* Strain energy due to moment.

$$U = \int \frac{M^2}{2EI} ds.$$

* Strain energy due to shear stress

$$U = \int \frac{\tau^2}{2G} ds$$

* Strain energy due to Torsion

$$U = \int \frac{T^2}{2GJ} . ds$$

Max Shear Strain

$$U = \frac{\tau_{max}^2 \times V}{4G}$$

Hollow circular Rod

$$U = \frac{\tau_{max}^2 \times V}{4G}\left(\frac{D^2+d^2}{D^2}\right)$$

$$U_{spring} = \frac{R^2 f}{2}$$

* Theorem 1: (i) $\frac{\partial U}{\partial P} = \delta$

Partial derivative of total strain energy of a loaded structure w.r.t applied load is equal to deflection under that load.

(ii) $\frac{\partial U}{\partial M} = \theta$

Partial derivative of total strain energy of a loaded structure w.r.t to moment is equal to slope in the direction of applied moment.

* Theorem 2: $\frac{\partial U}{\partial P} = 0$

Partial derivative of total strain energy of a loaded indeterminate structure w.r.t redundant force is equal to zero.

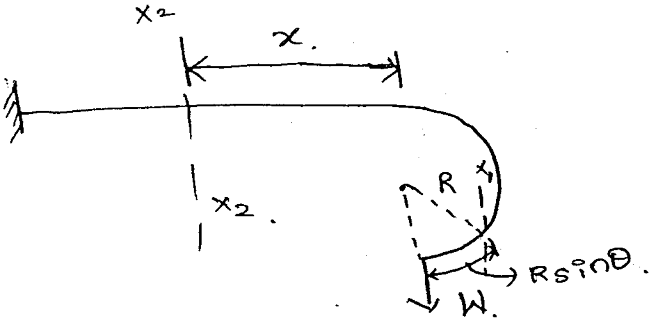
* For a given beam, $U_{total} = \int \frac{M^2}{2EI} ds + \int \frac{F_s^2}{2GA} ds$

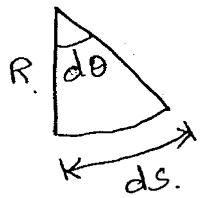
25/11/2015

1.)

– Find Deflection. –

$$\delta = \frac{W^2}{2EI}\left[\frac{R^3\pi}{2} + \frac{l^3}{3}\right]$$





$ds = R\,d\theta.$

$$M_{xx1} = WR\sin\theta.$$

$$M_{xx2} = Wx$$

$$U = \int_0^{\pi} \frac{M_{xx_1}^2\, ds}{2EI} + \int_0^{l} \frac{M_{xx_2}^2\, dx}{2EI}$$

$$= \int_0^{\pi} \frac{(WR\sin\theta)^2\, R\,d\theta}{2EI} + \int_0^{l} \frac{(Wx)^2\, dx}{2EI}$$

$$= \int_0^{\pi} \frac{W^2R^3\sin^2\theta}{2EI}\,d\theta + \int_0^{l} \frac{W^2x^2}{2EI}\,dx.$$

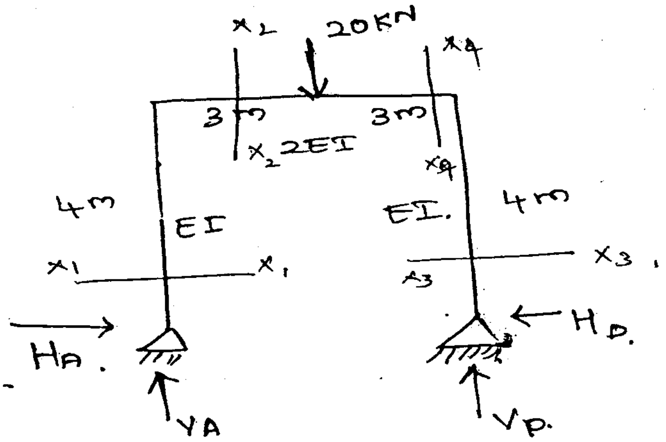
$$= \frac{W^2R^3}{2EI}\int_0^{\pi}\left(\frac{1-\cos 2\theta}{2}\right)d\theta + \frac{W^2}{2EI}\int_0^{l} x^2\,dx.$$

$$= \frac{W^2R^3}{4EI}\left[\theta - \frac{\sin 2\theta}{2}\right]_0^{\pi} + \frac{W^2}{2EI}\left[\frac{x^3}{3}\right]_0^{l}.$$

$$= \frac{W^2R^3}{4EI}\left[\pi - 0\right] + \frac{W^2}{6EI}\,l^3.$$

$$= \frac{W^2R^3\pi}{4EI} + \frac{W^2l^3}{6EI} = \frac{W^2}{2EI}\left[\frac{R^3\pi}{2} + \frac{l^3}{3}\right]$$

2.) Determine the horizontal Reaction:



		Limit
M_{xx1}	$= -H_A \times x$	0 - 4
M_{xx2}	$= (V_A \times x) - (H_A \times 4)$	0 - 3
M_{xx3}	$= -H_D \times x$	0 - 4
M_{xx4}	$= (V_D \times x) - (H_D \times 4)$	0 - 3

1.98

$$\frac{\partial U}{\partial R} = 0$$

$$-\int_0^4 \frac{H_A \times x}{2EI} \cdot dx + \int_0^3 \frac{V_A x - 4H_A}{4EI} \cdot dx - \int_0^4 \frac{H_D \times x}{2EI} dx$$

$$+ \int_0^3 \frac{V_D x - 4H}{4EI} \cdot dx = 0$$

$$\frac{-H_A}{2EI} \left(\frac{x^2}{2}\right)_0^4 + \frac{1}{4EI}\left[\frac{V_A x^2}{2} - 4H_A x\right]_0^3 - \left[\frac{x^2}{2}\right]_0^4 \times \frac{H_D}{2EI}$$

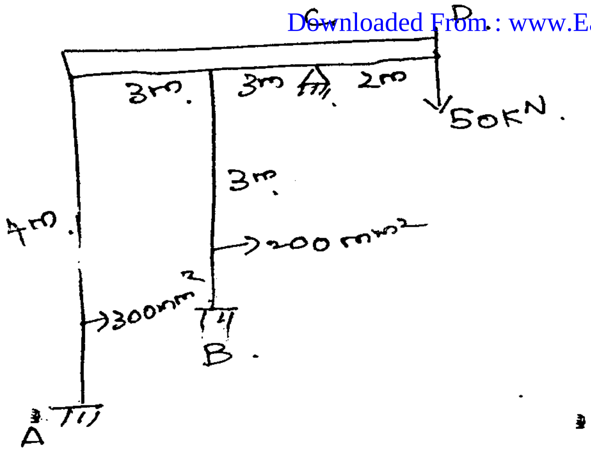
$$+ \frac{1}{4EI}\left[\frac{V_D x^2}{2} - 4Hx\right]_0^3 = 0$$

$$\frac{-16H_A}{4EI} + \frac{1}{4EI}\left[\frac{9V_A}{2} - 12H_A\right] - \frac{16H}{4EI} + \frac{1}{4EI}\left[\frac{9V_D}{2} - 12H\right] = 0$$

$$\frac{-32H}{4EI} + \frac{9V_A}{8EI} - \frac{12H_A}{4EI} + \frac{9V_D}{8EI} - \frac{12H}{4EI} = 0$$

$$-\frac{56H}{4EI} + \frac{18V_A}{8EI} = 0.$$

3)



$\Sigma V = 0.$

$R_A + R_B + R_C = 50.$

$\Sigma M_C = 0$

$(R_A \times 6) + (R_B \times 3) = -50 \times 2$

$6R_A + 3R_B = -100.$

$R_B = -\left(\frac{100 + 6R_A}{3}\right)$

$U_{rod} = U_{RA} + U_{RB}.$

$$= \frac{R_A^2 l}{2AE} + \frac{R_B^2 l}{2AE}.$$

$$= \frac{R_A^2 l}{2AE} + \frac{\left(\frac{100 + 6R_A}{3}\right)^2 l}{2AE}.$$

$$U = \frac{R_A^2 l}{2AE} + \left(\frac{100^2 + 1200R_A + 36R_A^2}{18AE}\right) l.$$

$$\frac{\partial U}{\partial R} = 0.$$

$$\frac{R_A^2}{2AE} \int_0^4 l\, dx + \int_0^3 l\, dx \left[\frac{100^2 + 1200R_A + 36R_A^2}{18AE}\right] = 0$$

$$\frac{R_A^2}{2AE}\left(\frac{l^2}{2}\right)_0^4 + \frac{10000 + 1200R_A + 36R_A^2}{18AE}\left(\frac{l^2}{2}\right)_0^3 = 0.$$

$$\frac{16R_A^2}{4AE} + \frac{(10000 + 1200R_A + 36R_A^2) \cdot 9}{36AE} = 0.$$

$$2 \times 1.44 R_A^2 + 3(90000 + 10800R_A + 324R_A^2) = 0.$$

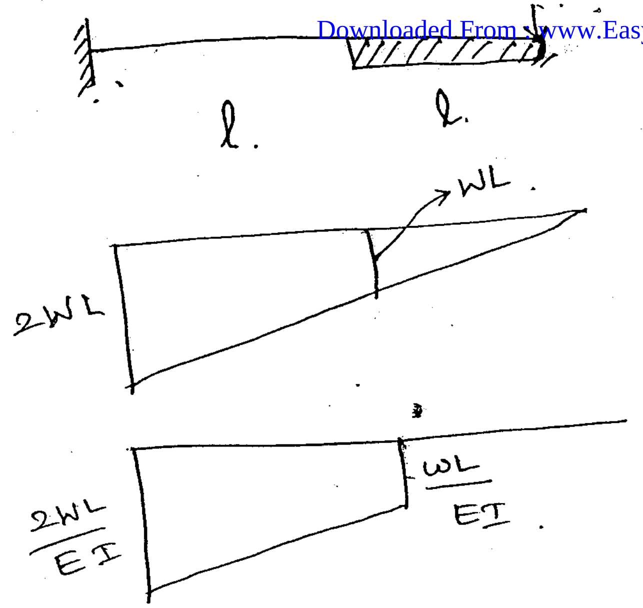
$$468R_A^2 + 10800R_A + 90000 = 0.$$

$R_A = -11.53.$ kN.

$288 R_A^2 + 270000 + 32400R_A + 972R_A^2 = 0.$

$1260 R_A^2 + 32400R_A + 270000 = 0.$

4.)



$A = \frac{h}{2}(a+b)$

$\bar{x} = \frac{h}{3}\left(\frac{a+2b}{a+b}\right)$

$$\delta_B = A \times \bar{x}$$

$$= \left(\frac{WL}{EI}(2+1) \times \frac{l}{2}\right) \times \left[\frac{l}{3} \cdot \left[\frac{\frac{5WL}{EI}}{\frac{3WL}{EI}}\right]\right]$$

$$= \frac{\cancel{3}WL^2}{2EI}\left[\frac{L}{\cancel{3}} \times \frac{5}{3}\right]$$

$$\boxed{\delta_B = \frac{5WL^3}{6EI}}$$

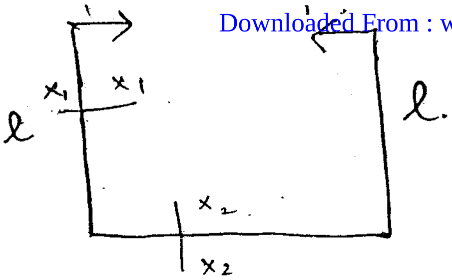
$$\delta_C = \frac{3WL}{EI} \cdot \frac{l}{2} \times \left[\frac{l}{3} \times \frac{5}{3} + l\right]$$

$$= \frac{3WL^2}{2EI} \times \left[\frac{5l}{9} + l\right]$$

$$= \frac{\cancel{3}\omega L^2}{\cancel{2}EI} \times \frac{\overset{7}{\cancel{14}}\, l}{\underset{3}{\cancel{9}}}$$

$$\boxed{\delta_C = \frac{7\omega L^3}{3EI}}$$

6.)



$$\delta = \frac{\partial U}{\partial P}$$

$M_{xx1} = Px$

$M_{xx2} = Pl$

$$= 2\int_0^l \frac{(Px)^2}{2EI}dx + \int_0^l \frac{(Pl)^2 dx}{2EI}$$

$$= \frac{2}{2}\left[\frac{P^2x^3}{3EI}\right]_0^l + \left[\frac{P^2l^2x}{2EI}\right]_0^l$$

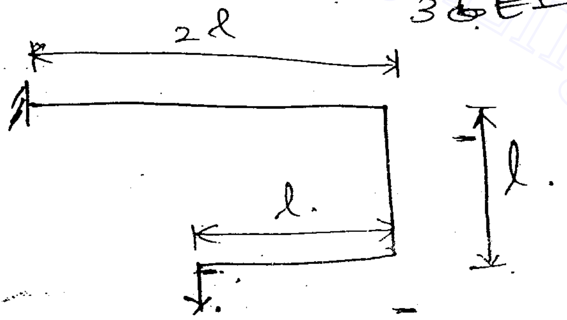
$$= \frac{P^2l^3}{3EI} + \frac{P^2l^3}{2EI}$$

$$U = \frac{5P^2l^3}{6EI}$$

$$\frac{\partial U}{\partial P} = \frac{10Pl^3}{6EI} = \frac{5Pl^3}{3EI}$$

$$\delta = \frac{5Pl^3}{3EI}$$

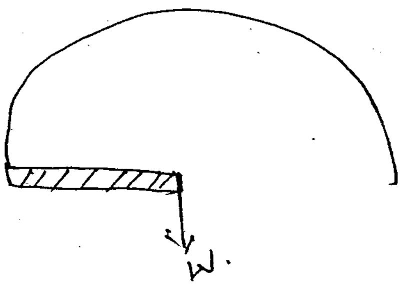
7.)



$$\delta_{vertical} = \frac{3}{2}\cdot\frac{Ml^2}{EI}$$

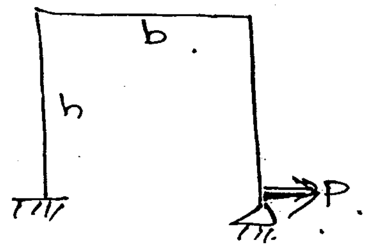
$$\theta = \frac{4Ml}{EI}$$

8.)



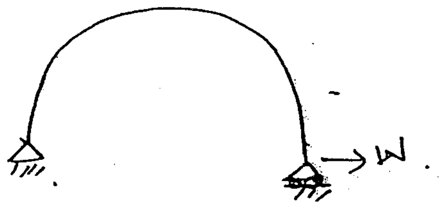
$$\delta = \frac{WR^3\pi}{2EI}$$

9.)



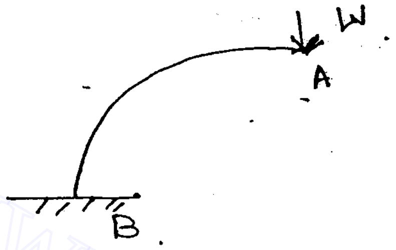
$$\delta_D = \frac{Ph^2(2h+3b)}{3EI}$$

10.)



$$\delta_B = \frac{WR^3\pi}{2EI}$$

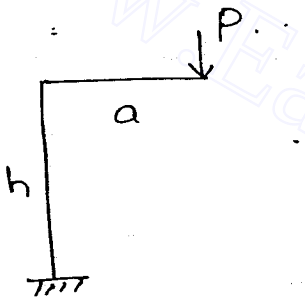
11.)



$$\delta_v \text{ @ } A = \frac{Wr^3\pi}{4EI}$$

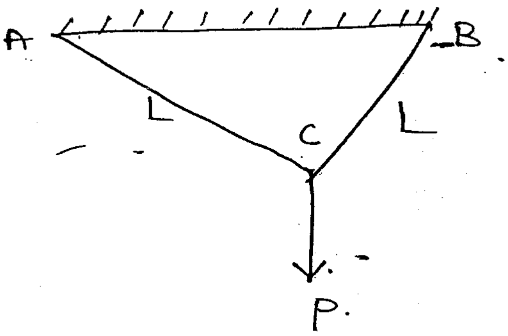
$$\delta_h \text{ @ } A = \frac{Wr^3}{2EI}$$

12.)



$$\delta_h = \frac{Pah^2}{2EI}$$

13.)



$$\delta_c = \frac{PL}{AE}$$

28/11/2015

STRUCTURAL ANALYSIS

COLUMNS.

* A structural member which is subjected to axial compressive load and the lateral ~~dimens~~ dimensions are very smaller than the longitudinal dimension then it is called a strut.
* A strut is a compression member.
* A struct which is truly vertical is known as a column or stanchion.
* Following are the examples of compression member are:
  · strut, column, Boom (cranes), web of an I-section @ a support, web of I-section under point load away from the support, thin narrow depth girder, thin shaft subjected to heavy torsion.

* Types.
  - Short column. $\lambda \leq \sqrt{\frac{\pi^2 E}{f_p}}$
  - Long column. $\lambda \geq \sqrt{\frac{\pi^2 E}{f_p}}$

* ~~Short~~ ~~Colt Column~~:

f_p - yield stress.

* Short column
  → Always fails in crushing where actual compressive stress exceeds yield stress.
* Long column.
  → Fails in buckling eventhough actual compressive stress may be less than yield stress.
* Buckling:
  The lateral deflection of compression member due to axially applied load.

* A long column may buckle due to following reasons.
  - → No columns is truely vertical.
  - → No load is purely axial.
  - → No column is homogenous throughout.
  - → Some occasional lateral load may occur.

$$\boxed{r = \sqrt{\frac{I}{A}}.}$$

$$\boxed{\begin{aligned} I &= A \times r \times r. \\ &= A \times r^2. \\ r &= \sqrt{I/A}. \end{aligned}}$$

* If a column section has one effective length; is both the direction. Then only r_{min} shall be considered.

  where $r_{min} = \sqrt{\frac{I_{min}}{A}}$.

* If column has two effective length the r_{min} shall not be considered

  In case individual radius of gyration shall be considered in either direction and then maximum slenderness ratio may be applied.

<u>Example:</u>

A cantilever provided with two lateral type.

A column having bolted connection at top and bottom;

* The slenderness ratio of l_{eff} of compression member to the radius of gyration.

1.) $$\boxed{\lambda = \frac{l_{eff}}{r_{min}}}$$

2.) $$\underbrace{\lambda_x = \frac{l_{ex}}{r_x}, \quad \lambda_y = \frac{l_{ey}}{r_y}}_{\text{Greater value.}}$$

* If a ~~column steel~~ compression member is safe in buckling then sure it will safe in crushing.

But if it safe in crushing then it may or may not be safe in buckling

(i.e.) If a column is designed as a long column, then it is safe as a short column also.

∴ In practise every column is designed as a long column.

* The effective length of column is independent on end condition only.

l_{eff} is the distance b/w zero B.M to zero B.M

* There are 2 methods to analysis the buckling load of a column.

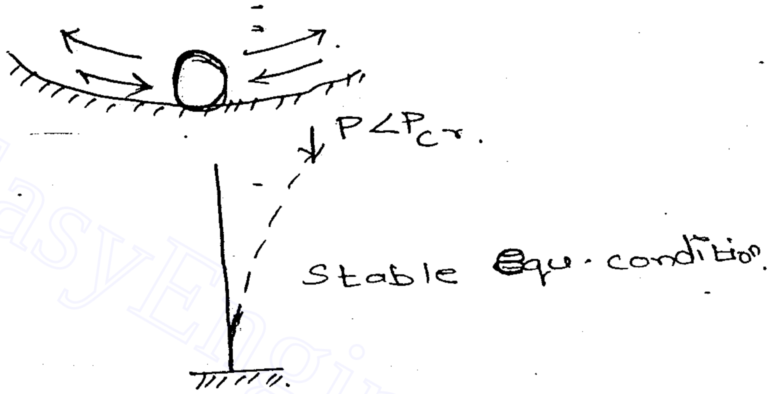
1. Euler's ~~eo~~ Theory's
2. Emphirical Formulae.

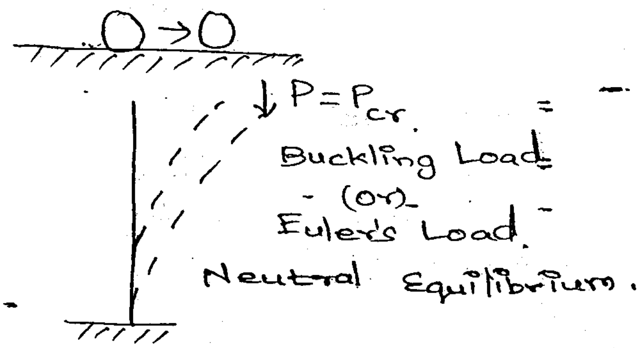
* ~~There are 5 Emphri...~~

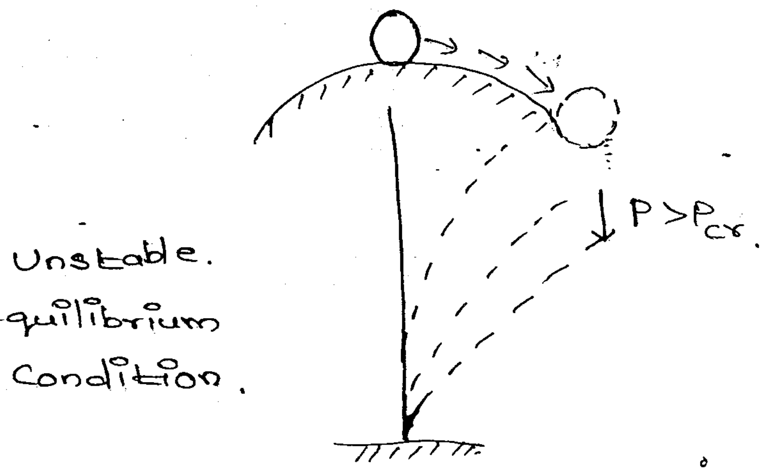
* There are 5 Emphrical Formulae.

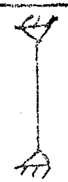
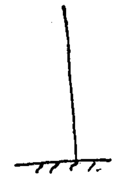
Rankine formula.
ISI Formula. (or) (Merchant Rankine)
Prof. Perry (Secant formula)
Straight Line formula
Parabolic formula

* Critical Load (or) Buckling Load (or) crippling Load is the load applied on a column @ neutral equilibrium condition.







End condition	Both end hinged. (held in position) and not restrained against rotation.	Both end fixed. (held in position) and restrained against rotation.	One end is fixed other end hinged. (Both ends held in position one end restrained against rotation)	One end is fixed. other end free. (one end held in position and ~~other~~ restrained against rotation and other end not held in position and not restrained against rotation).	one end fixed other end is guided roller. (one end held in position and restrained against rotation and other end not held in position but restrained against rotation)
Diagram					
~~Leff~~ l_{eff}	$l_{eff} = l$	$l_{eff} = \frac{l}{2}$	$l_{eff} = \frac{l}{\sqrt{2}}$	$l_{eff} = 2l$	

Downloaded From : www.EasyEngineering.net

Column: Subjected to axial and lateral dimension is smaller than length

Column (or) stanchion : truly vertical strut.

Column types

Pedestal
$\lambda < 3$

Short
$\lambda < 12$
$\lambda < \sqrt{\frac{\pi^2 E}{f_y}}$
Failure : crushing

Long
$\lambda \geq 12$
$\lambda > \sqrt{\frac{\pi^2 E}{f_y}}$
Failure Buckling (lateral deflection)

Radius of gyration

$$r = \sqrt{I/A}$$

Case (i) only one l_{eff}

$$\lambda = \sqrt{\frac{I_{min}}{A}}$$

Case (ii) More ~~only~~ than one l_{eff}

$$\lambda_x = \sqrt{\frac{I_{xx}}{A}} \qquad \lambda_y = \sqrt{\frac{I_{yy}}{A}}$$

λ = greater of λ_x (or) λ_y

l_{eff} * Depends on end condition * 0 B.M to 0 B.M

* All columns are designed as long column in general.

Methods to design

Euler

Assumptions
* long column
* Fails by buckling
* Initially straight
* Axial load applied
* loaded upto Elastic limit
* Both ends are pinned

Emphirical
- Rankine $P = \frac{f_c A}{1 + \alpha \lambda^2}$
- ISI (Merchant Rankine formula)
- Prof Perry (Secant Formula)
- Straight line
- Parabolic

Case (i) $l_e = l$ $\quad P_e = \frac{\pi^2 EI}{l^2}$

Case (ii) $l_e = \frac{l}{2}$ $\quad P_e = \frac{4\pi^2 EI}{l^2}$

Case (iii) $l_e = 2l$ $\quad P_e = \frac{\pi^2 EI}{4l^2}$

Case (iv) $l_e = \frac{l}{\sqrt{2}}$ $\quad P_e = \frac{2\pi^2 EI}{l^2}$

Critical (or) crippling (or) crushing load : load ...

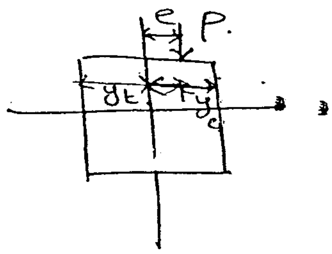
Neutral equilibrium condition

Equilibrium condition

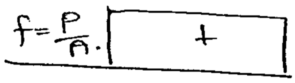
Stable $\qquad$ Neutral $\qquad$ Unstable

CORE (or) KERNEL OF THE SECTION:

* Core is the central zone of column where Load is applied then tension doesnot takes place anywhere in the column.

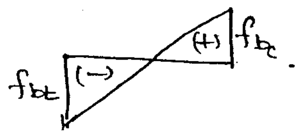


$$\frac{P}{A} - \frac{M}{Z} = 0.$$



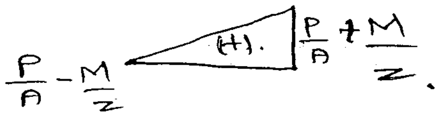
$$\frac{P}{A} - \frac{Pe}{Z} = 0.$$

$$P\left[\frac{1}{A} - \frac{e}{Z}\right) = 0.$$



$$\frac{1}{A} = \frac{e}{I}y.$$

$$\boxed{e = \frac{I}{Ay}}$$



Solid circular.

$$e = \frac{I}{Ay_t} = \frac{\frac{\pi}{64}D^4}{\frac{\pi}{4}D^2 \times \frac{D}{2}} = \frac{D}{8} \qquad e=\frac{D}{8}.$$

$$\text{DIA OF CORE} = \frac{D}{4}.$$

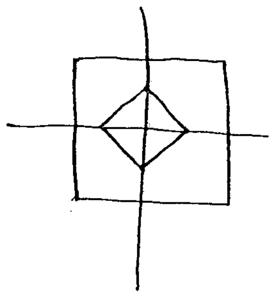
Hollow circular.

$$e = \frac{I}{Ay_t} = \frac{\frac{\pi}{64}\left(D^4 - d^4\right)}{\frac{\pi}{4}\left(D^2 - d^2\right) \times \frac{D}{2}} = \frac{D^2+d^2}{8}.$$

$$\text{DIA OF CORE} = \frac{D^2+d^2}{4}.$$

Square.

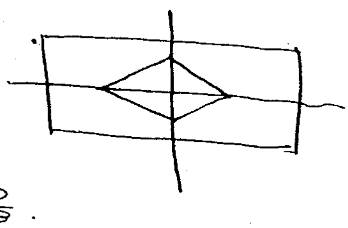
$$e = \frac{I}{Ay_t} = \frac{\frac{b^4}{12}}{b^2 \times b/2} = \frac{b}{6}.$$



$$\text{DIA OF CORE} = \frac{b}{3}.$$

Rectangle.

$$e_x = \frac{d}{6} \qquad e_y = \frac{b}{6}.$$



DIA OF CORE along $x = \frac{d}{3}$ along $y = \frac{b}{3}$.

$a^4 - b^4 = (a^2)^2 - (b^2)^2$

$(a^2 - b^2)(a^2 + b^2)$

$$\frac{\frac{\pi}{64}(D^2+d^2)(D^2-d^2)}{\frac{\pi}{4}(D^2-d^2)\frac{D}{2}}$$

$$\frac{D^2+d^2}{8}$$

$$\frac{P}{A}\left(\frac{1}{n} - \frac{e_y}{\frac{I_{xx}}{D/2}}\right) = 0.$$

$$e_y = \frac{2I_{xx}}{A.D.}$$

$$e_x = \frac{2I_{yy}}{Ab_f}$$

Core (or) kernel of the section

$$e = \frac{I}{Ay}$$

Solid circle $e = d/8$ Dia : $d/4$

Hollow circle $e = \frac{D^2+d^2}{8D}$ $e = \frac{D^2+d^2}{4D}$

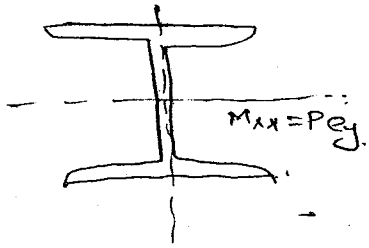
Square $e = \frac{b}{6}$ $e = \frac{b}{3}$

Rectangle

$e_y = \frac{b}{6}$ $e_x = \frac{d}{6}$ = dia. $\frac{b}{3}$ & $\frac{d}{3}$

I

$e_x = \frac{I_{yy}}{Ab_f}$ $e_y = \frac{I_{xx}}{AD}$



$$\frac{P}{A} - \frac{M}{Z} = 0$$

$$\frac{P}{A} - \frac{Pe}{\left(\frac{I}{y}\right)} = 0$$

$$\frac{P}{A} = \frac{Pe_y}{\frac{I_{xx}}{D/2}} = 0$$

$$\boxed{e_y = \frac{2 I_{xx}}{A D}} \qquad \boxed{e_x = \frac{2 I_{yy}}{A b_f}}$$

RANKINE FORMULA (EMPHIRICAL FORMULA)

$$\frac{1}{P_R} = \frac{1}{P_c} + \frac{1}{P_E} = \frac{P_E + P_c}{P_c P_E}$$

$$P_R = \frac{P_c P_E}{P_E + P_c} =$$

$$= \frac{P_c}{\frac{P_E}{P_E} + \frac{P_c}{P_E}}$$

$$= \frac{f_c A}{1 + \frac{f_c A}{\frac{\pi^2 E I}{l_{eff}^2}}}$$

$$= \frac{f_c A}{1 + \frac{f_c A}{\frac{\pi^2 E A r^2}{l_{eff}^2}}}$$

$$= \frac{f_c A}{1 + \frac{f_c}{\pi^2 E}\left(\frac{l_{eff}}{r}\right)^2}$$

$$= \frac{f_c A}{1 + \frac{f_c}{\pi^2 E} \lambda}$$

$$= \frac{f_c A}{1 + \alpha \lambda}$$

α - Rankine's constant.

$$\boxed{\alpha = \frac{f_c}{\pi^2 E}}$$

$$\boxed{P_R = \frac{f_c A}{1 + \alpha \lambda}}$$

Assumptions for Eulers Theory:

* column is a ~~large~~ long column.
* column fails only due to ~~balking~~ buckling
* column is subjected concentric load.
* column is initially straight (No initial curvature)
* column is loaded upto limit of proportionality
* Both ends of column are pinned (Ideal condition):

case (i) Both ends are pinned:

$$P_{euler} = \frac{\pi^2 EI}{l_{eff}^2} = \frac{\pi^2 EI}{l^2}$$

case (ii) Both ends are fixed.

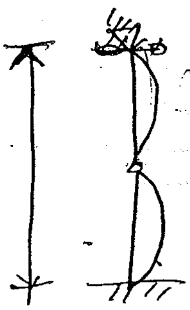
$$P_{euler} = \frac{4\pi^2 EI}{l^2}$$

case (iii) one end fixed ~~other~~ end pinned.

$$P_{euler} = \frac{2\pi^2 EI}{l^2}$$

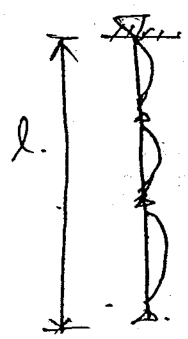
case (iv) one end fixed other end free.

$$P_{euler} = \frac{\pi^2 EI}{4l^2}$$



$\frac{l}{2} = l_{eff}$.

$$P_{euler} = \frac{4\pi^2 EI}{l^2}.$$



$\frac{l}{3} = l_{eff}$.

$$P_{euler} = \frac{9\pi^2 EI}{l^2}.$$

CONCRETE STRUCTURES

→ Concrete Technology

* properties of concrete
* Basics of mix design.

→ Concrete Design :

* Basic working stresses and limit state design concepts
* Analysis of ultimate load capacity
* Design of members subjected to flexure, shear, compression and torsion by limit state method.

→ Prestressed Concrete

* Basic elements of prestressed Concrete
* Analysis of beam sections at transfer and service load.

REINFORCED CONCRETE STRUCTURE

1.) WSM - Beams
 (i) Singly reinforced beam
 (ii) Doubly reinforced beam
 (iii) T-beam

2.) LSM - Beams
 (i) Singly reinforced beam
 (ii) Doubly reinforced beam
 (iii) T-beams.

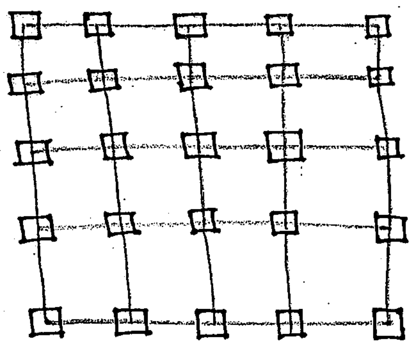
3.) Shear bond and Torsion.

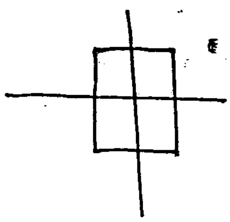
4.) Columns

5.) Foundation.

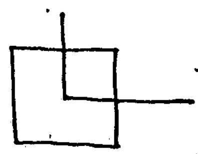
* Beam is a flexural member (or) bending member.

* Slab is also a flexural member.

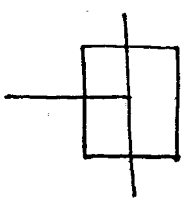




- Axial bending



- Biaxial bending



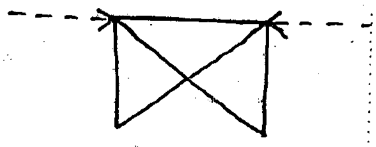
- Uniaxial bending

Father of Cement : Joseph Aspridin

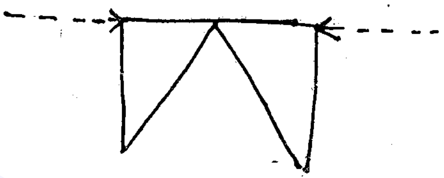
f_{ck} - characteristics compressive strength in concrete.

Base isolation technique : To prevent from seismic forces.

Bracings :



X - Bracing



K - Bracing.

Beam → steel failure → spalling of con

Minimum % of Reinforcement :

Slab - 0.12% of Ag for Fe415.

0.15% of Ag for Fe250.

Beam - $P_t = \frac{0.85\,bd}{f_y}$

Column - 0.8% of Ag

Maximum % of Reinforcement

Beam - 4% of Ag in compression

4% of Ag in tension.

Column - 4% of Ag (with overlapping)

6% of Ag (without overlapping)

RCC :

* RCC Stands for Reinforced Cement Concrete.
* Concrete is a mixture of cement, fine aggregate, coarse aggregate, ~~and~~ admixture and water in a pre-determin proportion.
* Concrete is very strong in compre and weak in tension.
* Tensile strength of concrete $= \frac{1}{10} \times$ compressive streng
* Concrete is designated by M_5, M_{10} M_{20} etc.,

where

$M \rightarrow$ Design mix

$20 \rightarrow f_{ck}$ @ 28 days.

* f_{ck} test result of sample shall not be less than 5%.
* f_{ck} @ 7 days $= \frac{2}{3} \times f_{ck}$ @ 28 days.
* f_{ck} @ 365 days $= (1.1 - 1.2) \times f_{ck}$ @ 28 d
* Cracking (or) Bending tensile strength of concrete $\Big\}$ $f_{cr} = 0.7\sqrt{f_{ck}}$

In PSC,

$E_c = 5700\sqrt{f_{ck}}$ (According to IS 1343-19

$E_c = 5000\sqrt{f_{ck}}$ (According to IS 456-20

Two major classification of RC section:

* Cracked section
* Uncracked section.

Cracked section:

RC section where the crac are permitted or in other word the crack doesnot affect the struc

Eg: Beam, column, wall, slab etc.

Uncracked section:

RC sections where hair line cracks are not permitted is called as uncracked section.

Eg: water tank, retaining wall and chimney. (storage ~~structures~~)

* Maximum permissible stress in concr in bending compression.

$$\sigma_{cbc} = \frac{1}{3} f_{ck}.$$

* Maximum permissible stress in concrete in direct compression.

$$\sigma_{cc} = \frac{1}{4} f_{ck}$$

Grade of Steel	$\sigma_{st} = \frac{f_y}{1.8} = 0.55 f_y$ (N/mm^2) (factor of safety of steel) = 1.8	σ_{sc} (r
* Mild steel Fe 250 Plain steel Grade-I steel	140 ($\phi \leq 20mm$) 130 ($\phi > 20mm$)	130
* HYSD Fe 415 TORSION STEEL	230	190
* HYSD Fe 500 TORSION STEEL	275	190

→ Bond strength of HYSD bars is 60% more than the plain bars.

Neutral Axis:

when a beam is subjected to a transverse load there will be bending compression @ top and bendin tension @ bottom fibre and at t point where there is zero stress (Neither bending tension (nor) Bending compress is called neutral axis.

* In uncracked section ~~the~~ tension and compression acts all over ~~base~~ the section.

Hence whole depth of the section is considered.

N.A lies @ the C.G of the section.

* In the cracked section compression is taken by concrete and tension is ~~taken~~ by steel alone. concrete in tension side is neglected. and then N.A is derived.

* Code book

IS 3370 water tank.
IS 1343. Prestressed concrete.
IS 1893. seismic Load.
IS 13920 Ductility Load.

* If the actual bending tensile stress developed in concrete due to applied bending moment is less than fcr the section is called uncracked section.

$$f_{bt} \leq f_{cr}.$$

$$f_{bt} = \frac{M}{I} \times y_t \leq f_{cr}.$$

bt – Bending tension.

* Maximum permissible B.M in concrete based on for is called cracking moment. (i.e) beyond ~~that~~ this B.M beam cracks.

As per IS 456 : 2000 there are 3 grades of concrete.

* ordinary – M_{10}, M_{15}, M_{20}
* standard – M_{25}, M_{30}, M_{35}, M_{40}, M_{45}, M_{50}, M_{55}.
* High quality – $M_{60} - M_{80}$

For concrete grade higher than M_{2} the mix proportion is based on mix design rule.

where,

Mean target strength. $f_t = f_{cm} = f_{ck} + k\,S$

As per IS 10262 – guidelines for concrete mix design.

where,

$k \rightarrow$ probability factor (or) tolerance factor. which depends upon percentage of error permitted in concrete.

* If error increases value of "k" decreases

According to IS 456-2000 for 5% of error the value of k is 1.65.

s - Standard deviation.

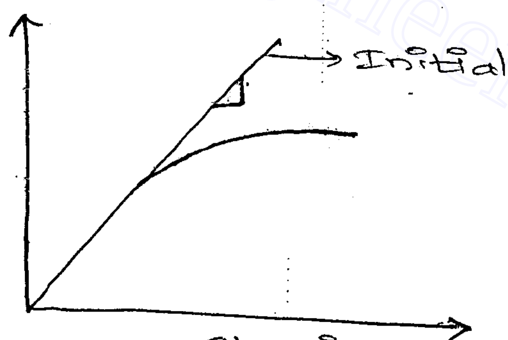
Depending upon Concrete grade va[illegible] in the range b/w 3·5 and 5.

Concrete grade	Standard deviati[illegible]
M_{10}, M_{15}.	3·5 N/mm^2
M_{20}, M_{25}.	4 N/mm^2
M_{30}, M_{50}.	5. N/mm^2

There are 3 methods to determin[illegible] modulus of elasticity of concrete.

* Initial tangent Method
* Tangent method
* Secant method.

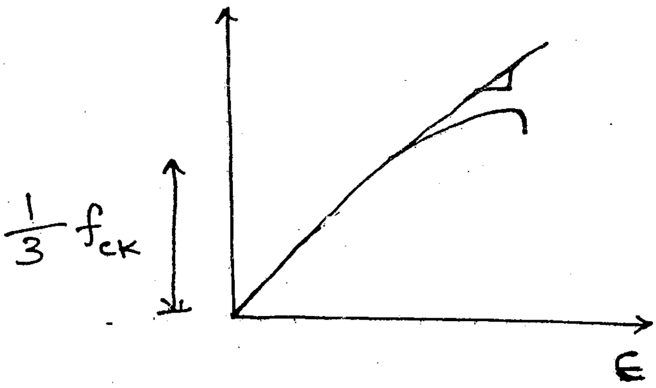
Initial Tangent Method:



Slope of initial tangent

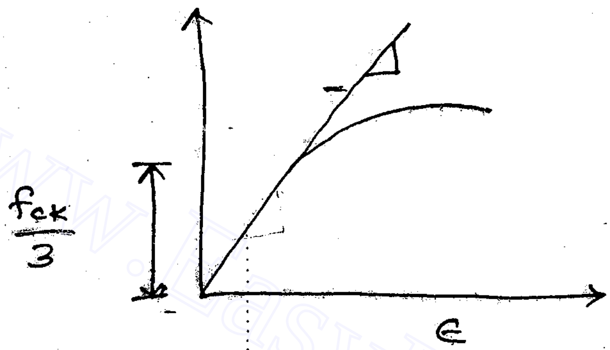
$$\tan\theta = \frac{\Delta f}{\Delta E}$$

Tangent Method :



Slope of tangent } = $\frac{\Delta f}{\Delta}$

Secant Method :



Slope of straight li

$= \frac{\Delta f}{\Delta \epsilon}$

$$E_c = 5000 \sqrt{f_{ck}}$$

where,

E_c - Modulus of elasticity of concrete (or) short term elastic Modulus. (or) short term static Young's modulus

* E_c is arrived from Secant meth

* Long term (or) effective Modulus of elastic of concrete due to creep - effect

$$\boxed{E_{eff} = \frac{E_c}{1+\theta}}$$

θ - creep co-effic

Modular Ratio (m):

* It is defined as the ratio of modulus of elasticity of steel to that of modulus of elasticity of concrete.

* It's symbol is m.

* It is very important to conv steel area into equivalent concrete area in WSM. for analysis purp

* $m = \frac{E_s}{E_c}$ is not considered in RCC analysis based on WSM. because Young's modulus of concrete does not take creep and shrinkage into account.

* Above value is used in pre-stre only where losses due to creep and shrinkage are considered seperately.

* For RCC based on WSM the modular ratio is given by,

$$m = \frac{280}{3\sigma_{cbc}}$$

* Above formula partially takes into account long term effects such as creep.

* According to IS : 456-2000 the minimum clear cover for steel bar shall be as below.

Type of Exposure	Pcc grade	Rcc grade	Minimum clear cover
Mild	M_{10}	M_{20}	20 mm
Moderate	M_{15}	M_{25}	30 mm
Severe	M_{20}	M_{30}	45 mm
Very severe	M_{20}	M_{35}	50 mm
Extreme.	M_{25}.	M_{40}.	75 mm

Exposure : As per IS 456-2000 cl-8.2.2 and 35.3.2 (Table-3).

Mild Exposure:
Structure protected from rain, heat, etc., (weather condition)
For eg : Beams, slabs.

Moderate Exposure:
Structure exposed to rain, heat etc.,
* Structure burried in non-aggressive soil and water.

Severe Exposure:
* Structure exposed to severe rain
* under sea structure.
* Surface exposed to alternate wetting and drying.
* Exposed to coastal environment.

Very Severe Exposure:

* Concrete surface exposed to sea spray.
* Concrete is burried or in con[tact] with aggressive sub soil / ground water.

Ex: coastal areas.

Extreme Exposure:

* Surface of members in tidal zon[e]
* Members in direct contact wi[th] solid / liquid aggressive chemicals.

Minimum clear cover for structural units:

For slabs. - 20mm
For Beams. - 25 mm
For column. - 25mm (<200mm), 40mm (>200mm)
For Foundation. - 50 mm
For Retaining wall - 40-50mm.

Quantity of cement.

Exposure Types.	Qty of cement Pcc	Qty of cement Rcc	W/c ratio Pcc	W/c ratio Rcc
Mild	220kg	300kg	0.6	0.55
Moderate	240kg	300kg	0.6	0.5
Severe	250kg	320kg	0.5	0.45
Very Severe	260kg	340kg	0.45	0.45
Extreme	280kg	360kg	0.4	0.4

To avoid corrosion of steel bars:

(i) Adequate concrete cover.

(ii) More quantity of cement.

(iii) Maximum compact of concrete to reduce permeabi

(iv) Coating of steel bars.

(v) cathodic portection.

Dia of steel bars:

6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 3

40, 45, 50

Minimum dia of column - 12mm.

* Isolated footing same as one ways

* Combined footing same as beam.

* Retaining wall same as slab.

Analysis Of Section By WSM

Statement :

* Analysis of uncracked section and cracked section based on WSM are based on the principle of strain compatibility.

* Due to good bond b/w concrete and steel (i.e) the strain developed in concrete in the tension zone in the vicinity of tension steel is equal to the strain developed in the tension steel.

Area equivalent stress

$$\epsilon_c = \epsilon_s$$

$$\frac{f_c}{E_c} = \frac{f_s}{E_s}$$

$$f_c = \frac{f_s}{(E_s/E_c)}$$

$$f_c = \frac{f_s}{m}$$

$$\boxed{\sigma_c = \frac{\sigma_{st}}{m}}$$

Area equivalent concrete

$$F_c = F_s$$

$$f_c A_c = f_s A_{st}$$

$$\epsilon_c \times E_c \times A_c = \epsilon_s \times E_s \times A_s$$

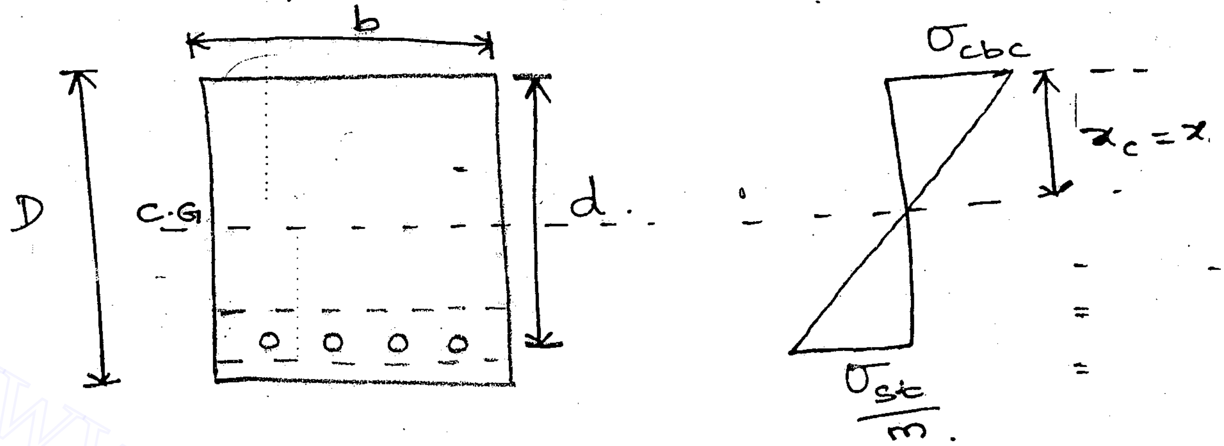
$$A_c = \frac{E_s}{E_c} \times A_{st}$$

$$\boxed{A_c = m A_{st}}$$

Analysis Of Uncracked Section.

* Neutral Axis of uncracked section lies at C.G of the section.

Analysis of Cracked Section:



Depth of actual neutral Axis:

Area $\times$ C.G of comp = Area $\times$ C.G of tensi

$$b \times x_a \times \frac{x_a}{2} = m \times A_{st} \times (d - x_a)$$

$$\boxed{\frac{b x_a^2}{2} = m A_{st} (d - x_a)}$$

Depth of critical N.A :

From stress - block diagram, by similar triangle method.

$$\frac{\sigma_{cbc}}{(\sigma_{st}/m)} = \frac{x_a}{(d - x_a)} \qquad \boxed{x_a = x_c}$$

$$\sigma_{cbc}(d - x_c) = x_c \frac{\sigma_{st}}{m}$$

$$\sigma_{cbc} d - \sigma_{cbc} x_c = x_c \frac{\sigma_{st}}{m}$$

$$m \sigma_{cbc} d - m \sigma_{cbc} x_c = x_c \sigma_{st}$$

$$m\sigma_{cbc}\, d = x_c(m\sigma_{cbc} + \sigma_{st})$$

$$\cancel{x_c} = \cancel{\frac{m\sigma_{cbc} + \sigma_{st}}{m\sigma_{cbc}\, d}}$$

$$x_c = \frac{m\sigma_{cbc}\, d}{(m\sigma_{cbc} + \sigma_{st})}$$

$$\boxed{x_c = k \cdot d}$$

$$\boxed{k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}}$$

$$= \frac{\frac{280}{3\cancel{\sigma_{cbc}}} \times \cancel{\sigma_{cbc}}}{\frac{280}{3\cancel{\sigma_{cbc}}} \times \cancel{\sigma_{cbc}} + \sigma_{st}}$$

$$= \frac{280}{\cancel{3}} \times \frac{\cancel{3}}{280 + 3\sigma_{st}}$$

$$\boxed{k = \frac{280}{280 + 3\sigma_{st}}}$$

Compressive stress:

$$F_c = \text{Stress} \times \text{Area}$$

$$= \frac{0 + \sigma_{cbc}}{2} \times b \times x_a$$

$$= \frac{\sigma_{cbc}\, b\, x_a}{2}$$

$$\boxed{F_c = \frac{1}{2} c\, b\, x_a}$$

Tensile stress:

$$F_t = \frac{\sigma_{st}}{\cancel{m}} \times \cancel{m}\, A_{st}.$$

$$\boxed{F_t = t\, A_{st}}$$

Lever Arm:

$$z = d - \frac{x_a}{3}.$$

$$\boxed{x_a = x_c}$$

$$= d - \frac{x_c}{3}.$$

$$= d - \frac{kd}{3}.$$

$z = dj$.

$$= d\left[1 - k/3\right]$$

$$\boxed{z = dj}$$

Moment of Resistance (comp):

$$M = F \times L.A.$$

$x_a = x_c$

$$= \frac{c\,b\,x_a}{2}\left(d - \frac{x_a}{3}\right).$$

$$= \frac{c\,b\,x_c}{2}\left(d - \frac{x_c}{3}\right).$$

$$= \frac{c\,b\,kd}{2}\left(d - \frac{kd}{3}\right).$$

$$= \frac{c\,b\,kd^2}{2}\left(1 - k/3\right).$$

$$= \frac{ckj}{2}\, bd^2$$

$$\boxed{M = Q\,bd^2}$$

Moment of Resistance: (Tension)

$$M = \text{Force} \times \text{Lever Arm}$$

$$= t \times A_{st} \times \left(d - \frac{x_a}{3}\right)$$

$x_a = x_c$.

$$= t \times A_{st} \left(d - \frac{x_c}{3}\right)$$

$$= t \times A_{st} \left(d - \frac{kd}{3}\right)$$

$$= t A_{st}\, d\, (1 - k/3)$$

$$= t A_{st}\, d j.$$

This is the reason why depth is always greater than its breadth

Note: (cracking) tensile stress
(Rayleigh)

$$f_{cr} = 0.7\sqrt{f_{ck}}.$$

$$\text{cracking tensile strain} = \frac{f_{cr}}{E_c}.$$

$$= \frac{0.7\sqrt{\cancel{f_{ck}}}}{5000\sqrt{\cancel{f_{ck}}}}.$$

Strain in steel $= 1.4 \times 10^{-4}$.

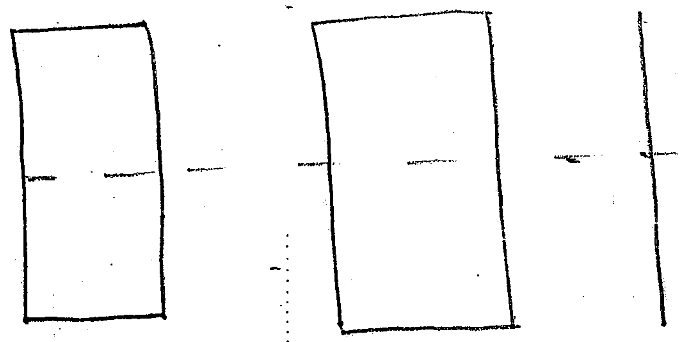
stress in steel $= \varepsilon \times E$

$$= 1.4 \times 10^{-4} \times 2 \times 10^{5}$$

$$= 28\ N/mm^2.$$

Types of section:

* Under reinforced section.
* Balanced section.
* over reinforced section.



Balanced section.

* A_{st} required = A_{st} provided.
* $x_a = x_c$.
* x_a and x_c lies on N.A.
* Failure is balanced one.
* $c = \sigma_{cbc}$, $t = \sigma_{st}$.

URS:

* A_{st} pro < A_{st} required.
* $x_a < x_c$ — x_a lies below
* Ductile failure.

ORS:

* A_{st} pro > A_{st} requir.
* $x_a > x_c$ — x_a lies below x_c
* Brittle failure.

1. Given data:

$M_{external} = 60$ kN.m.

M_{20}, Fe 415. $b = ?$ $d = ?$ A_{st}

$M = Q\,bd^2$.

$Q = \frac{cjk}{2}$ $c = \sigma_{cbc} = \frac{f_{ck}}{3} = \frac{20}{3}$

$\boxed{c = 7}$

$k = \frac{280}{280 + 3\sigma_{st}}$.

$= \frac{280}{280 + 3(230)}$.

$\boxed{k = 0.28}$

$j = 1 - \frac{k}{3}$.

$= 1 - \frac{0.28}{3}$

$\boxed{j = 0.906}$

$Q = \frac{7 \times 0.28 \times 0.906}{2}$.

$60 \times 10^6 = \frac{7 \times 0.28 \times 0.906}{2} \times \frac{d}{2} \times d^2$.

$\boxed{d = 513\text{mm}}$

$b = d/2$. $b =$

$D = 513 + 20 + 16/2$

$= 541$

$\boxed{D = 550\ mm}$

$d = 550 - 20 - \frac{16}{2}$

$\boxed{d = 522\,mm}$

$M.R = t\,A_{st}\left(d - \frac{x_a}{3}\right)$

$= t\,A_{st}\,d\left(1 - \frac{K}{3}\right)$

$60 \times 10^6 = 230 \times A_{st} \times 513 \times \left(1 - \frac{0.28}{3}\right)$

$x_c = A_{st} = 560\ mm^2$

(*) pure B.M is there, shear force = zero.

$\text{No. of bars} = \dfrac{560}{\frac{\pi \times 16^2}{4}}$

$= 3$ nos.

$\therefore$ provide $3\#\,16mm\ \phi$ bar.

Min % of reinforcement

$= \dfrac{0.85bd}{fy}$

$= \dfrac{0.85 \times \frac{522}{2} \times 522}{415}$

$= 280\,mm$

Max % of reinforcement

$= 4\%$ of A_g.

$= \frac{4}{100} \times b \times D$

$= \frac{4}{100} \times \frac{522}{2} \times 550$

$A_{st} = 5742\ mm^2$

2.) Given data:

$b = 230mm$ & $D' = 500 mm$.

M_{20} , Fe 415.

$M.R = Q\, bd^2$.

$k = \frac{280}{280+3\sigma_{st}}$

$Q = \frac{cjk}{2}$

$= \frac{280}{280+3(230)}$

$= \frac{7 \times 0.28 \times 0.906}{2}$

$\boxed{k = 0.28}$

$j = 1 - \frac{k}{3}$.

$= 1 - \frac{0.28}{3}$

$M.R = \frac{7 \times 0.28 \times 0.906}{2} \times 230 \times 472$.

$\boxed{j = 0.906}$

$M = 45.49$ KN.m.

$D = 550 - 28$

$= 472mm$

$b \frac{x_a^2}{2} = m \times A_{st} (d - x_a)$.

$M = t\, A_{st}\, dj$.

$A_{st} = 603 \; mm^2$.

$b \cdot \frac{x_a^2}{2} = m\, A_{st}\, (d - x_a)$.

$x_a = 150mm$.

$x_c = k \times d$.

$x_c = 132.16$.

$x_a > x_c$.

$\therefore$ It is over reinforced.

14/8/2015

(9)

REINFORCED CEMENT CONCRETE

~~calc~~

1. If M_{20} and Fe 415 calculate the ratio N.A depth factor and Lever arm factor.

$\sigma_{cbc} = \frac{f_{ck}}{3} = 7$

$m = 14$

$$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$$

$$= \frac{14 \times 7}{14 \times 7 + 230}$$

$$k = 0.2987$$

$$j = 1 - \frac{k}{3}$$

$$j = 0.9$$

$$\frac{k}{j} = \frac{0.2987}{0.9}$$

$$\frac{k}{j} = 0.3317$$

Percentage of tension Reinforcement

$$F_c = F_t$$

$$\frac{1}{2} \times c \times b \times x_a = t \times A_{st}$$

$$\boxed{x_a = x_c}$$

$$\frac{1}{2} \times c \times b \times x_c = t \times A_{st}$$

$$\frac{1}{2} \times c \times b \times k \times d = t \times A_{st}$$

$$\boxed{\frac{A_{st}}{bd} = \frac{kc}{2t}}$$

$$\boxed{P_t = \frac{kc}{2t} \times 100}$$

2.) Given: $b = 300m$, $D = 600mm$, $d = 550mm$
4 #25mm Fe 415. $M_{max} = 50$ kN.m, M

$$M = t \times A_{st}$$

$$t = \frac{M}{A_{st}} = \frac{50 \times 10^6}{4 \times \left(\frac{\pi \times 25^2}{4}\right)}$$

$$t = 25.46 \times 10^3 \ N/mm^2 < 230 \ N/mm$$

$$M = \frac{Qbd^2}{2}$$

$$Q = \frac{ckj}{2}$$

$$k = 280 \times$$

$$k = \frac{m \sigma_{cbc}}{m \sigma_{cb} + t}$$

$$M = F_c \times \left(d - \frac{x_a}{3}\right)$$

$$= \frac{c}{2} \times b \times x_a \left(d - \frac{x_a}{3}\right)$$

$$\frac{b\,x_a^2}{2} = m\,A_{st}\,(d - x_a).$$ $m = \frac{280}{3\sigma_{cbc}}$

$m = 14$

$$150\,x_a^2 = 14 \times \cancel{4} \times \frac{\pi \times 25^2}{\cancel{4}}(550 - x_a).$$

$$150\,x_a^2 + 27488.93\,x_a - 15.1189 \times 10^6 =$$

$$x_a = 238.807\,mm.$$

$$50 \times 10^6 = \frac{c}{2} \times 300 \times 238.81\left(550 - 238.81/3\right)$$

$$c = 2.967\ N/mm^2. \quad < \sigma_{cbc}$$

3.) R.C.C beam size $300 \times 600\,mm$ is reinforced with 4 # 16mm ϕ in compression zone. 5 # 20mm ϕ in tension zone. Determine (i) M.R.

(ii) M.R = 60 kN-m c, t

Used. M_{20} Fe 415.

Given; $d = 560\,mm$.

$$\frac{b\,x_a^2}{2} = m\,A_{st}\,(d - x_a).$$

$$\frac{300\,x_a^2}{2} =$$

$$M.R = \sigma_{st} \times A_{st}.\left(d - \frac{x_a}{3}\right).$$

$$= 230 \times 5 \times \left(\frac{20^2 \times \pi}{4}\right)$$

$$= \cancel{4.6968\ kN\text{-}mm}.$$

$$= 361.28\ N\text{-}mm.$$

$$M.R = \frac{c\,k\,j}{2} \times b$$

$$\frac{b\,x_a^2}{2} = m\,A_{st}\,(d - x_a)$$

~~300~~

$$M.R = \frac{ckj}{2} \times b \times d^2$$

$c \doteq \frac{20}{3}$ m

$= 7$

$$= \frac{7 \times 0.298 \times 0.9}{2} \times 300 \times 560^2$$

$k = \frac{14 \times 7}{14 \times 7 + ...}$

$= 0.298$

$M.R = 88.31$ kN.m

$$M.R = \frac{c}{2} \times b \times x_a \left(d - \frac{x_a}{3}\right)$$

$j = 1 - \frac{k}{3}$

$= 0.9$

(ii)

$$M.R = F_c \times \left(d - \frac{x_a}{3}\right)$$

$$= \frac{c \times b \times x_a}{2} \left(d - \frac{x_a}{3}\right)$$

$$\frac{b x_a^2}{2} = m A_{st} (d - x_a)$$

$$\frac{300 \times x_a^2}{2} = 14 \times 5 \times \frac{\pi \times 20^2}{4} (560 - x_a)$$

$$150 x_a^2 + 21991.15 x_a - 12.315 \times 10^6 = 0$$

$x_a = 222.46$ mm

$$60 \times 10^6 = \frac{c}{2} \times 300 \times 222.46 \left(560 - \frac{222.46}{3}\right)$$

$c = 3.7$ N/mm^2 $<$ 7 N/mm^2

$$= F_t \times \left(d - \frac{x_a}{3}\right)$$

$$60 \times 10^6 = t \times A_{st} \left(d - \frac{x_a}{3}\right)$$

$$= t \times 5 \times \frac{\pi \times 20^2}{4} \left(560 - \frac{222.46}{3}\right)$$

$t = 78.61$ N/mm^2 $<$ 230 N/mm^2

18/8/2015

Notes:

* Structures more than 45m length should be designed with one or more expansion joints.

* $E_{ce} = \frac{E_c}{1+\theta}$ θ - creep co-effi

Days	θ
7 days	2.2
28 days.	1.6
1 years	1.1.

*.- Individual variation in the compressive strength of 3 cubes in the sample should not exceed ±15%.

* fck @ 1 day 16%.

fck @ 3 day ~~45%~~ 40%.

@ 7 day

fck @ 7 day. 65%.

fck @ 14 day 90%.

fck. @ 28 day 99%.

* The rate of increase in compressive strength decreases with increase in time. Therefore we consider 28days fck of concrete.

* Minimum grade of concrete ~~co~~ va[rious] types of structures :

Type of structure	Minimum grade of concrete
1. Lean concrete bases	M_5, $M_{7.5}$.
2. P.C.C	M_{10}
3. R.C.C (General construction & under normal condition)	~~Gene~~ M_{20}
4.) water tank, Dome, folded plates, shell roof.	M_{20}.
5.) R.C.C structures in sea water	M_{20} (P.C.C) M_{30} (R.C.C)
6.) Post tensioned Pre stressed concrete	M_{30}.
7.) Pre tensioned Pre stressed concrete	M_{40}.

* ~~Types of Concrete as per~~.

* Factors affecting crushing strength cube :

→ <u>Size</u> <u>factor</u> :

As the size of the cube decreas[es] strength increases because of better homogeneity.

For eg: cube of 100mm size will hav[e] 5% more strength than 150mm cub[e]

→ shape factor

cylinder of size h = 30cm d = 15c has strength 80% ~~of~~ as that of cube of 150mm.

→ Slednderness Ratio:

As slenderness ratio of specimen increase strength decreases.

* weight batching is preferable the compared to volume batching.

* Qty of water required per one bag of cement for M_{15} mix is 32 litres, and for M_{20} mix 30 litres.

* As the size of the cube increases the strength of concrete reduces due to chances of more weak spot

2/9/2015

R.C.C

DOUBLY REINFORCED BEAM:

If steel bars are provided in compression zone ~~in~~ of an R.C.C section in addition to steel bars in the tension zone ~~then~~ ~~that~~ RCC section is called as doubly reinforc section.

Doubly Reinforced Beam are adopte under following circumstances:

* If depth restricted
* If breadth restricted
* If external B.M is greater than M.R.
* If there is reversal of stre
* If precast pile is used.
* If there is shortage of heigh during form work.

→ If ~~tensio~~ steel is provided in 2 rows in tension zone only 25% of M.R increases (singly reinforc beam).

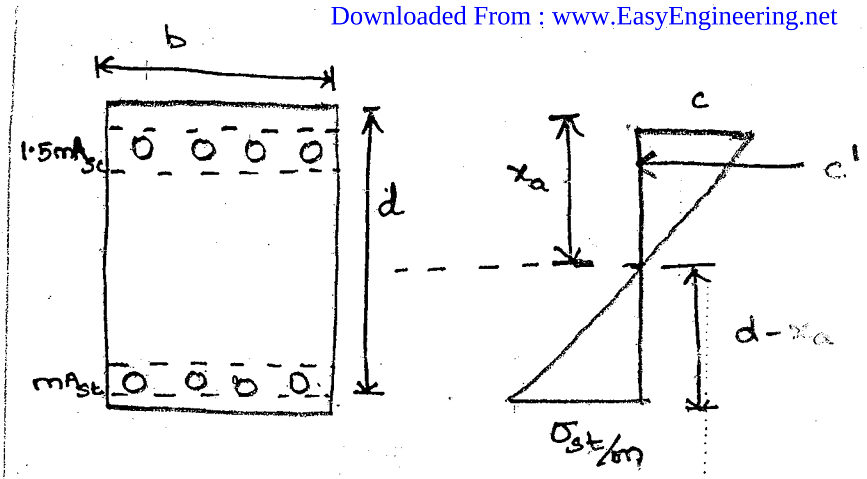
→ when steel rods provided in compressi zone the equivilaent area of concrete is 1.5m Asc, due to developm of inelastic strain caused by creep and shrinkage induced in the ~~creep~~ compression zone of the beam.

→ The stress developed in the compress steel = 1.5mc'.

It means the stress developed in compression steel is mainly governed by concrete in the compression zone

∴ use of high grade steel in compression is meaningless and hence doubly reinforced section in working stress is not economical.

→ If hanger bars are provided to supports ~~to~~ strippus in a singly reinforce beam, then it cannot be called as doubly reinforced section i.e. to be called as a compression steel the min. Area of hanger bars shall be more than or equal to 0.2 % concrete area.



Actual neutral Axis.

$$(A \times C.G)_c = (A \times C.G)_T$$

$$\left(b \times x_a \times \frac{x_a}{2}\right) - A_{sc}(x_a - d') + 1.5m A_{sc}(x_a - d') = mA_{st}(d - x_a)$$

$$\boxed{b\frac{x_a^2}{2} + (1.5m - 1) A_{sc}(x_a - d') = m A_{st}(d - x_a)}$$

critical neutral Axis

$\boxed{x_c = k \times d}$ same as singly reinforced beam.

Force in ~~Concrete~~ Compression Zone (F_c)

$$F_c = F_{c_1} - F_{holes} + F_{c_2}$$

$$= \left(\frac{0 + c}{2} \times b \times x_a\right) - (c' \times A_{sc}) + (c' \times 1.5mA_{sc})$$

$$= \frac{c b x_a}{2} + \left[(1.5m - 1) \times [c' \times A_{sc}]\right]$$

$$\boxed{F_c = \frac{1}{2} c b x_a + (1.5m - 1)(c' \times A_{sc})}$$

By similar triangle property

$$\boxed{c' = \frac{c(x_a - d')}{x_a}}$$

Force in Tension zone (F_{st}):

$$\boxed{F_{st} = \sigma_{st} \times A_{st}}$$

Moment Of Resistance Due to Compression Force:

$$M.R = (F_{c_1} \times Z_1) - (F_{holes} \times Z_2) + (F_{steel} \times Z_2)$$

$$= \left[\frac{1}{2} c b x_a \left(d - \frac{x_a}{3}\right)\right] - A_{sc} \times c'(d-d') + 1.5 m A_{sc} c'(d-d')$$

$$\boxed{M.R = \left[\frac{1}{2} c b x_a \left(d - \frac{x_a}{3}\right)\right] + (1.5m-1) A_{sc} c'(d-d')}$$

1. Design R.C.C section 250 × 500 mm subjected to B.M = 110 kN.m. Fe 415, M25, $d' = 50$ mm.

$$M.R = Q b d^2 \qquad Q = \frac{ckj}{2} \qquad c = \frac{f_{ck}}{3}$$

$$C = \frac{25}{3} = 8.33 \text{ N/mm}^2 \qquad k = \frac{280}{280 + 3\sigma_{st}}$$

$$j = 1 - \frac{k}{3} = 0.9037 \qquad = \frac{280}{280 + 3 \times 230}$$

$$= \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$$

$$M.R = \left(\frac{8.33 \times 0.288 \times 0.9037}{2}\right) \times 250 \times 500^2 \qquad k = 0.288$$

$$= 67.75 \times 10^6 < 110 \text{ kNm}$$

~~Hence~~ Since external moment is greater than M.R (i.e) 110 > 67.75 kN.m adopt doubly reinforced section.

$$M \cdot R = \frac{c b x_a}{2} \times \left(d - \frac{x_a}{3}\right) + (1.5m - 1) A_{sc} (d - d') c'$$

$x_a = kd$

$= 500 \times 0.28$

$x_a = \cancel{450}\ 140$

$m = \frac{280}{3\sigma_{cb}}$

$= \frac{280}{3 \times 8.5}$

$m = 10.98$

$c' = \frac{c(x_a - d')}{x_a}$

$= \frac{8.5(140 - 50)}{140}$

$c' = 5.46$

$$110 \times 10^6 = \left(\frac{8.5 \times 250 \times 140}{2}\right) \times \left(500 - \frac{140}{3}\right)$$

$$+ ((1.5 \times 10.98) - 1) \times A_{sc} (500 - 50) \times 5.46$$

$$= \cancel{1593750}\ 67.43 \times 10^6 + 6736.5\, A_{sc} \times 5.46 \quad 36781.29$$

$A_{sc} = \cancel{1119.88}\ 1157.38\ mm^2$

To find A_{st}. $f_c = F_t$

$F_c = F_t$

$$\frac{c b x_a}{2} + (1.5m - 1) A_{sc} c' = t \times A_{st}$$

$$\left(\frac{8.5 \times 250 \times 140}{2}\right) + \left(\left[(1.5 \times 10.98) - 1\right] \times 1157.38 \times 5.46\right) = 230 \times A_{st}$$

$$148750 + 97.76 \times 10^3 = 230 \times A_{st}$$

$$\boxed{A_{st} = 1071.78\ mm^2}$$

FLANGED BEAMS: (T-beam & L-beam)

T- BEAM :

* T-Beam is a R.C.C beam where beam and slabs are constructed monolithicall such that slab is always under compression and hence extra concrete is not required for the beam to behave like compression zone, as a result depth of the beam is reduced to a greater extent.

* T- beam is the most economical R.C.C beam where compression zone is totally or partially not required

* A simply supported beam having monolithical ~~base~~ slab @ the top is called T-beam.

But if the beam is kept above the slab to get plane soffit then the beam is not a T-beam (i.e) it is an inverted rectangular beam.

* For a continous beam @ support, hogging B.M develops (Tension @ top fibre due to which the slab does not contribute compression zone for beam

Flanged beam

T-beam: slab and beam are considered monolithic structure

Uses.

* Depth can be reduced
* Economical section.

Hypothetical uniform load = Max. load @ centre of the beam

F_c due to actual compressive stress } = f_c due to hypothetical stress.

hypothetical width = $b_{eff} \leq B_f$ actual.

hypothetical width is effective width of flange
↓
$\leq$ Actual width of the flange

and hence the beam shall be designed as ~~rectangular beam~~ only @ intermediate support.

* In case of portico (canopy) if the beam is kept @ the bottom of the cantilever slab projecting both the sides the beam shall be designed as rectangular beam.

But if the beam is kept @ the top it shall be designed as T-beam

* In case of T-beam the max. Bending compressive stress developed in the slab is always @ the centre of web. which gradually decreases towards middle of the slab b/w two webs.

As a result the T-beam cannot be analaysed due to variation of stress @ one section in such case a hypothetical width of the flange shall be considered where uniform stress having maximum value equal to the stress @ the centre of web.

* The compressive force due to actual bending compressive stress must be equal to the compressive force due to hypothetical uniform stress.

The hypothetical width of the flange is known as effective width

Effective width of T-beam depend upon

- → l_o for continuous beam $l_o = 0.7l$.
- → b_w
- → D_f.
- → c/c spacing of T-beam
- → Support condition
- → Type of load.

of the flange which is always less than or equal to actual width of flange.

* The effective ~~dep~~ width of T-beam depend upon following factors:

→ Effective span of T-beam (l_o)
(Distance b/w zero B.M to zero B.M)

→ In case of Continous Beam $\boxed{l_o = 0.7 \times l}$

→ width of the web. (b_w)

→ Depth of the slab (D_f)
(or) flange

→ C/c spacing of T-beam.

→ Support condition of beam

→ Type of Load

no of beams

$$b_f = \frac{l_o}{6} + b_w + \frac{D_f}{6}$$ for T

$$b_f = \frac{l_o}{12} + b_w + \frac{D_f}{3}$$ for L

Inverted T beams

$$b_f = \frac{l_o}{\left(\frac{l_o}{6} + 4\right)} + b_w$$

28/9/2015

T- BEAM

Effective width of Flange (b_f):

* For no. of. T-beams.

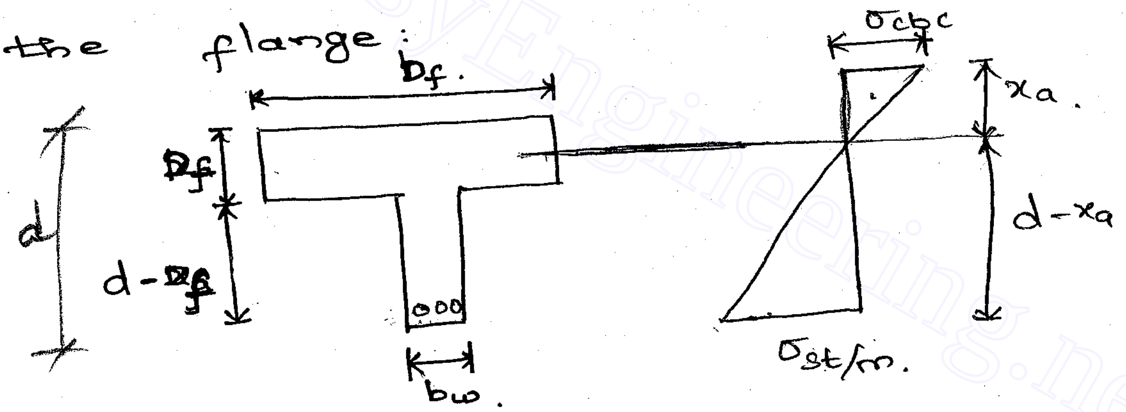
$$b_f = \frac{l_0}{6} + b_w + 6D_f \leq b \text{ (T-beam)}$$

$$b_f = \frac{l_0}{12} + b_w + 3D_f \leq b \text{ (L-beam)}$$

* For isolated T-beam.

$$b_f = \frac{l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w$$

case (i) If the T-beam N.A lies within the flange:



→ Actual Neutral Axis:

$$(\text{Area} \times C.G)_C = (A \times C.G)_T$$

$$b \times x_a \times \frac{x_a}{2} = mA_{st}(d - x_a)$$

$$\boxed{\frac{b x_a^2}{2} = mA_{st}(d - x_a)}$$

b_{eff}:

→ For no. of. T-beams

* $b_{eff} = \frac{l_0}{6} + b_w + 6D_f \leq b$ (T-beam)

* $b_{eff} = \frac{l_0}{12} + b_w + 3D_f \leq b$ (L-beam)

→ For Isolated ~~no. of.~~ Flanged beam

* $b_{eff} = \frac{l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w$ (T-beam)

Design of T-beam:

case (i) N.A lies with in the flange: (Design of T-beam is similar to design of singly reinforced beam)

* A.N.A:

$$\frac{b_f x_a^2}{2} = m A_{st}(d - x_a)$$

* C.N.A: $x_c = kd$.

* F_c:

$$F_c = \frac{1}{2} c\, b_f x_a.$$

* F_T: $F_t = t A_{st}$ * Lever arm: $z = dj$.

* M.R due to concrete:

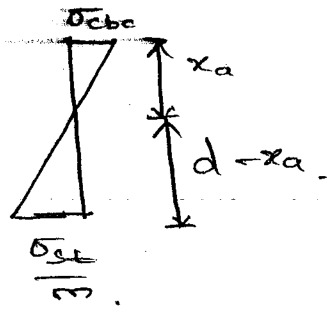
$$M.R = \frac{1}{2} c\, b_f x_a \left(d - \frac{x_a}{3}\right).$$

* M.R due to Tension zone.

$$M.R = t A_{st}\left(d - \frac{x_a}{3}\right)$$

→ Critical Neutral Axis

$$\boxed{x_c = kd}$$



→ Force in Compression zone.

$$F_c = \frac{0 + C}{2} \times b \times x_a$$

$$\boxed{F_c = \frac{1}{2} C b x_a}$$

→ Force in Tension zone.

$$F_t = \frac{\sigma_{st}}{\cancel{m}} \times \cancel{m} A_{st}$$

$$\boxed{F_t = \sigma_{st} A_{st}}$$

→ Lever Arm :

$$\boxed{z = dj}$$

→ M.R Due To compression zone :

$$\boxed{M.R = \frac{1}{2} c b x_a \left(d - \frac{x_a}{3}\right)}$$

→ M.R Due To Tension zone :

$$\boxed{M.R = t A_{st} \left(d - \frac{x_a}{3}\right)}$$

Note:

* In T-beam if actual neutral axis lies within the flange. the design similar to the design of singly reinforced Beam.

case (ii) If N.A lies outside the flange.

* A.N.A

$$b_f D_f \left(x_a - \frac{D_f}{2}\right) + \left(b_w \times \frac{(x_a - D_f)^2}{2}\right) = m A_{st} (d - x_a)$$

* C.N.A

$$x_c = kd$$

* F_c :

$$F_c = F_{c_1} + F_{c_2}$$

$$F_c = \left(\frac{c + c'}{2} \times D_f b_f\right) + \left[\frac{1}{2} c' b_w (x_a - D_f)\right]$$

$$c' = \frac{c(x_a - D_f)}{x_a}$$

* F_t :

$$F_t = t A_{st}$$

* Lever arm:

$$z_1 = d - \bar{y} = d - \left(\frac{c + 2c'}{c + c'} \times \frac{D_f}{3}\right)$$

$$z_2 = d - D_f - \left(\frac{x_a - D_f}{3}\right)$$

* M.R due to compression zone

$$M.R = \left[\frac{c + c'}{2} \times b_f D_f \times \left(d - \left(\frac{c + 2c'}{c + c'} \times \frac{D_f}{3}\right)\right)\right]$$

$$+ \left[\frac{1}{2} c' b_w (x_a - D_f) \times \left(d - D_f - \left(\frac{x_a - D_f}{3}\right)\right)\right]$$

* M.R due to Tension zone

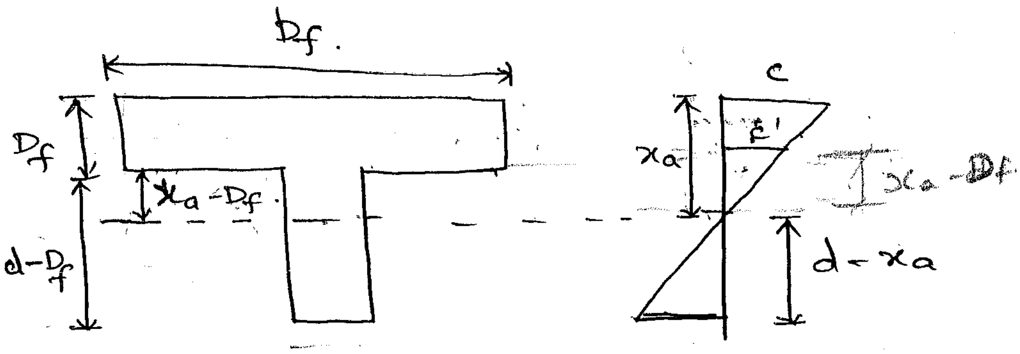
$$M.R = t A_{st} \left(d - \frac{x_a}{3}\right)$$

* Most Economical depth.

Total cost = cost of concrete + cost of steel.

$$d = \sqrt{\frac{r_r \, M_{max}}{\sigma_{st} \, j \times b_w}} \qquad r_r = 70 - 80\%$$

Case (ii) If N.A lies outside the flange



→ Actual Neutral Axis:

$$(A_1 \times C.G_1) + (A_2 \times C.G_2) = (A \times C.G)_T$$

$$\left[b_f \times D_f \times \left(x_a - \frac{D_f}{2}\right)\right] + \left[b_w \times (x_a - D_f) \times \left(\frac{x_a - D_f}{2}\right)\right] = m A_{st} (d-$$

$$\boxed{\left[b_f \times D_f \times \left(x_a - \frac{D_f}{2}\right)\right] + \left[b_w \times \frac{(x_a - D_f)^2}{2}\right] = m A_{st}(d-x_a)}$$

→ Critical Neutral Axis:

$$\boxed{x_c = kd}$$

→ Force in compression zone:

$$F_c = F_{c1} + F_{c2}$$

$$c' = \frac{c\,(x_a - D_f)}{x_a}$$

$$F_{c1} = \underbrace{\frac{c+c'}{2}}_{\text{Stress}} \times \underbrace{b_f \times D_f}_{\text{Area}}$$

$$F_{c2} = \underbrace{\frac{0+c'}{2}}_{\text{stress}} \times \underbrace{b_w \times (x_a - D_f)}_{\text{Area}}$$

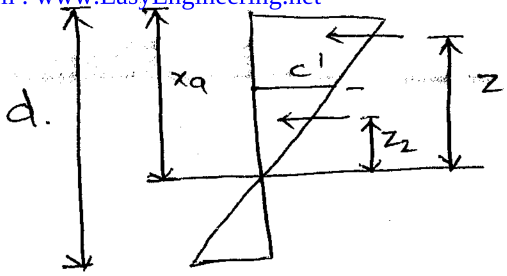
$$F_{c2} = \frac{1}{2} \times c' b_w (x_a - D_f)$$

$$\boxed{F_c = \left[\left(\frac{c+c'}{2}\right) \times b_f \times D_f\right] + \frac{1}{2} \times c' \times b_w (x_a - D_f)}$$

→ Force in Tension zone.

$$\boxed{F_t = t\,A_{st}}$$

→ Lever Arm:



$$z_1 = d - \bar{y}$$

$$\bar{y} = \frac{a+2b}{a+b} \times \frac{h}{3}.$$

$$\bar{y} = \frac{c+2c'}{c+c'} \times \frac{D_f}{3}.$$

$$\boxed{z_1 = d - \left[\left(\frac{c+2c'}{c+c'}\right) \times \frac{D_f}{3}\right]}$$

$$\boxed{z_2 = d - D_f - \left(\frac{x_a - D_f}{3}\right)}$$

→ M.R Due to compression Force.:

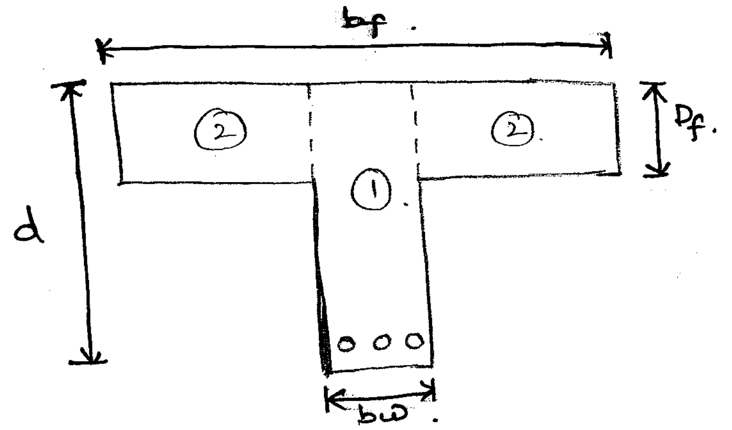
$$\boxed{\begin{aligned} M.R &= \left[\left(\frac{c+c'}{2}\right) \times b_f D_f \times \left(d - \left[\left(\frac{c+2c'}{c+c'}\right) \times \frac{D_f}{3}\right]\right)\right] \\ &+ \frac{1}{2} c' b_w \left(x_a - D_f\right) \times \left(d - D_f - \left(\frac{x_a - D_f}{3}\right)\right) \end{aligned}}$$

→. M.R Due to Tension zone:

$$\boxed{M.R = t A_{st} \left(d - \frac{x_a}{3}\right)}$$

ECONOMICAL DEPTH OF T – BEAM / (COST RATI

Rate Concrete = 5500
= 6500/m
1 m³ Conc: 180 kg
1 kg = 65



$$\text{Total cost} = (\gamma_c \times V_c) + (\gamma_s \times V_s)$$

$$= [\gamma_c \times A_c \times L] + [\gamma_s \times A_{st} \times L]$$

$$= \gamma_c \times \left[(b_w \times d) + [(b_f - b_w) \times D_f] \right] \times L + \left[\gamma_s \times \frac{M_{max}}{t \times j \times d} \times L \right]$$

$$C = \gamma_c \times L \left(\left[(b_w \times d) + [(b_f - b_w) \times D_f] \right] + \left(\frac{\gamma_s}{\gamma_c} \times \frac{M_{max}}{t \times j \times d} \right) \right)$$

$$\frac{\partial C}{\partial d} = 0$$

$$0 = \gamma_c \times L \left[b_w + \frac{\gamma_s}{\gamma_c} \frac{M_{max}}{t \times j} \left(\frac{-1}{d^2} \right) \right]$$

$$0 = b_w - \frac{\gamma_s}{\gamma_c} \frac{M_{max}}{tj} \left(\frac{1}{d^2} \right)$$

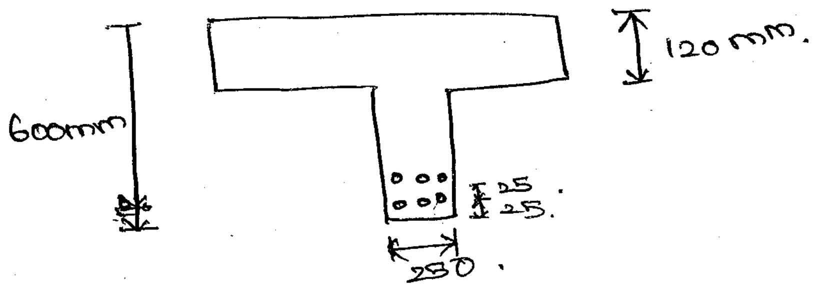
$$\boxed{d = \sqrt{\gamma_r \frac{M_{max}}{t j b_w}}}$$

$$\boxed{\gamma_r = \frac{\gamma_s}{\gamma_c} = 70 - 80\%}$$

30/9/2015

R.C.C.

1. An isolated T-beam $b = 1100\,mm$. and $D = 60$[...]
D_f = Thickness of slab, $D_f = 120\,mm$, $b_w = 250$[...]
There are 6 bars of 25mm dia in Tension zone. Determine M.R of T-beam if $l_o = 6m$. use M_{20} and Fe 250.



$$d' = 600 - 25 - \frac{25}{2} - 25$$

$$d' = 537.5.$$

$$b_{eff} = \frac{l_o}{\left(\frac{l_o}{b}\right)+4} + b_w = \left(\frac{6000}{\frac{6000}{1100}+4}\right) + 250.$$

$$b_f = 884.61\,mm.$$

$$\boxed{b_f = 885\ mm}$$

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times 6}$$

$$\boxed{m = 14}$$

$$\boxed{m = 13.33}$$

$$\frac{b_f x_a^2}{2} = m A_{st}\,(d - x_a).$$

$$\frac{885 \times x_a^2}{2} = 13.33 \times 2945.24\,(537.5 - x_a)$$

$$442.5\,x_a^2 + 39260.0492\,x_a - 21.102\times 10^6 = 0$$

$$\boxed{x_a = 178.47\,mm} > 120\,mm.$$

Horizontal spacing. (whichever is great)

→ ϕ bar

→ size of aggregate + 5mm.

Vertical spacing (whichever is great)

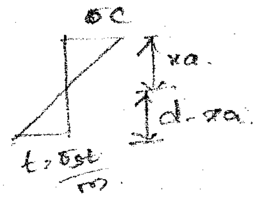
→ ϕ bar

→ $\frac{2}{3} \times$ size of aggregate

→ 15mm.

$x_c > x_d$

O.R.S



Note:

* In U.R.S. Steel yields first

$\therefore t = \sigma_{st}$ and $c \neq \sigma_{cbc}$.

we have to find out c from the following expression:

$$c = \frac{t \times x_a}{(d - x_a)} \qquad c = \frac{t \times x_a}{(d - x_a)}$$

* In O.R.S concrete yields first

$\therefore c = \sigma_{cbc} \qquad t \neq \sigma_{st}$

$$t = \frac{c\,(d - x_a)}{x_a} \qquad t = \frac{c\,(d - x_a)}{x_a}$$

$$b_f D_f \left(x_a - \frac{D_f}{2}\right) + \left(b_w \cdot \frac{(x_a - D_f)^2}{2}\right) = m A_{st} (d - x_a)$$

$$\left[885 \times 120 \left(x_a - \frac{120}{2}\right)\right] + \left[\frac{250}{2}(x_a - 120)^2\right] = 13.33 \times 2945.24 \, (537.5 - x_a)$$

$$\left.\begin{matrix}106200 x_a - 6.372 \times 10^6 + .125 x_a^2 - 30000 x_a \\ + 1.8 \times 10^6\end{matrix}\right\} = 13.33 \times 2945.24 \, (537.5 - x_a)$$

$$125 x_a^2 + 115460.0492 x_a - 25.674 \times 10^6 = 0$$

$$\boxed{x_a = 185.22 \text{ mm}}$$

$$M.R = \frac{c + c'}{2} \times b_f D_f \left[d - \left(\frac{c + 2c'}{c + c'}\right) \times \frac{D_f}{3}\right]$$

$$+ \frac{c'}{2} b_w (x_a - D_f) \times \left[d - D_f - \left(\frac{x_a - D_f}{3}\right)\right]$$

$$c' = \frac{c \times (x_a - D_f)}{x_a} = \frac{7 \times (185.22 - 120)}{185.22}$$

$$\boxed{c' = 2.465 \ N/mm^2}$$

$$= \frac{7 + 2.465}{2} \times 885 \times 120 \times \left[537.5 - \left[\frac{7 + 2(2.465)}{7 + 2.465} \times \frac{120}{3}\right]\right]$$

$$+ \frac{2.465}{2} \times 250 \times (185.22 - 120) \times \left(537.5 - 120 - \left(\frac{185.22 - 120}{3}\right)\right)$$

$$= (502591.5 \times 487.08) + 7.0953 \times 10^6$$

$$\boxed{M = 252.76 \times 10^6 \ N.mm}$$

$x_c = kd$ $\qquad k = \dfrac{280}{280+3\sigma_{st}} = 0.418$

$= 0.418 \times 537.5$

$x_c = 224.626 > x_a$

$\therefore \; c < \sigma_{cbc} \qquad t = \sigma_{st}$

$$\frac{c}{\sigma_{st}/m} = \frac{x_a}{d - x_a}$$

$$c = \frac{185.22}{(537.5 - 185.22)} \times \frac{180}{13}$$

$$\boxed{c = 5.257}$$

$$c' = \frac{c(x_a - D_f)}{x_a} = \frac{5.257(185.22-120)}{185.22}$$

$$\boxed{c' = 1.85}$$

$$M.R = \frac{c+c'}{2} b_f D_f \left(d - \frac{c+2c'}{c+c'} \times \frac{D_f}{3}\right) + \frac{c'}{2} \times b_w (x_a - D_f) \times \left[d - D_f - \left(\frac{x_a - D_f}{3}\right)\right]$$

$$= \left(\frac{5.257+1.85}{2}\right) \times 885 \times 120 \left(537.5 - \left(\frac{5.257+2(1.85)}{5.257+1.85} \times \frac{120}{3}\right)\right)$$

$$+ \frac{1.85}{2} \times 250 \left(185.22 - 120 - \left(\frac{185.22-120}{3}\right)\right)$$

$$= 377381.7 \times 487.08 + 10.05 \times 10^3$$

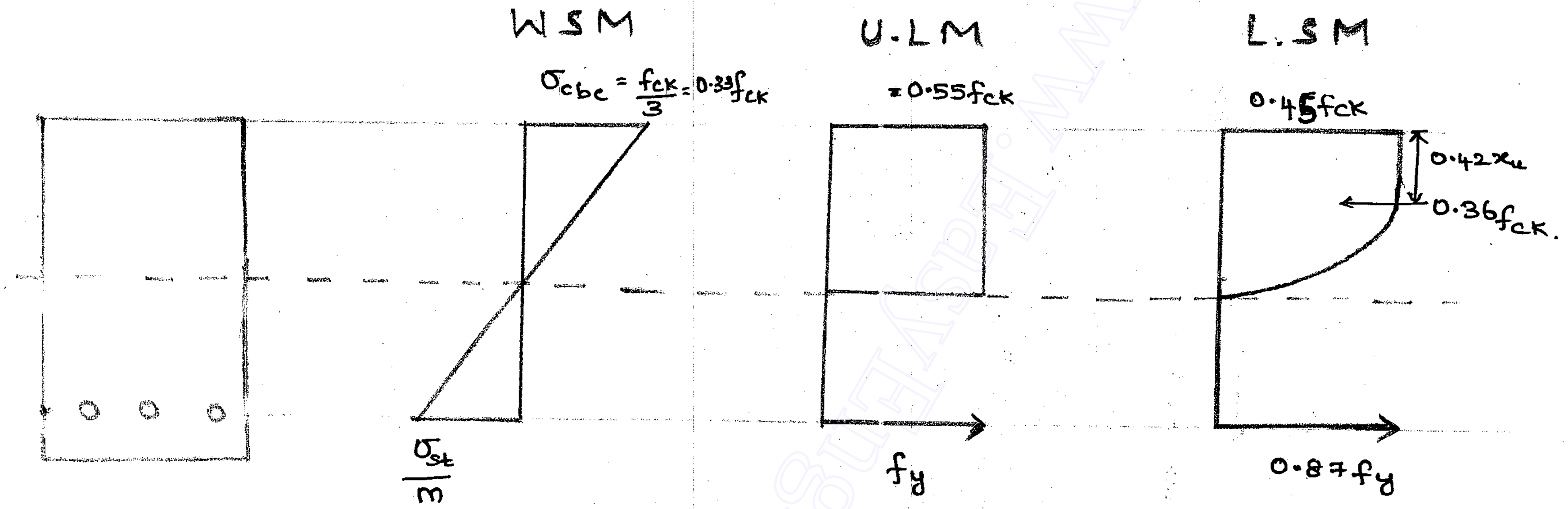
$$\boxed{M.R = 183.82 \times 10^6 \text{ N.mm}}$$

1/10/2015

R.C.C.

LIMIT STATE METHOD

LIMIT STATE OF COLLAPSE



$\sigma_{cbc} = \frac{fck}{3} = 0.33f_{ck}$

$\sigma_{st} = \frac{f_y}{1.8} = 0.55f_y$

$\frac{f_{ck}}{1.8} = 0.55f_{ck}$

$f_y = 1f_y$

$\frac{f_y}{1.15} = 0.87f_y$

ULM

* ULM was developed in 1950.

Type of Load	Live Load	Dead Load
F.O.S.	2.2	1.5

Material	Steel	Concrete
Permissible stress	100% (f_y)	55% ($\frac{f_{ck}}{1.8}$)

* Shape of stress block daigram - Rectangle.
  Depth of stress block daigram - 0.43d.

* ULM was specified in IS456:1964.

* ULM Safe - collapse condition. Unsafe - Serviciability Condition

* WSM. Safe - servicable condition Unsafe - collapse condition.

LSM.

* It is a rationalized method of ULM.
* LSM was developed in 1970
* Adopted in IS 456:1978 and revised code IS 456:2000 and used for R.C.C structure.

* In LSM
  B.M. is increased by 50%.
  M.R is increased by 200%.

* LSM

Limit state of collapse
* Used to determine. Factored B.M & Factored Load.
* To determine b, d, Ast, stirrups, check for sliding, overturning, buckling, fatigue.

Limit state of serviceability
* used to check for deflection, crack, leakage, vibration.

* LSM ~~was~~ uses multiple safety factor called partial safety factor (i.e) ~~8~~ F.O.S seperately for load and material

Material	Steel	Concrete.
Partial safety factor	1.15	1.5

* LSM is a rationalized method of ULM.
* ULM was developed in 1950, where Load factor for Live load was 2.2 and Dead Load 1.5.
* In ULM the permissible stress for steel was 100% fy for concrete 55% fck and the stress block was rectangle.
  Depth = 0.43d.
* ULM was specified in IS 456 : 1964
* ULM was safe for collapse condition but unsafe for serviciability condition.
* WSM is safe for service condition but it is unsafe for collapse condition
* LSM has been developed to take care of safety @ collapse and serviceability condition.
* LSM was developed in 1970. and adopts by IS 456 in 1978.
* Now IS 456:2000 is widely used for R.C.C structure.
* In LSM the extral bending moment is increased by 50% but M.R of the section increases to 200%
* There are 2 methods of LSM.
  → Limit state collapse
  → Limit state of serviceability.

* Partial safety factor is dependent on Load combination. (collapse load condition)

→ 1.5 D.L + 1.5 L.L

→ 1.5 D.L + 1.5 W.L

→ 1.5 D.L + 1.5 E.L

→ 0.9 D.L + 1.5 W.L (Reversal of stresses and also for overturning check)

→ 1.2 D.L + 1.2 L.L + 1.2 W.L

→ 1.2 D.L + 1.2 L.L + 1.2 E.L.

→ ~~1.2 D.L + 1.2 L.L + 1.2 E.L + 1.2 W.L.~~ (wind and Earthquake load do not occur at the same time as per IS 456:2000

* Partial safety factor of Limit state of serviceability.

(a) D.L + L.L

(b) D.L + W.L.

(c) D.L + E.L

(d) D.L + 0.8 L.L + 0.8 W.L

(e) D.L + 0.8 L.L + 0.8 E.L.

* Max. permissible strain in concrete. in ben

$$\epsilon_{cbc} \not> 0.0035. \quad \text{(flexural compression)}$$

* $$\epsilon_{cbc} \not> 0.002 \quad \text{(axial compression)}$$

* $$\epsilon_{st} \not> 0.002 + \frac{0.87 f_y}{E_s} \text{ (or) } 0.002 + \frac{\left(\frac{f_y}{\gamma_{m}}\right)}{E_s}.$$

* Limit state of collapse is used to design an R.C.C structure based on collapse Load. (Factored B.M & Factored Load (i.e) to.
* determine breadth, depth, A_{st}, stirrusp, Check for sliding, check for overturning, buckling, fatigue).

* Limit state of serviceability is used to check deflection, crack, leakage, vibration.

* LSM uses <u>multiple safety factor</u> (i.e) seperately for <u>load</u> and for <u>material</u>. This safety factor is also called as <u>partial safety factor</u>.

* The partial safety factor for concrete is <u>1.5</u> and for steel <u>1.15</u>.
* The partial safety factor load is depender on load combination.

(a) 1.5 DL + 1.5 LL (IL)

(b) 1.5 DL + 1.5 WL

(c) 1.5 DL + 1.5 EL.

(d) <u>0.9 DL + 1.5 WL</u> - Reversal of stresse (or) overturning check.

(e) 1.2 DL + 1.2 L.L + 1.2 W.L

(f). 1.2 DL + 1.2 L.L + 1.2 E.L

~~(g) 1.2 DL + 1.2 L.L + 1.2 WL + 1.2 EL.~~

wind and Earthquake Load do not occur. at the same time as per IS 456: 2000.

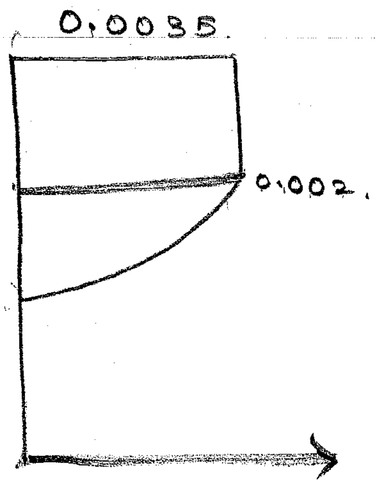
* The partial factor of safety of Limit state of serviceability.

a) 1.0 DL + 1.0 L.L
(b) 1.0 DL + 1.0 W.L
(c) 1.0 DL + 1.0 E.L
(d) 1.0 DL + 0.8 L.L + 0.8 W.L
(e) 1.0 DL + 0.8 L.L + 0.8 E.L

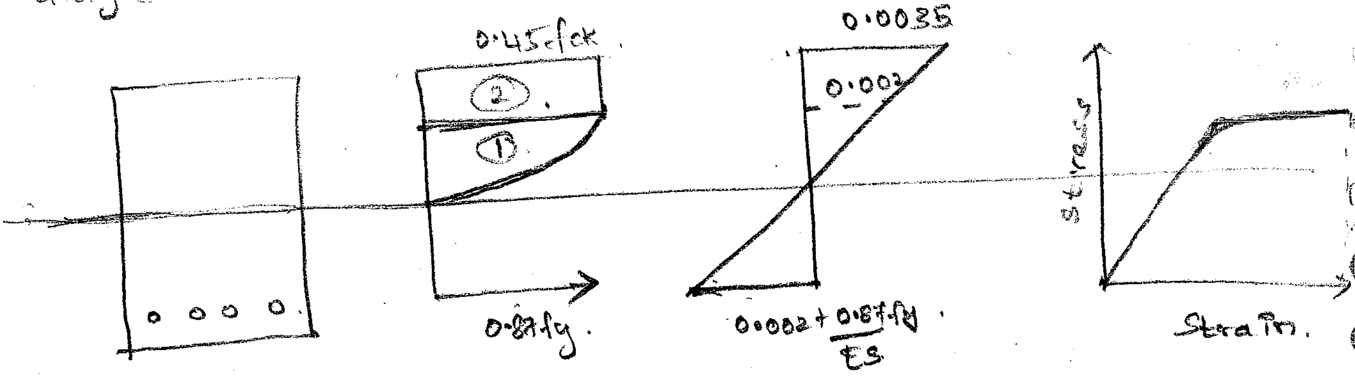
* The maximum permissible strain in concrete in bending compression (flexural compression) $\epsilon_{cbc} \ngtr$ 0.0035.

* The maximum permissible strain in concrete in axial compression. $\ngtr$ 0.002

* The strain in tension steel. shall not be less than. $0.002 + \frac{0.87 f_y}{E_s}$.



Limit State Method Stress Block & strain block diagram



* Characteristic strength of concrete in L.S.M } = 0·85 compressive strength of cylinder

= 0·85 × (0·8 × cube strength of Concrete)

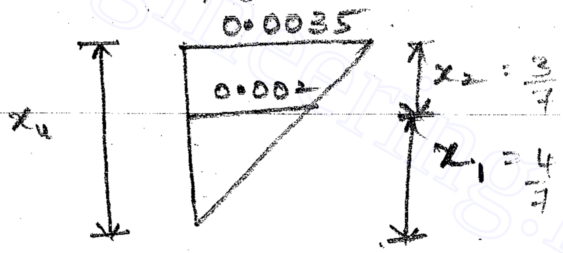
= 0·67 cube strength of concrete.

* Design strength of concrete for LSM } = $\frac{0{\cdot}67\,f_{ck}}{1{\cdot}5} = 0{\cdot}45 f_{ck}$.

* Design strength of steel for LSM = $\frac{f_y}{1{\cdot}15} = 0{\cdot}87 f_y$.

* Average stress.

$$\boxed{\begin{aligned} x_1 &= \frac{4}{7} x_u \\ x_2 &= \frac{3}{7} x_u \end{aligned}}$$



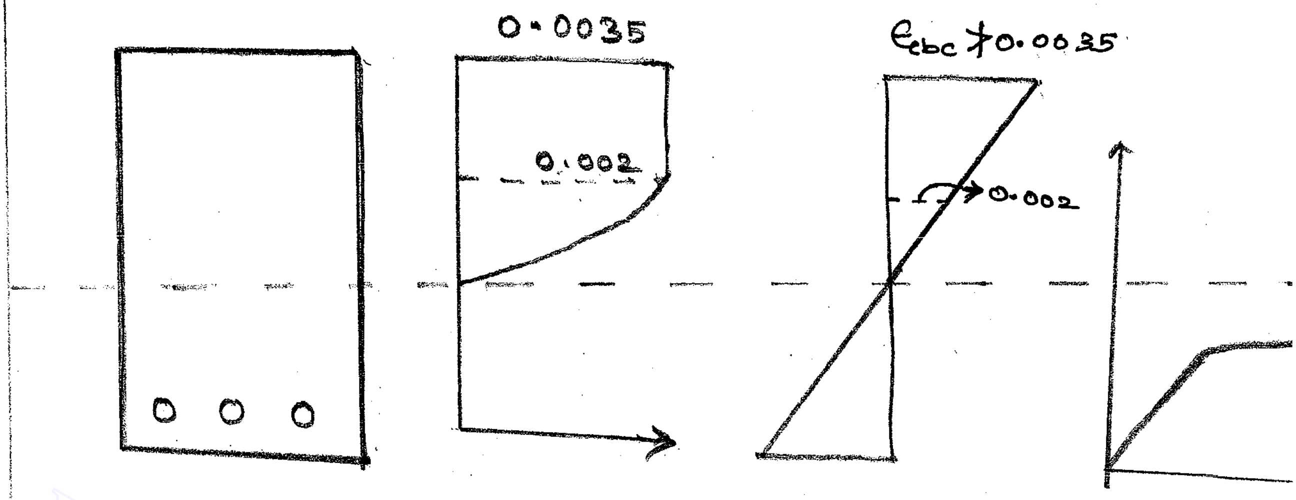
$$\boxed{f_{avg} = 0{\cdot}36 f_{ck}}$$

* Expression for critical neutral axis. (from strain diagram)

$$\frac{E_{cbc}}{x_{umax}} = \frac{E_{cbc}}{E_{cbc} + E_{st}} \times d = fd.$$

Grade of steel	$\frac{x_{umax}}{d}$
Fe 250	0·53
Fe 415	0·48
Fe 500	0·46

Limit state Method stress Block & strain block diagram.



* The characteristic strength of concrete for limit state = 0.85 times of compressive strength of cylinder.

cylinder strength = 0.8 times of cube stren

∴ ~~fck~~ characteristic strength of concrete for L.S.M = $0.8 \times (0.85 f_{ck})$.

$= \underline{0.67 f_{ck}}$.

The design strength of concrete for LSM $\Big\} = \dfrac{0.67 f_{ck}}{\gamma_{mc}}$

$= \dfrac{0.67 f_{ck}}{1.5}$.

Design strength of concrete for LSM $\Big] = 0.45 f_{ck}$.

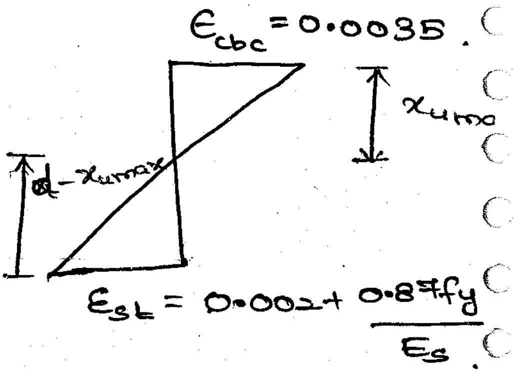
* The design strength of Steel according to LSM $\left.\right\} = \dfrac{\text{Characteristic strength of Steel}}{\text{Partial safety factor}}$

$$= \frac{f_y}{1.15}$$

The design strength of steel according to LSM $\left.\right\} = 0.87 f_y = \dfrac{f_y}{1.15}$.

EXPRESSION FOR CRITICAL NEUTRAL AXIS:

It is based on strain daigram which is linear one.



By $\parallel^{rd}$ triangle property.

$$\frac{\varepsilon_{cbc}}{\varepsilon_{st}} = \frac{x_{umax}}{d - x_{umax}}.$$

$$\varepsilon_{cbc}(d - x_{umax}) = \varepsilon_{st}\, x_{umax}.$$

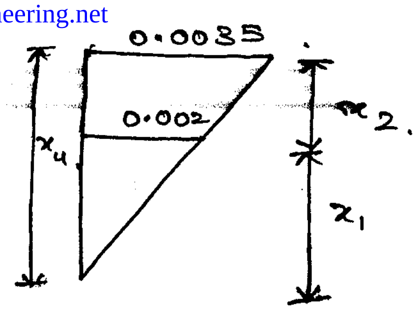
$$\varepsilon_{cbc}\, d = (\varepsilon_{cbc} \times x_{umax}) + (\varepsilon_{st} \times x_{umax})$$

$$x_{umax}(\varepsilon_{cbc} + \varepsilon_{st}) = \varepsilon_{cbc}\, d.$$

$$\boxed{\begin{aligned} x_{umax} &= \left(\frac{\varepsilon_{cbc}}{\varepsilon_{cbc} + \varepsilon_{st}}\right) \times d \\ x_{umax} &= K \times d. \end{aligned}}$$

Grade of Steel	$\frac{x_{umax}}{d}$.
Fe 250	0.53
Fe 415	0.48
Fe 500	0.46.

By ||^{el} triangle property.



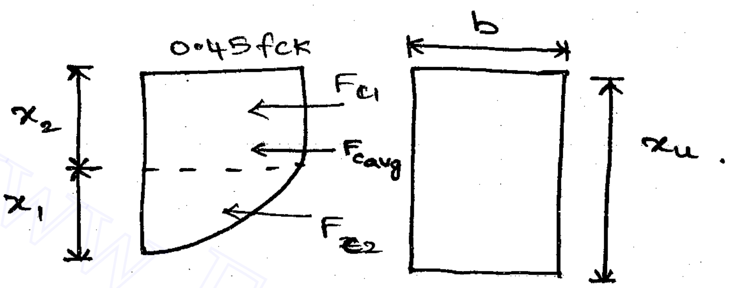
$$\frac{0.0035}{x_u} = \frac{0.002}{x_1}$$

$$x_1 = \frac{0.002}{0.0035} \times x_u$$

$$\boxed{x_1 = \frac{4}{7} x_u}$$

$$\boxed{x_2 = \frac{3}{7} x_u}$$

Average ~~stress~~:



$$F_{c\,avg} = F_{c_1} + F_{c_2}.$$

$$f_{cavg} \times b \times x_u = (f_{c_1} \times A_1) + (f_{c_2} \times A_2)$$

$$= \left(\frac{2}{3} \times 0.45 f_{ck} \times b x_1\right) + (0.45 f_{ck} \times b \times x_2)$$

$$= b\left[\frac{2 \times 0.45}{3} f_{ck} \times \frac{4 x_u}{7} + 0.4 f_{ck} \times \frac{3 x_u}{7}\right]$$

$$f_{cavg} \times \cancel{b} \times \cancel{x_u} = \cancel{b} \times \cancel{x_u} \left[\frac{3.6}{21} f_{ck} + \frac{1.2}{7} f_{ck}\right]$$

$$f_{avg} = 0.36\, f_{ck}.$$

$$\boxed{f_{avg} = 0.36 f_{ck}}$$

Note:

To increase the bond strength economically we have to provide more no. of smaller dia bars.

Grade of Steel	Strain in Extreme fiber.
Fe 250	0.0030
Fe 415	0.0038
Fe 500	0.0041

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SPACING OF REINFORCEMENT;

1 * Minimum spacing b/w 2 bars shall be greater of the following.

1.) ϕ bar (If ϕ of bars are of equal ϕ).

2.) ϕ of largest bar.

3.) 5mm + nominal size of aggregate.

→ If needle vibrator is used compact concrete,

1.) $\frac{2}{3}$ × ~~size~~ nominal max. size of aggregate.

* Minimum vertical distance : (whichever is greater)

1.) 15 mm (or) $\frac{2}{3}$ x max. size of aggregate

(or) Max. size of bar.

* Maximum distance b/w bars in tension depending on cracking of concrete.

1.) Max. crack width in mild and aggressive environment

2.) 0.3mm and 0.1 mm

* side face Reinforcement.

→ If depth of the web in a beam exceeds 750mm, side face reinforcement should be provided along the two faces.

→ Total Area of side face reinforcement. } = 0.1% of web area.

It should be equally distributed on both faces. and spacing of reinforcement should not exceed 300mm c/c.

* Deep beams.

$\frac{l}{d} \ngtr 2$ simply supported

$\frac{l}{d} \ngtr 2.5$. continous beam.

1.) Minimum percentage of reinforcement :

Fe 415.

$$P_t = \frac{0.85 \times 100}{f_y} = \frac{0.85 \times 100}{415} = 0.2\%$$

Fe 250

$$P_t = \frac{0.85 \times 100}{250} = 0.34\%$$

Fe 500

$$P_t = \frac{0.85 \times 100}{500} = 0.17\%$$

2). Assume straight line instead of parabola for stress strain for curve of concrete. as follows:

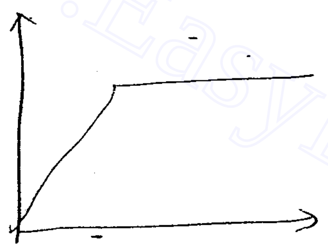
F.O.S = 1. Rectangular beam.

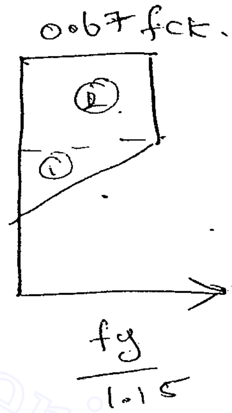
$b = 300\,mm$, $d = 500\,mm$.

with 3 bars of 16 mm ϕ M_{20}, Fe415.

Depth of N-A is from the compressive fiber is ________. Depth of the N.A. as obtained by IS 456 2000. and the difference obtained above is ________.

case (i)





$$F_c = F_t$$

$$F_{c_1} + F_2 = 0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 16^2$$

$$\left(0.67 f_{ck} \times \frac{1}{2} \times b \times \frac{4x_u}{7}\right) + \left[0.67 f_{ck} \times b \times \frac{3x_u}{7}\right] = 217.78$$

$$\left[\frac{1}{2} \times \frac{4x_u}{7}\right] + \left[\frac{3x_u}{7}\right] = \frac{217.78}{0.67 \times 20 \times 300}$$

$$x_u = 54.174 \times \frac{7}{5}$$

$$\boxed{x_u = 75.84\ mm}$$

$x_u = 75.8$

case (ii) : x_u $\qquad$ $x_u = 99.3$

$$F_c = F_{st}$$

$$\cancel{0.67\times}.\ F_{c_1} + F_{c_2} = F_{st}.$$

$$\left[0.45 f_{ck} \times \frac{2}{3} \times b \times \frac{4}{7} x_u\right] + \left[0.45 f_{ck} \times b \times \frac{3}{7} \times x_u\right]$$

$$= 0.87 f_y A_{st}$$

$$\left[0.45 \times 20 \times \frac{2}{3} \times 300 \times \frac{4}{7} x_u\right] + \left[0.45 \times 20 \times 300 \times \frac{3}{7} x_u\right]$$

$$= 0.87 \times 415 \times \frac{\pi}{4} \times 16^2 \times 3.$$

$$1028.57 x_u + 1157.143 x_u = 217.780 \times 10^3.$$

$$\boxed{x_u = 99.64\,mm}$$

$$\text{Difference} = 99.64 - 75.84$$

$$= 23.8\ mm.$$

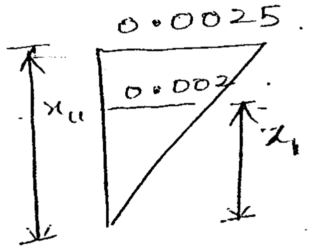
2.) In design of beam for L.S of collapse and flexure as per IS 456:2000. Strain in concrete is ~~limited to~~ limited to 0.0025. (Instead of 0.003 For this situation consider a rectangu beam $b = 280mm$ $d = 350mm$. $A_{st} = 15$ mm

Use Fe ~~415~~ 250, M30.

(a) Depth of N.A for balanced failure. 156 mm

(b) @ L.S. of collapse in flexure the Force acting on compression zone is ___

Solution:



$$\frac{x_1}{x_u} = \frac{0.002}{0.0025}$$

$$x_1 = \frac{4}{5} x_u$$

$$x_2 = \frac{1}{5} x_u$$

$$F_c = F_t$$

$$\left[0.45 f_{ck}\, b\, \cancel{x_u} \times x_1 \times \frac{2}{3}\right] + 0.45 f_{ck} \times b \times x_2 \cancel{= 0.87 \times f_y \times A_{st}}$$

$\cancel{0.45 \times 30 \times}$

$$x_{umax} = \left(\frac{\varepsilon_{ctc}}{\varepsilon_{ctc} + \varepsilon_{st}}\right) \times d$$

$$= \left[\frac{0.0025}{0.0025 + \left(\frac{0.87 \times 250}{2 \times 10^5}\right) + 0.002}\right] \times 350$$

$x_{umax} = 156.59$ mm

$$F_c = 0.45 f_{ck} \times \frac{2}{3} b x_1 + 0.45 \times b x_2 . f_{ck}$$

$$= \left[0.45 \times 30 \times \frac{2}{3} \times 250 \times \frac{4}{5} x_u\right] + \left[30 \times 0.45 \times 250 \times \frac{x_u}{5}\right]$$

$$= \left[0.45 \times 30 \times \frac{2}{3} \times 250 \times \frac{4}{5} \times 156.59\right] + \left[30 \times 0.45 \times 250 \times \frac{156.59}{5}\right]$$

$$= 281880 + (105.705 \times 10^3)$$

$$= 387.583 \text{ kN}$$

$$f_t = 0.87 f_y A_{st}$$

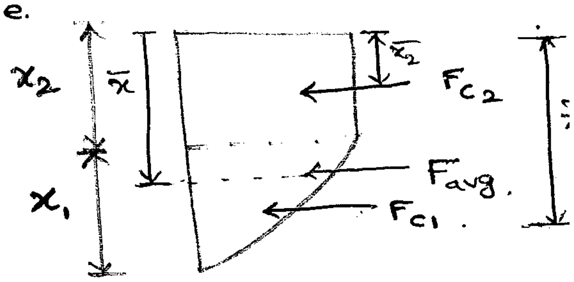
$$= 0.87 \times \cancel{415}\,250 \times 1500$$

$$= 326.25 \text{ kN}$$

LEVER ARM:

According to Varignon's Principle.

$$\boxed{F_{avg} \times \bar{x} = F_{c1} \times \bar{x}_1 + F_{c2} \times \bar{x}_2}$$



$$F_{avg} = 0.36 f_{ck} \times b \times x_u$$

$$F_{c1} = \frac{2}{3} \times 0.45 f_{ck} \times b \times x_u$$

$$\bar{x}_1 = \frac{3x_u}{7} + \frac{3}{8} \times \frac{4}{7} x_u = \frac{9}{14}$$

$$F_{c2} = 0.45 f_{ck} \times b \times x_u$$

$$\bar{x}_2 = \frac{1}{2} \times \frac{3}{7} x_u = \frac{3}{14} x_u$$

$$0.36 f_{ck}\, b \times x_u \times \bar{x} = \left[\frac{2}{3} \times 0.45 \times f_{ck} \times b \times \frac{4}{7} x_u \times \frac{9}{14} x_u\right]$$

$$+ \left[0.45 f_{ck} \times b \times x_u \frac{3}{7} \times \frac{3}{14} x_u\right]$$

$$0.36\, \cancel{f_{ck}} \times \cancel{b} \times \cancel{x_u} \times \bar{x} = \cancel{f_{ck}}\, \cancel{b}\, x_u^{\cancel{2}} \left[0.1102 + 0.041\right]$$

$$\bar{x} = \frac{x_u}{0.36}\left[0.1102 + 0.041\right]$$

$$= x_u \frac{0.1515}{0.36}$$

$$\boxed{\bar{x} = 0.42 x_u}$$

Lever arm $= d - \bar{x}$

$$\boxed{\text{Lever arm} = d - 0.42 x_u}$$

x_u lim :

$$F_c = F_t.$$

$$0.36 f_{ck} \times b \times x_{a\,max} = 0.87 f_y A_{st}.$$

For Balanced section:

$$0.36 f_{ck} \times b \times x_{u\,max} = 0.87 f_y A_{st}.$$

$$\boxed{\frac{A_{st}}{bd} \times 100 = \frac{0.36 f_{ck} x_{u\,max} \times 100}{0.87 f_y d}.}$$

$$\boxed{P_{t\,lim} = \frac{0.36 f_{ck}}{0.87 f_y} \times 100 \times} \frac{x_{u\,max}}{d}.$$

ANALYSIS OF SINGLY REINFORCED SECTION BY LIMIT STATE METHOD.

(i) To Find x_u

compressive force, $F_c = 0.36 f_{ck} b x_u$.

Tensile force, $F_t = 0.87 f_y \times A_{st}$.

$$F_c = F_t.$$

$$0.36 f_{ck} b x_u = 0.87 f_y A_{st}$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}.$$

$$\boxed{x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}.}$$

$$\boxed{\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b d}.}$$

(ii) Limiting Neutral Axis: (x_{ulim} (or) x_{umax}).

For		$\frac{x_{umax}}{d}$.
	Fe 250	0.5
	Fe415	0.48.
	Fe500	0.46.

(iii) Lever arm

$$\boxed{z = d - 0.42 x_u.}$$

(iv) Moment of Resistance.

$$M.R = 0.36 f_{ck}\ b \times x_u\ (d - 0.42 x_u).$$

For Balanced section

$$\boxed{x_u = x_{ulim}}.$$

$$M.R_b = 0.36 f_{ck} b \times \frac{d}{d} \times x_{umax}\, d \left(1 - 0.42\frac{x_{umax}}{d}\right)$$

$$M \cdot R = 0.36 f_{ck}\, bd^2 \frac{x_{umax}}{d}\left[1 - 0.42\frac{x_{umax}}{d}\right]$$

$$\boxed{M_{ulim} = 0.36 f_{ck}\, bd^2 \frac{x_{umax}}{d}\left[1 - 0.42\frac{x_{umax}}{d}\right]}$$

Moment of Resistance of Under Reinforced Section :

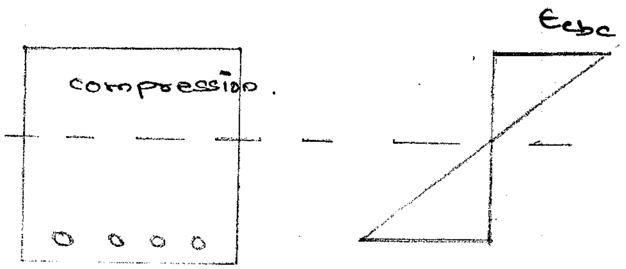
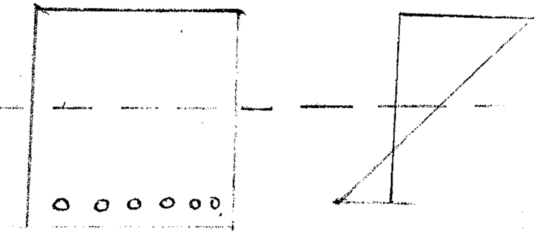
$$M \cdot R = 0.87 f_y A_{st} (d - 0.42 x_u)$$

$$= 0.87 f_y A_{st}\left[d - \frac{0.4 \times 0.87 \times f_y A_{st}}{0.36 f_{ck} b}\right]$$

$$= 0.87 f_y A_{st}\, d\left[1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right]$$

$$\boxed{M \cdot R = 0.87 f_y A_{st}\, d\left[1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right]}$$

Annexure G in 1:1

Balanced Section.	Under Reinforced Section	Over Reinforced section.
		
* Concrete and steel both attain max. permissible strain simultaneously	* Strain in Steel > Strain in concrete	* In this section steel does not attains it max. permissible strain.
* $x_u = x_{umax}$	* Strain in steel > E_{st}	* Actual stress in steel < E_{st}.
* $P_{t\,actual} = P_{t\,lim}$	* $x_u < x_{umax}$	* $x_u > x_{umax}$.
* $M_{ulim} = 0.36 f_{ck}\, b\, x_{umax}\,[d - 0.42 x_{umax}]$	* $P_{t,actual} < P_{t\,lim}$.	* $P_{t\,actual} > P_{t\,lim}$.
$M_{ulim} = 0.87 f_y A_{st}\,[d - 0.42\, x_{umax}]$	* $M_u = 0.87 f_y A_{st}\, d \left[1 - \frac{f_y A_{st}}{f_{ck} b d}\right]$	* ~~M_u = [illegible]~~
* $M_{ulim} = M_u$	* $M_u < M_{ulim}$	* $M_u = f_{st} A_{st}\,(d - 0.42 x_u)$.
		* $M_u > M_{ulim}$
		* As per IS 456 2000 over reinforced section is not at all allowed hence it is designed as Balanced section (or) Doubly reinforced section.

1\. A singly reinforced □^{ce} concrete beam has a b = 150mm and d = 330mm. fck = 20 N/mm² fy = 415 MPa. Adopt the stress block diagram for concrete as given in IS 456:2000 and take xu lim = 0.48d. What is the Mulim in kN.m. Limiting Area of tension steel.

solution:

$$M.R = 0.36 f_{ck} bd^2 \frac{x_{umax}}{d}\left[1 - 0.42\frac{x_{umax}}{d}\right]$$

$$= 0.36 \times 20 \times 150 \times (330)^2 \times 0.48\left[1 - 0.42 \times 0.48\right]$$

$$\boxed{M_{ulim} = 45.07 \text{ kN.m}}$$

$$\cancel{M_{ulim} = 0.87 f_y}.$$

$$\frac{A_{st}}{bd} = \left[\frac{0.36 f_{ck}}{0.87 f_y}\cdot\frac{x_{umax}}{d}\right]$$

$$\frac{A_{st}}{150 \times 300} = \frac{0.36 \times 20 \times 0.48}{0.87 \times 415}$$

$$A_{st} = 473.81$$

$$\boxed{A_{st} = 475 \text{ mm}^2}$$

Concrete Grade	Mix Proportion	Mix Method	f_{ck} @ 28 days	$\sigma_{cbc} = \frac{f_{ck}}{3}$	$\sigma_{cc} = \frac{f_{ck}}{4}$	τ_{bd} (Fe 250)	m $\left(\frac{280}{3\sigma_{cbc}}\right)$	Purpose.
M_5	1:5:10	Nominal mix.	5	–	–	–		* Lean concreting – Foundation
$M_{7.5}$	1:4:8	Nominal	7.5	–	–	–		* Lean concreting
M_{10}	1:3:6	Nominal	10	3	2.5	–		* Mass concreting (ordinary concreting)
M_{15}	1:2:4	Nominal	15	5	4	0.6	19 (18.67)	* Mass concreting (As per IS 456:2000) ordinary concreting (As per IS 456: 1978)
M_{20}	1:1.5:3	Nominal	20	7	5	0.8	13	* Ordinary R.C.C, Mild Exposure.
Standard concrete								
M_{25}	1:1:2	Design mix	25	8.5	6	0.9	11	* Water Tank, Foundation, Moderate Exposure.
M_{30}	–	Design mix	30	10	8	1	9.3	* Minimum grade of Post tensioned concrete, Severe. exposure.
M_{35}	–	Design mix	35	11.5	9	1.1	8	* Aggressive sub soil, Very Severe exposure
M_{40}	–	Design mix	40	13	10	1.2	7	* Pre tensioned concrete, Extreme exposure
M_{45}	–	Design mix	45	14.5	11	1.3.		* Pre tensiond
M_{50}	–	Design mix	50	16	12	1.4		

STEEL STRUCTURES

→ Analysis and design of tension and Compression members, beam-column and column bases.

→ Connections:

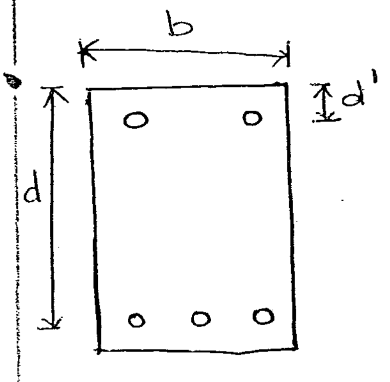
* Simple and eccentric
* Beam - column connections
* plate girders and trusses.

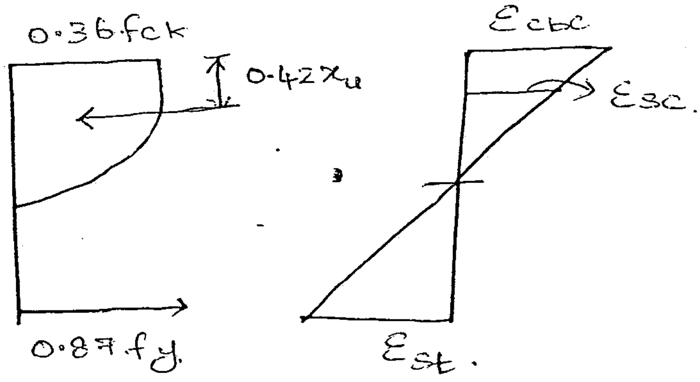
→ Plastic analysis of beams and frames.

22/10/2015

R.C.C.

DOUBLY REINFORCED BEAM (L.S.M):





By similar triangle property.

$$\frac{\varepsilon_{sc}}{\varepsilon_{cbc}} = \frac{(x_u - d')}{x_u}$$

$$\boxed{\varepsilon_{sc} = \frac{\varepsilon_{cbc}(x_u - d')}{x_u}}$$

$$\varepsilon_{sc} \times E = f_{sc}$$

$$f_{sc} \not> 0.87 f_y$$

$$F_c = 0.36 f_{ck}\, b\, x_u + f_{sc} A_{sc}$$

$$F_t = 0.87 f_y \cdot A_{st}$$

$$M.R = 0.36 f_{ck}\, b\, x_u (d - 0.42 x_u) + f_{sc} A_{sc} (d - d')$$

x_u:

$$F_c = F_t$$

$$0.36\, x_u\, b\, f_{ck} + f_{sc} A_{sc} = 0.87 f_y A_{st}$$

Take $x_u = 0.7 - 0.8\,(x_{umax})$

$$\boxed{x_u = 0.75\, x_{umax}}$$

STEPS TO SOLVE DOUBLY REINFORCED BEAM:

1. Assume. $x_u = 0.7$ to $0.8\ x_{umax}$

2. Find out ε_{sc}.

3. Find out f_{sc}. where

$$\boxed{f_{sc} = \varepsilon_{sc} \times E_s}$$ - for mild steel only

→ For deformed bar find ε_{sc} and find f_{sc} from given graph.

4.) Find out ~~the~~ actual value of x_u by equating $F_c = F_t$.

$$\boxed{0.36 f_{ck} b x_u + f_{sc} A_{sc} = 0.87 f_y A_{st}}$$

5.) After finding value of x_u from above equation compare it with assumed value of x_u. If both are equal then no need to take next trial. But if not take some more trial

6.) Find out $x_{u\,max}$.

$x_{u\,max}$	Fe
0.53d	250
0.48d	415
0.46d	500

7.) Compare x_u and x_{umax} and decide the type of section.

8.) Find $M_{u\,lim}$,

9.) If $M_{external} > M_{ulim}$ then the section must be designed as balanced doubly reinforced section.

where, $x_u = x_{umax}$ and

M.R is given by the formula,

$$M_u = 0.36 f_{ck} b x_{umax} (d - 0.42 x_{umax}) + f_{sc} A_{sc} (d - d')$$

$$= 0.36 f_{ck} b \times d \times d \frac{x_{umax}}{d} \left(d - 0.42 \frac{x_{umax}}{d}\right) + f_{sc} A_{sc} (d - d')$$

$$M_u = M_{u\,lim} + f_{sc} A_{sc} (d - d')$$

$$\boxed{A_{sc} = \frac{M_u - M_{u\,lim}}{f_{sc} (d - d')}}$$

10.) Find out Ast from

$$F_c = F_t$$

$$0.36\, x_u b f_{ck} = 0.87 f_y A_{st1}$$

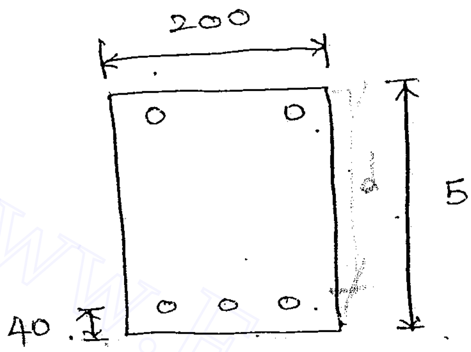
$$\boxed{f_{sc} A_{sc} = 0.87 f_y A_{st2}}$$

$$\boxed{M_u = 0.87 f_y A_{st1} (d - 0.42 x_{umax})}$$

$$\boxed{A_{st} = A_{st1} + A_{st2}}$$

1. Size of an R.C.C beam is restricted to b 250 × 500 mm. It carries I.L = 25 kN/m $\bar{l}$ = 6 m. Determine the reinforcement. Use M_{20}, Fe 415. Use following table given.

Stress.	0.8 fyd	0.85 fyd	0.9	0.95	0.975	1 fyd
Strain.	0.00144	0.00163	0.00192	0.0024	0.00276	0.003



d = 500 − 40

= 460 mm.

$$M_{u\,lim} = 0.36 f_{ck}\, b d^2 \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)$$

$$= 0.36 \times 20 \times 250 \times 460^2 \times 0.48\,(1 - 0.42\,(0.48))$$

$$M_{u\,lim} = 145.96 \text{ kN.m}$$

$$W_u = 1.5\,(D.L + I.L)$$

$$= 1.5\,((0.25 \times 0.5 \times 25) + (25))$$

$$\boxed{W_u = 42.18 \text{ kN}}$$

$$M_u = \frac{W_u l^2}{8}$$

$$= \frac{42.18 \times 6^2}{8}$$

$$M_u = 189.84 \text{ kN.m}$$

$$M_u > M_{u\,lim}$$

∴ Design as doubly reinforced beam.

$$\varepsilon_{sc} = \frac{\varepsilon_{cbc}(x_{umax} - d')}{x_{umax}}$$

$$= \frac{0.0035((0.48 \times 460) - 40)}{(0.48 \times 460)}$$

$$\underline{\varepsilon_{sc}} = 0.002865$$

$$\underline{f_{sc}} = \underline{0.976\, f_{yd}}$$

$$= 0.976 \times 0.87 \times 415$$

$$f_{sc} = 352.39 \ N/mm^2$$

$$M_u - M_{ulim} = f_{sc} A_{sc} (d - d')$$

$$A_{sc} = \frac{(189.84 - 145.96) \times 10^6}{352.39 \times (460 - 40)}$$

$$\underline{A_{sc}} = 296.47 \ mm^2$$

$$F_c = F_t$$

$$(0.36 f_{ck}\, b\, x_u) + f_{sc} A_{sc} = 0.87 f_y A_{st}$$

$$(0.36 \times 20 \times 250 \times 0.48 \times 460) + (352.39 \times 296.47) = 0.87 \times 415 \times A_{st}$$

$$\boxed{A_{st} = 1390 \ mm^2}$$

$$M_{ulim} = 0.87 f_y A_{st_1} (d - 0.42 x_{umax})$$

$$145.96 \times 10^6 = 0.87 \times 415 \times A_{st_1} (460 - 0.42(0.48 \times 460))$$

$$A_{st_1} = 1100.75 \ mm^2$$

$$f_{sc} A_{sc} = 0.87 f_y . A_{st2}.$$

$$352.39 \times 296.47 = 0.87 \times 415 \times A_{st2}$$

$$A_{st2} = 289.35 \text{ mm}^2.$$

2. 200 × 450 mm is reinforced with 4 # 20 mm in Tension 3 # 12 mm in compression Determine M_u lim use M_{20} and ~~Fe~~ Fe 250.

$$x_u = 0.75 \times 0.53 \times 410.$$

$$x_u = 162.975 \text{ mm}.$$

$$\varepsilon_{sc} = \frac{\varepsilon_{cbc}(x_u - d')}{x_u}$$

~~$M_u = 0.36 f_{ck} b x_u$~~

$$= \frac{0.0035(162.975 - 40)}{162.975}$$

$$\varepsilon_{sc} = 2.6409 \times 10^{-3}.$$

$$f_{sc} = \varepsilon_{sc} \times E_s.$$

$$= 2.6409 \times 10^{-3} \times 2 \times 10^5.$$

$$\boxed{f_{sc} = 528.19 \text{ N/mm}^2}$$

$$M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) + f_{sc} A_{sc} (d - d')$$

$$= 0.36 \times 20 \times 200 \times 162.975 [410 - 0.42(162.975)]$$

$$+ 528.19 \times \frac{3 \times \pi \times 12^2}{4} \times (410 - 40).$$

$$\boxed{M_u = 146.46 \text{ kN.m}}$$

3. Determine Mulim of R.C.C of size 250 × 500 mm overall is reinforced with 3 # 12 mm in compression and 4 # 20mm in Tension. $d' = 40mm$. Use M_{20} and Fe415.

$\rightarrow 0.48d$

$x_u = 0.75\ x_{umax}$

$= 0.75 \times 0.048 \times 460 \rightarrow d$

$x_u = 165.6$

$\varepsilon_{sc} = \dfrac{\varepsilon_{cbc}(x_u - d')}{x_u}$

$= \dfrac{0.0035(165.6 - 40)}{165.6}$

$= 0.00265$

$f_{sc} = 0.967\ f_{yd}$

$= 0.967 \times 0.87 \times 415$

$f_{sc} = 349.187\ N/mm^2 < 361.05$

$M_{ulim} = 0.36 f_{ck}\ b x_u\ (d - 0.42 x_u)$

$+ f_{sc} A_{sc}\ (d - d')$

$= 0.36 \times 20 \times 250 \times 165.6\ (460 - 0.42(165.6))$

$+ 349.187 \times \frac{\pi}{4} \times 3 \times 20^2 \times (460 - 40)$

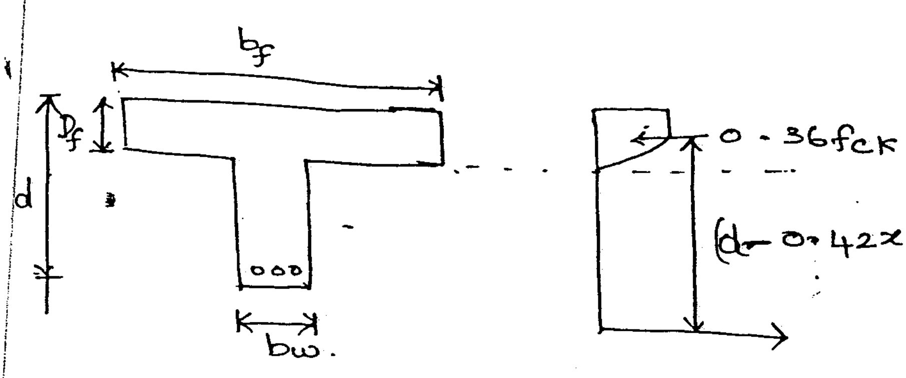
12^2

$M_{ulim} = 254.6\ kN.m$

26/10/2015.

R.C.C.

T-BEAM (L.S.M)



Case (i)

(i) N.A Lies with flange (Design as Rectangular Beam):

$$F_c = F_t$$

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$\boxed{x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}}$$

M.R of Beam:

$$M.R = 0.36 f_{ck}\, b\, x_u (d - 0.42 x_u)$$

$$M.R = 0.87 f_y A_{st} (d - 0.42 x_u)$$

case (ii): N.A lies outside the flange.

If $\frac{D_f}{d} \leq 0.2$.

$$F_c = F_{c1} + F_{c2}$$

$$\boxed{F_c = 0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) D_f}$$

$$\boxed{M.R = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)}$$

If $\frac{D_f}{d} > 0.2$. Put $D_f = y_f$.

$$\boxed{y_f = 0.15 x_u + 0.65 D_f.}$$

$$\boxed{M.R = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right).}$$

1. A large hall of inner dimension 6m × 30m is provided with no. of R.C.C. beams monolithical constructed with slab along shorter span ~~thickness~~ of brick wall 30cm thickness of slab = 150mm $b_w = 250$mm. overall depth is 550mm. 4 # 20mm bars in tension zone. Determine.

(i) Effective width of flange.

(ii) Depth of A.N.A

(iii) Type of section.

(iv) Ultimate Moment of resistance

(v) Safe Super imposed load.

Use M_{20} and Fe250. spacing of beam is 3m c/c.

Solution: $d = D - 40 = 550 - 40 = 510$.

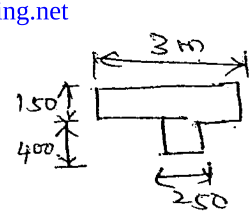
(i) L_{eff}

$$L_{eff} = 6m + \frac{0.3}{2} + \frac{0.3}{2} = 6.3m.$$

$$L_{eff} = 6m + 0.51 = 6.51m.$$

$$\boxed{L_{eff} = 6.3m}$$

(ii) b_f.



$$b_f = \frac{l_o}{6} + b_w + 6D_f$$

$$= \left(\frac{6300}{6}\right) + 250 + 6(150)$$

$$\boxed{b_f = 2200 \text{ mm}} < 3000 \text{ mm}$$

(iii) x_u:

$$F_c = F_t$$

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$x_u = \frac{\left(0.87 \times 250 \times 4 \times \frac{\pi \times 20^2}{4}\right)}{0.36 \times 20 \times 2200}$$

$$= 17.25 \text{ mm} < 150 \text{ mm}$$

$0.53d$

(iv) M.R

$$M.R = 0.36 f_{ck} b_f x_{u_{max}} (d - 0.42 x_{u_{max}})$$

$$= 0.36 \times 20 \times 2200 \times 0.53 \times 510$$

$$(510 - (0.42 \times 0.53 \times 510))$$

$$M_{ulim} = 1.697 \times 10^3 \text{ kN.m}$$

(v)
$$M_u = \frac{wl^2}{8}$$

$$1.697 \times 10^3 = \frac{w \times (6.3)^2}{8}$$

$$w = 342.15 \times 10^3 \text{ kN}$$

$$w = D.L + I.L$$

$$342.15 \times 10^3 = (0.15 \times 3 \times$$

(iv)

$M_R = 0.36 f_{ck} b_f x_u . (d - 0.42 x_u)$

$= 0.36 \times 20 \times 2200 \times 17.25 \left(d - (0.42 \times 17.25)\right)$

$M_R = 137.37$ kN.m

(v)

$$M_R = \frac{w_u l_{eff}^2}{8}$$

$$w_u = \frac{137.37 \times 8}{6.3^2}$$

$w_u = 27.68$ kN. $w = \frac{27.68}{1.5} = 18.46$ kN

w_a = D.L + L.L

$18.46 = \left[\left((0.15 \times 3) + (0.4 \times 0.23)\right) \times 25\right]$ + L.L.

$18.46 = 13.75$ + L.L

$\boxed{L.L = 4.71 \text{ kN/m.}}$

$$L.L = \frac{4.71}{3 \times 1}$$

$\boxed{L.L = 1.57 \text{ kN/m}^2.}$

2. Determine ultimate M.R of T-beam.

$b_f = 750mm$. $D_f = 120mm$ $D = 550mm$

$d' = 50mm$ $A_{st} = 5 \# 25mm\phi$.

$b_w = 300mm$. Use M_{20} Fe 415.

Solution:

Assume $x_u < D_f$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

$$= \frac{0.87 \times 415 \times 5 \times \frac{\pi \times 25^2}{4}}{0.36 \times 20 \times 750}$$

$$= 164.1 mm > 120 mm$$

$$\frac{D_f}{d} = \frac{120}{500} = 0.24 > 0.2$$

$D_f = y_f$. $y_f = 0.15 x_u + 0.65 D_f$.

$$0.36 f_{ck} . b_w x_u + 0.45 f_{ck} . (b_f - b_w) D_f = 0.87 f_y A_{st}.$$

$$0.36 \times 20 \times 300 \times x_u + 0.45 \times 20 \times (750 - 300) \times (0.15 x_u + 0.65 \times 120) = 0.87 \times 415 \times 5 \times \frac{\pi \times 25^2}{4}$$

$$2160 x_u + 4050 (0.15 x_u + 78)$$

$$2767.5 x_u + 315900 = 886.15 \times 10^3.$$

$$\boxed{x_u = 206.052 mm}$$

$$y_f = 0.15 (177.52) + 0.65 (120)$$

$$= 104.63 \; mm < D_f$$

Terrogations 2016
altitude

$$M.R = 0.36 f_{ck} b_w x_u (d - 0.42 x_u)$$

$$+ 0.45 f_{ck} (b_f - b_w) \times y_f \left(d - \frac{y_f}{2}\right)$$

$$= 0.36 \times 20 \times 300 \times 206.052 \left[500 - 0.42(206.052)\right]$$

$$+ 0.45 \times 20 (750 - 300) \times 104.63$$

$$\times (500 - 104.63)$$

$$\boxed{M.R = 351.46 \text{ kN.m.}}$$

STEEL STRUCTURES

→ Analysis and design of tension and compression members, beam-column and column bases.

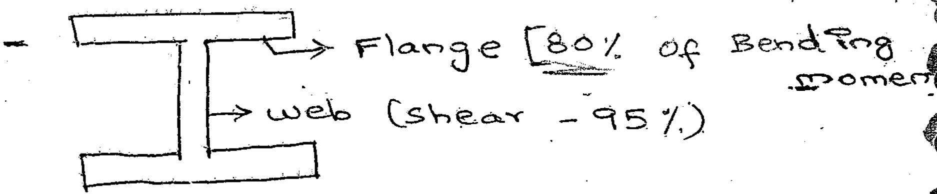
→ Connections:

* Simple and eccentric
* Beam - column connections
* plate girders and trusses.

→ Plastic analysis of beams and frames.

STEEL STRUCTURES

* $\gamma_{con} = 2500\ kg/m^3$ (R.C.C)
* $\gamma_{steel} = 7850\ kg/m^3$
* Structural steel $\mu = 0.3$.
* Rigidity modulus of steel $= 0.769 \times 10^5$ MPa.
* Specific gravity of steel $= 7.85$.



* Drainage cover plate → checkers plate
* Steel structure : Made up of structural steel

Section Available:

* Flate plate.
* Angle section
* channel section.
* I - section
* T - section.

* All structural steel consists of Mild steel -250

  Truss - All Joints are pinned.

* I - Section is very strong section.

  There is no torsion moment produced in I - Section.

STEEL STRUCTURES:

* Steel structures is made up of structural steel and structural steel are made up of hot rolled section.
* Structural steel is of mild steel and of grade Fe 250.
* Mild steel is used because it is highly ductile and hence can be moulded into shapes.

Advantages of Steel structures:

* Strength of depth ratio is high.
* Prefabricated
* ~~Reusable~~ Reusable
* High scrap value.
* Can be dismanteled easily.

Disadvantage of steel structures:

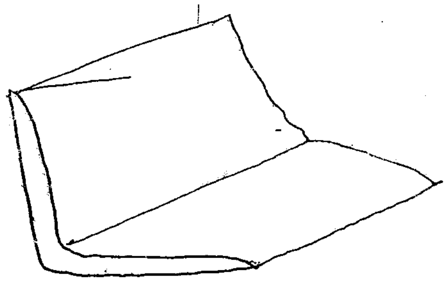
* Corrossion is high.
* Vibration and noise.
* Fire resistance is low.
* Fatigue failure.

Type Of Steel Section:

* Flat Plate:

* Designated by thickness x breadth.
* Used for walkways, plate girder.
* Thickness shall not be less than 8 mm when maintenance is not possible.
* Thickness shall not be less than 6mm when maintenance is possible.
* Thickness shall not be less than 4mm in case of secondary member.

* Angle Section:



* It is designated by length x breadth x thickness.
* Ex : ISA 90 x90x6 mm - Equal angle.

  ISA - Indian standard angle.
* 100 x 90x6 where, 100 → longer leg
  90 - shorter leng
  6 - thickness.

(generally longer legs are connected legs)

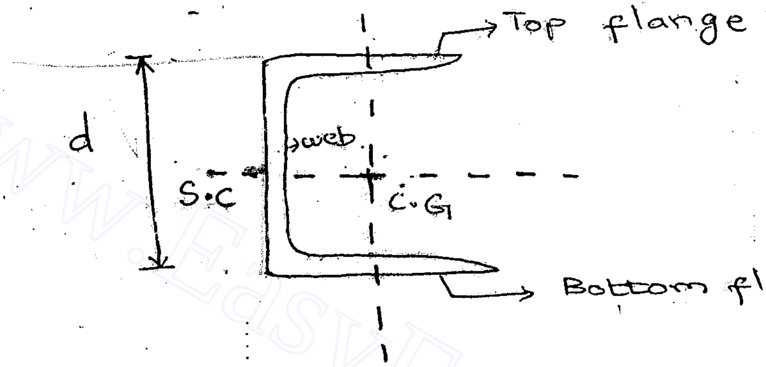
* Maximum available angle section is

  ISA - 200 x 200 x25

* Uses:
  - Purlins, Bottom chord, lacings.

channel section

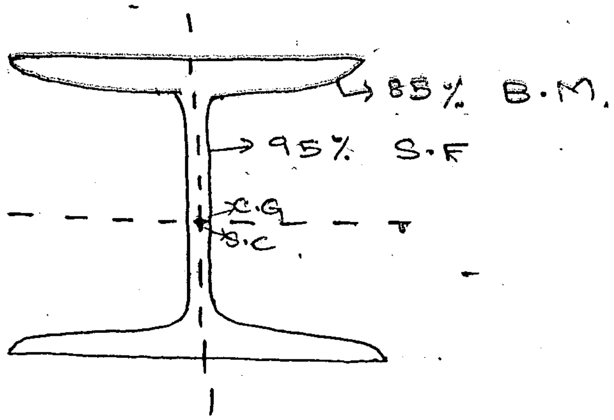
* Designated by ISJC, ISLC, ISMC, ISGC, MCP (Medium channel with parallel flange).
* Shear centre lies outside the section
* It is subjected to the torsional moment when it is loaded.
* Used as purlins, secondary beams (or) where light loads occurs and as bottom chord.
* Maximum available section is ISMC-400



I - section:

* Designated by ISJB, ISLB, ISMB, ISWB, and ISHB, SC

ISJB - Indian standard Junior Beam
ISLB - Indian standard light Beam.
ISMB - Indian standard Medium Beam.
ISWB - Indian standard wide flange Beam
ISHB - Indian standard Heavy Beam.
SC - Indian column section.



* Used as beams and column for heavy loa

maximum available I - section is ISMB-600

→ when $\frac{d}{t} < 85$ (No stiffner is used)

→ $85 < \frac{d}{t} < 200$ (use stiffners)

→ $\frac{d}{t} = [85 - 200]$ (vertical stiffner)

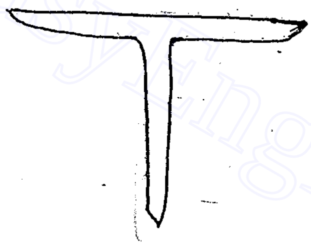
→ $\frac{d}{t} > 200$ (Horizontal stiffner along with vertical stiffner)

* Flanges are responsible for 85% of BM

* web is responsible for 95% of Shear force

*Uses: for beams and columns.

T- section:



*Designated by

ISNT → Indian standard Normal Tee.

ISHT - Indian standard heavy weight Tee

ISMT - Indian standard Medium weight Tee

ISLT - Indian standard light weight Tee

ISDT - Indian Standard Deep legged Tee

* Designated by DxbxT.

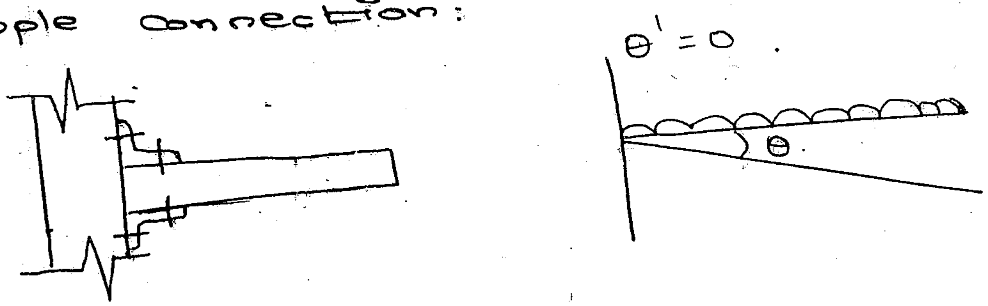
* Used in water tank and in bracket connections.

Connections:

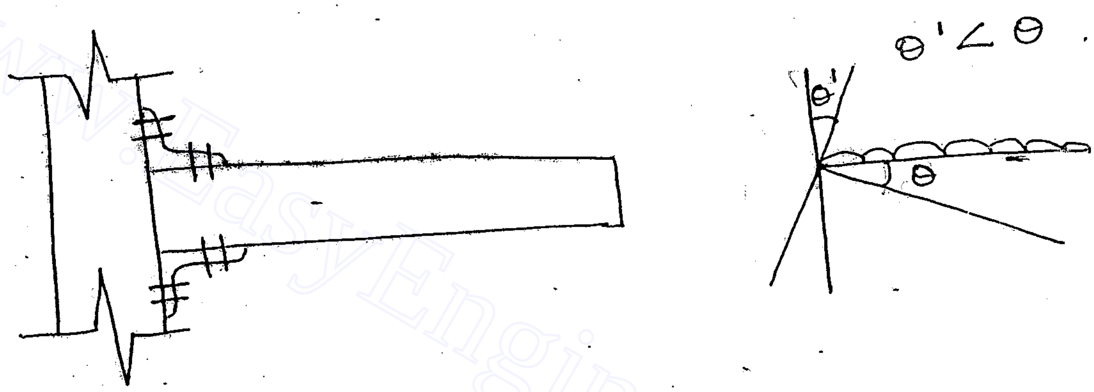
There are 3 types of connection.

* Simple connection.
* Semi-rigid connection.
* Rigid connection.

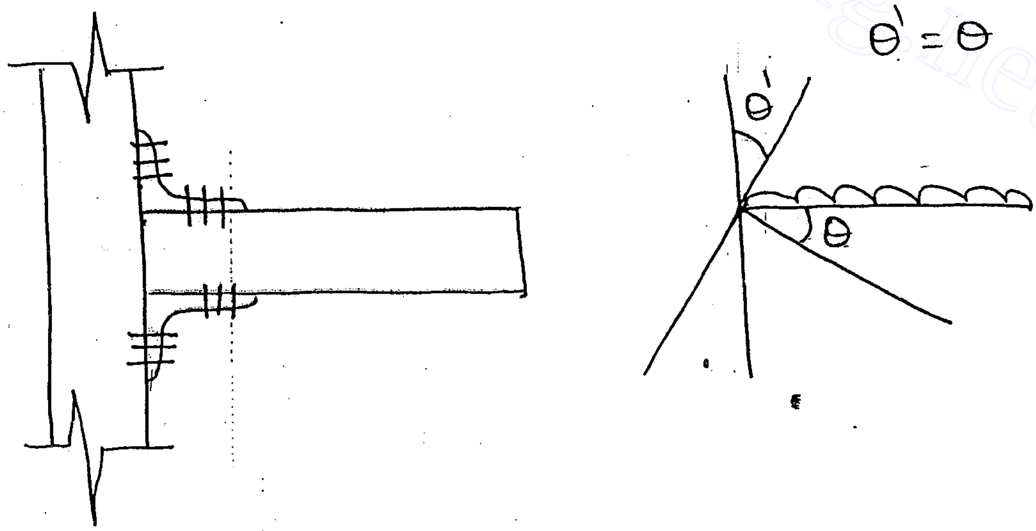
Simple connection:



Semi-rigid connection:



Rigid connection:



10/7/2015

STEEL

Connection:

* It is an important part in steel structure beacuse unlike concrete structure it is not casted monolithically.

* Initially connection were made using rivets but now they are obsolete.

Riveted Connections:

* Riveted connection has a disadvantage of making erection stresses. and high ~~temperature~~ is required for erection (Hot driven Rivet)

* Types
  - Hot driven Rivet
  - cold driven rivet

* In hot driven rivet steel is heated upto melting point and then rivet is inserted in the joint.

* cold driven rivet involves pressure injection or pressure erection.

* Gross area of plate ~~is~~ get affected in riveted connection.
Hence net area affects the load carrying capacity of plate

Bolted connection:

Types.
* BB
* HSFG (High strength friction grip)

	Grade	
* BB	4.6	4 - fub
HSFG	8.8	

* 4 × $\frac{1}{100th}$ of ultimate stress.

i.e $f_{ub} = 4 \times 100 = \underline{400 \text{ MPa}}$ fub

0.6 - the yield stress.

i.e $f_{yb} = 0.6 \times 400 = \underline{240 \text{ MPa}}$ fy

* **HSFG Bolt**: (8.8) → Grade

$f_{ub} = 8 \times 100 = \underline{800 \text{ MPa}}$ - fub

$f_{yb} = 0.8 \times 800 = \underline{640 \text{ MPa}}$ fyb

$$\text{Mpa} = N/mm^2 \Rightarrow \frac{N}{mm^2} \times 1000 \Rightarrow \frac{N}{m^2}$$

Advantages of Bolted connection:

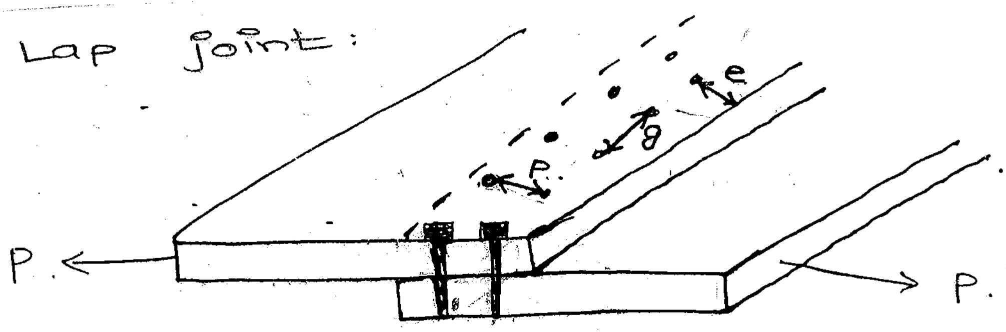
* Easy erection
* It can be dismantled easily.
* It can be resused.
* No skilled Labour required.
*

Disadvantage:

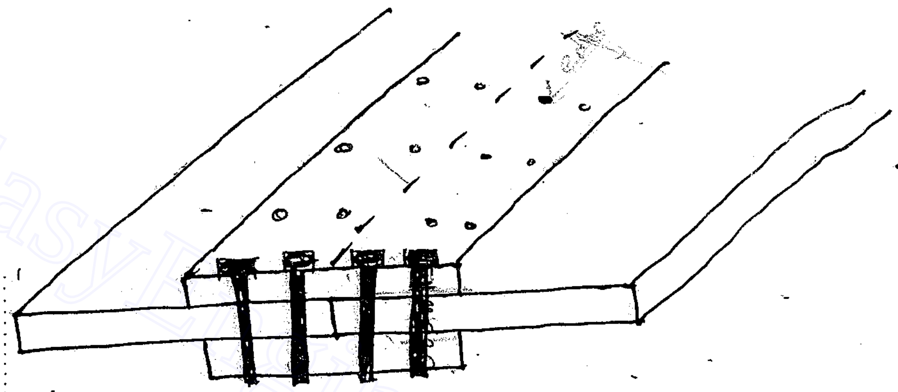
* Bolt connection get loosen due to vibration.
* Gross area of plate is affected
* Extra weight is added to the structure

Type of joints:

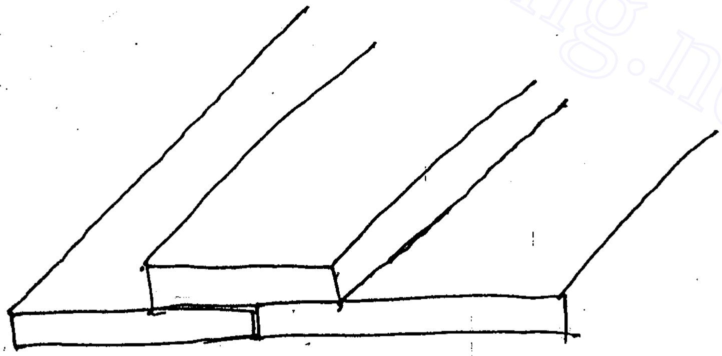
* Lap joint:



* ~~Single~~ Double. cover butt joint.



* Single cover butt joint.



Types of failure of bolt.

* Shearing failure bolt
  - → single shear
  - → Double shear.
* Bearing failure of bolt
  - →

18/8/15

6

D.S.S.

Pitch Distance:

It is the c/c distance b/w 2 consecutive ~~me~~ bolt measured along parallel to direction of forces. (or) stresses in a member.

- In a wide plate pitch may also be defined as c/c. ~~distance b/w~~ of bolt. ~~along~~ measured along length of member. (or) connection.
- When bolt's are placed in a staggered ~~member~~ manner. the pitch calculate is called staggered pitch.

Minimum pitch : $\underline{\underline{2.5\,d}}$

where d - nominal dia of bolt.

Max pitch → 12t (or) 200 mm for Comp

" → 16t (or) 200. for tension

→ 32t (or) 300. for tacking or stitch (not exposed to weather)

→ 16t (or) 200 for tacking or stitch (exposed to weather)

In case of two flat plates, angles, channels or Tee

for Comp member Max distance ≯ 600 mm

for tension member Max distance ≯ 1000 mm

19/8/2015

D.S.S

* Maximum pitch → 12t (or) 200 mm whichever is less for compression member.

* Maximum pitch → 16t (or) 200 mm whichever is less for tension member.

* 32t (or) 300mm whichever is less for tacking (or) stiching of plates (when the plates are not exposed to weather).

* 16 t (or) 200mm whichever is less for tacking (or) stiching of plates (when the plates are exposed to weather).

* The distance b/w the centre of any two consecutive bolts should not exceed 32t (or) 300 mm whichever less.

* In case of two flat plates, angles, channels (or) Tee-sections.

Maximum pitch of tacking bolts.

(In which tack (or) stick bolts are to be provided along the length to connect each of them)

→ Not exceeding 600mm for compressi member.

→ Not exceeding 1000 mm for tension member.

Gauge Distance:

* It is the distance b/w adjacent to the bolt line (or) the c/c distance b/w two consecutive bolts. measured along the width of the member (or) the connection.

Maximum gauge should not be more than (100 + 4t (or) 200mm) whichever is less.

End Distance:

It is the distance from centre of bolt hole to the nearest edge of member (or). cover plate in the direction of stress (or) force.

Edge Distance:

It is the distance from centre of bolt hole to the nearest edge of member (or) cover plate at right angle to the direction of stress.

*Minimum edge distance = $1.7\,d_H$

d_H = dia of bolt hole for sheared (or) hand flame cut edges.

* Minimum edge distance = $1.5\,d_H$.

In case of rolled, machine flame cut edge

Maximum Edge distance:

* Maximum edge distance to nearest edge of bolt hole to an edge unstiffened part should not exceed. $12t\varepsilon$

where, $\varepsilon = \sqrt{\frac{250}{f_y}}$

t - thickness of thinner outside plate

$\boxed{\text{*. } 40\text{mm} + 4t}$

where,

t = thickness of thinner outside plate (for corrossive environment).

ϕ of bolt holes (d_o):

d_o = Nominal dia of bolt + 1mm

for bolt ϕ (12mm & 14mm)

$d_o = \phi$ of bolt + 2mm.

for bolt (ϕ < 24mm) | => (16 - 24mm)

$d_o = \phi$ of bolt + 3mm

for bolt (ϕ > 24mm).

Failure Of Bolted Connection:

The failure of connections with bearing bolts in shear involves either bolt failure (or) the failure of the connected plates.

Types:

* Shearing failure of bolt.
* Bearing failure of bolt.
* Tension failure of bolt.
* Bearing failure of plate.
* Tearing failure of plate.
* Block shear failure.

Design Strength Of Bearing Type Bolts Against Factor Shear Force:

(a) Design Shear Strength Of Bolts (V_{dsb}).

Bolt subjected to factor shear force

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \qquad \boxed{\gamma_{mb} = 1.25}$$

where,

V_{nsb} → Nominal shear strength of bolt.

γ_{mb} → Partial factor of safety of material for bolts.

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

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

where,

f_{ub} - ultimate stensile strength of bolt

n_n - no. of. shear plane with threads intercepting the shear plane.

n_s → no. of. shear plane without thread intercepting the shear plane.

$A_{sb} \rightarrow \frac{\pi}{4} \times d^2$

$A_{nb} \rightarrow \frac{\pi}{4} \times d^2 \times 0.78$ (or) $0.78 \times A_{sb}$

$A_{nb} \rightarrow$ Net tensile area @ thread

$A_{sb} \rightarrow$ nominal plain shank area of the bol

γ_{m0} - Yield stress

γ_{m1} - ultimate stress.

21/8/2015

DESIGN OF STEEL STRUCTURE

$$\boxed{V_{dsb} = \frac{(f_{ub}/\sqrt{3})}{\gamma_{mb}} \left[n_n A_{nb} + n_s A_{sb} \right] \times \beta_{lj} \times \beta_{lg} \times \beta_{pkg}}$$

β_{lj} - Reduction factor for long joints.

β * In long joints distance b/w first and last joint exceeding. 15d in the direction of load, the nominal shear capacity of bolt can be reduced by factor $\boxed{\beta_{lj}}$

$$\beta_{lj} = 1.075 - \left(\frac{l_j}{200d}\right).$$

where l_j - Length of joint

$(0.75 \leq \beta_{lj} \leq 1.0)$

β_{lg} - Reduction factor for long grip length

* when grip length of bolt increases (if it exceeds. 5d (5 times nominal dia)) the bolt subjected ~~to~~ greater B.M due to shear force acting on its shank

$$\boxed{\beta_{lg} = \frac{8d}{3d + l_g}}$$

l_g - grip length shall not be greater than 8d.

β_{pkg} - Reduction factor for packing plate

When packing plate is more than 6mm thick the shank of bolt is subjected to bending which affects the ~~nomin~~ nominal shear capacity of the bolt:

$$\beta_{pkg} = 1.0 - 0.0125 t_{pkg}$$

t_{pkg} - thickness of thicker packing plate.

DESIGN BEARING STRENGTH OF BOLT AND PLATE: (V_{dpb})

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

where $\boxed{V_{npb} = 2.5 k_b \times d \times t \times f_u}$

$$\boxed{k_b = \frac{e}{3d_0} \text{ (or) } \left(\frac{p}{3d_0}\right) - 0.25 \text{ (or) } \frac{f_{ub}}{f_u} \text{ (or) } 1 \quad \text{whichever is less.}}$$

V_{npb} - Nominal bearing strength of bolt.

* e, and p are the edge and pitch distance of the fastner along the direction length of bearing.

f_{ub} → ultimate tensile stress of the bolt

f_u → smaller of the ultimate tensile stress of bolt (or) plate d - dia of bolt in mm.

t = ~~Σ~~ thickness of connected plates experience bearing stress in the same direction.

DESIGN TENSILE STRENGTH OF BOLT : (T_{db})

The nominal tensile strength of bolt in tension is given by.

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

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}.$$

where

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

DESIGN BOLT STRENGTH (OR) DESIGN BOLT VALU (V_{db}).

* It is the least value of design strength of bolt in shear, bearing, tension (if exist)

FORMULA:

DESIGN TENSILE STRENGTH OF A PLATE

Tensile strength of plate is given by

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

Net Area for chain bolting

$$\boxed{A_n = (b - n d_o) \times t}$$

Net Area for staggered bolting.

$$\boxed{A_n = \left(b - n d_o + \frac{s_1^2}{4g} + \frac{s_2^2}{4g} + \cdots\right) \times t}$$

DESIGN STRENGTH OF CONNECTION: (V_{dc}):

The strength of a bolted connection is the minimum design strength of bolt in shear, bearing and tension (if exist) and minimum design strength of connected member against gross cross section yielding (or) net section rupture.

Efficiency Of The Joint (or) Percentage strength Of Joint: (η)

Efficiency of bolted joint also called Percentage strength of joint is the ratio of design strength of joint to the design strength of main member, expressed as percentage.

$$\eta = \frac{\text{Design strength of joint}}{\text{Design strength of main member}} \times 100$$

1/9/2015

D.S.S.

1.) calculate the strength of 16mm dia bolt of grade 4.6 for a lap joint. The main plates to be joint are 10mm thick of Fe 410 grade. Assume pitch and edge distance of a bolt & 40mm and 30mm. and thread of the bolt is intercepting the shear plane

Solution:

Give Data:

$f_{ub} = 400 N/mm^2$ $f_{yb} = 240 N/mm^2$

$\phi = 16mm$ $f_{up} = 410$. $t = 10mm$.

$d_0 = 16+2 = 18mm$

$P = 40mm$ $e = 30mm$. Lap joint.

$d_0 = 16+2 = 18mm$.

Shear strength:

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times 1.25} \times (n_n \times 0.78 \times A_{sb})$$

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

$V_{dsb} = 28.97$ KN.

Bearing strength:

$$V_{dpb} = \frac{2.5 \times k_b \times d \times t \, f_u}{\gamma_{mb}}.$$

$$k_b = \frac{30}{3 \times 18}, \left(\frac{40}{3 \times 18} - 0.25\right), \frac{400}{410}, 1 \quad \frac{f_{ub}}{f_{up}}$$

$= 0.555, \boxed{0.49}, 0.9756, 1$

$k_b = 0.49$.

$= 2.5 \times 0.49 \times 16 \times 10 \times 400 / 1.25$

$\boxed{V_{dpb} = \frac{78.4}{1.25} \text{ KN}}$ $= 62.72$ KN

Strength of bolt is $\underline{28.97\ kN}$.

$$\text{No. of bolt} = \frac{\text{Total load}}{\text{one bolt value}}$$

~~Strength of Bolt~~

2. 2 plates each $300mm \times 16mm$ are to be joined using $20mm\ \phi$ bolts, B.B to form a lap connection. the connection is supposed to transfer a service load of 375 kN. calculate no. of bolt required for connection with minimum pitch and edge dist. Assume the thread of bolt does not intercept the shear plane.

Given:

$t = 16mm$ $\quad d = 20mm$ $\quad d_o = 20+2 = 22mm$

$f_{ub} = 400\ N/mm^2$ $\quad f_{yb} = 240\ N/mm^2$.

$P_u = 1.5 \times 375 = 562.5\ kN$. $\quad P = 2.5 \times d = 50mm$

$e = 1.5 \times d_o = 33mm$.

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

$$V_{dsb} = 58.04\ kN.$$

$k_b = 0.5, 0.507, 0.976,$

$$V_{dpb} = \frac{2.5\ k_b \times d \times t\ f_u}{1.25}$$

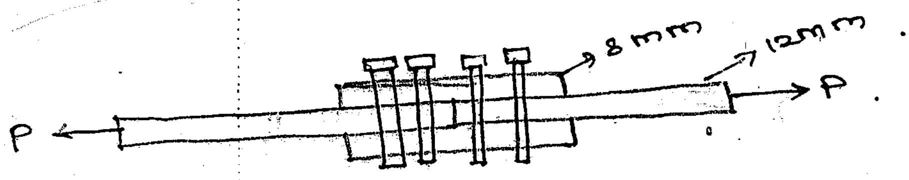
$$= \frac{2.5 \times 0.5}{1.25} \times 20 \times 16 \times 400.$$

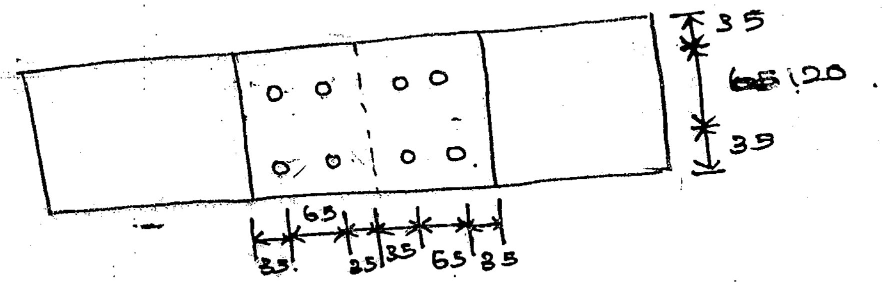
$$V_{dpb} = 128\ kN.$$

Strength of bolt $= \underline{58.04\ kN}$.

$$\text{No. of bolt} = \frac{562.5}{58.04} = \underline{10\ \text{bolts}}.$$

3. Compute the design strength and η of a bearing type of connection as shown in figure. The bolts are 4.6 grade and 20mm in dia. Plates are of grade Fe 410.





Solution:

Strength of bolt.

$$V_{dsb} = \frac{f_u}{\sqrt{3}\times 1.25}(n_n A_{nb}) \quad \text{one bolt}$$

total.

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

$$= 362.17 \text{ kN}.$$

$$V_{dpb} = \frac{2.5\, k_b\, d\times t\times f_u}{\gamma_{m1}} \qquad k_b = 0.53,\ 0.73,\ 0.97,$$

$$= \left(\frac{2.5\times 0.53\times 12\times 20\times 400}{1.25}\right)\times 4$$

$$= 407.04 \text{ kN}.$$

$$T_{dp} = \frac{0.9 f_{ub}\times A_{nb}}{\gamma_{m1}}$$

$$= \frac{0.9\times 410\times\left(190 - 2(22)\right)\times 12}{1.25} = 517.19 \text{ kN}.$$

STRUCTURAL ANALYSIS

- → Analysis of statically determinate trusses, arches, beams, cables and frames
- → Displacement in statically determinate structures.
- → Analysis of statically indeterminate structures by force/energy method
- → Analysis by displacement method (slope deflection and moment distribution method)
- → Influence lines for determinate and indeterminate structures.
- → Basic concept of matrix method of structural analysis.

STEEL STRUCTURES

→ Analysis and design of tension and Compression members, beam-column and column bases.

→ Connections:

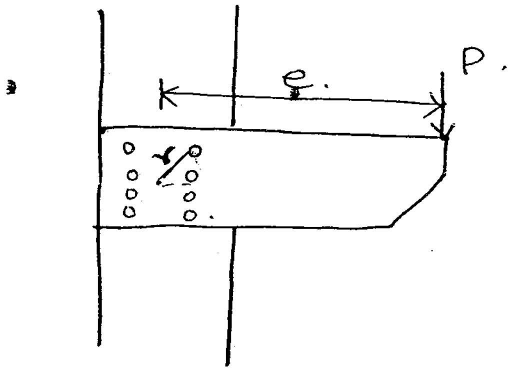
* Simple and eccentric
* Beam - column connections
* plate girders and trusses.

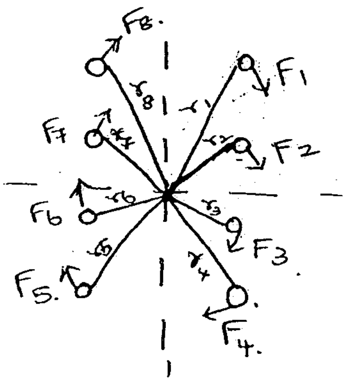
→ Plastic analysis of beams and frames.

1/10/2015

DESIGN OF STEEL STRUCTURE.

INPLANE MOMENT:





1.) Direct shear:

$\frac{P}{n}$ ↓   ↓ P/n

$\frac{P}{n}$ ↓   ↓ P/n

P/n ↓   ↓ P/n

↓ P/n.   ↓ P/n.

$$\boxed{\text{Direct stress} = \frac{\text{Load}}{\text{No. of bolts.}}}$$

2.) Moment.

$F \propto r.$

$F_1 \propto r_1.$

$\vdots \qquad \vdots$

$F_1 = K r_1.$

$F_2 = K r_2.$

$\vdots \qquad \vdots$

$F_1 r_1 = K r_1 r_1.$ (Multiplying "r" on both side)

$\vdots \qquad \vdots$

$\Sigma F \times r = K \Sigma r_1^2.$

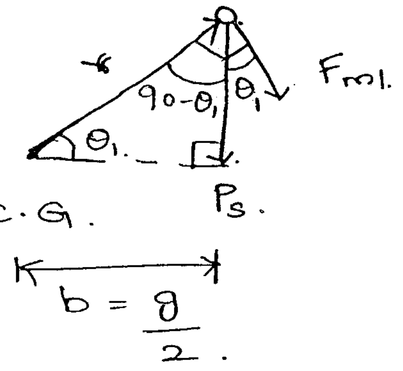
$\Sigma F_x r = K \Sigma r_1^2$

$M = K \Sigma r_1^2.$

$$\boxed{K = \frac{M.}{\Sigma r^2} = \frac{Pe}{\Sigma r^2}.}$$

$F_1 = K r_1.$

$$\boxed{F_1 = \frac{Pe}{\Sigma r^2} \times r_1.}$$



$$\boxed{R = \sqrt{P_s^2 + F_{m_1}^2 + 2P_s F_m \cos\theta_1}}$$

$$\boxed{\cos\theta_1 = \frac{b}{r_1}}$$

L.S.M

$$R \leq V_{db}$$

working stress method.

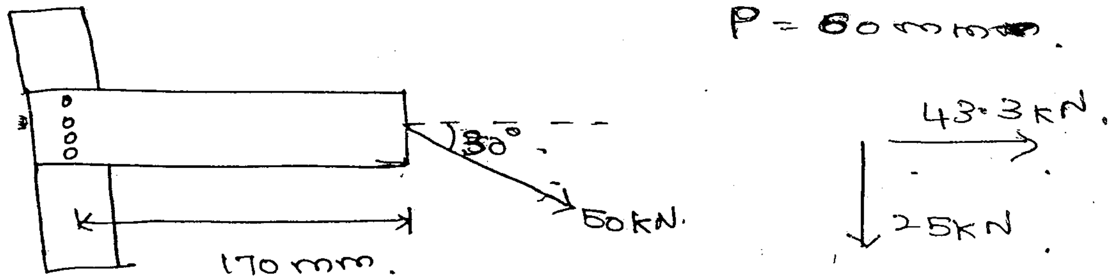
$$\tau_{vf} > \tau_{vf\ cal}$$

$$\boxed{\tau_{vf} > \frac{R_{max}}{\left(\frac{\pi}{4} \times d^2\right) \times 0.78}}$$

Max shear stress

$r \rightarrow$ distance from C.G to each bolt

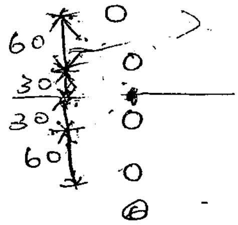
1.) Determine the max. force developed in a critical bolt and find out Max. shear stress.



Direct stress $P_{s_1} = \frac{25\times10^3 N}{4} = 6.25$ kN.

$P_{s_2} = \frac{43.3\times10^3 N}{4} = 10.83$ kN.

Torsional shear:

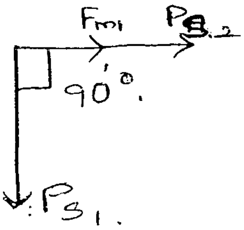


$$F_1 = \frac{P\times e_1}{\Sigma r^2} \times r_1$$

$$= \frac{25\times10^3\times170}{(90^2+30^2+30^2+90^2)} \times 90$$

$$\boxed{F_1 = 21.25\times10^3 \text{ kN}}$$

$$R = \sqrt{(P_{s_1})^2 + (F_{m_1}+P_{s_2})^2 + 2F_{m_1}\times P_{s_2} P_{s_1}\times \cos\theta_1}$$



$\cos\theta_1 = 90°$.

$\cos\theta_1 = \cos 90° = 0$.

$$= \sqrt{P_{s_1}^2 + (F_{m_1}+P_{s_2})^2}$$

$$= \sqrt{(\underset{P_{s_1}}{6.25})^2 + (\underset{P_{s_2}}{10.83} + \underset{F_{m_1}}{21.25})^2}$$

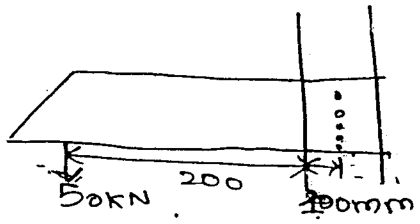
$R = 32.68$ kN.

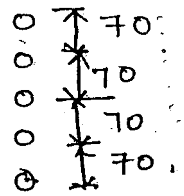
F.S.M

$$\text{Max. shear stress} = \frac{32.68 \times 10^3}{\left(\frac{\pi \times 20^2}{4}\right) \times 0.78}$$

Max. shear stress $= 133.37$ N/mm².

2.)





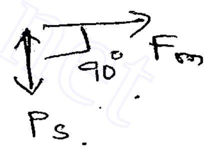
$$\text{Direct shear} = \frac{50 \times 10^3}{5} = 10 \text{ kN}.$$

$$\text{Torsional shear} = F_{m1} = \frac{P \times e}{\sum r^2} \times r_1.$$

$$= \frac{50 \times 10^3 \times 200}{[2(70^2) + 2(140^2)]} \times 140.$$

$$(70^2 + 140^2 + 70^2 + 140^2)$$

$= 42.86$ kN.



$$R_{max} = \sqrt{F_m^2 + P_s^2}.$$

$$R_{max} = \sqrt{(42.86)^2 + (10)^2}.$$

$$\boxed{R_{max} = 44 \text{ kN}.}$$

(4)

$R_{max} \leq V_{db} \rightarrow 0.462 f_u$

V_{db} Least of V_{dsb}, V_{dpb}.

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

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

$$5.58 \times 10^3 = \frac{400}{\sqrt{3} \times 1.25} \times 0.78 \times \frac{\pi}{4} \times d^2$$

$$5.58 \times 10^3 = 113.18 \, d^2$$

$0.462 f_u$

$$d = \sqrt{\frac{5.58 \times 10^3}{113.18}}$$

$$= 7.02$$

$$d = 8 \, mm$$

$$V_{dpb} = \frac{\cancel{f_u} \, 2.5 \times K_b \times d \times t \cdot f_u}{\gamma_{mb}}$$

$$5.58 \times 10^3 = \frac{2.5 \times t \times d \times 10 \times 400}{1.25}$$

$$d = \frac{5.58 \times 10^3}{8000}$$

$$d = 0.697 \, mm$$

$\therefore$ Dia of bolt is $\underline{8 mm}$.

OUT PLANE MOMENT:

WELDING.

Types:
(i) Forge welding
(ii) Thermite welding.
(iii) oxy acetylene welding
(iv) Electric arc welding.

According to IS 816-1969 and IS 800-2007 the min. size of fillet ~~well~~ weld is dependent on thickness of thicker member:

(a) If the thickness of thicker member is upto 10mm Min size of weld 3mm

(b) If thickness is 10 - 20mm then size of fillet weld 5mm.

(c) If t is 20 - 32mm then size of weld is 6mm.

(d) If t > 32mm min size of weld is subjected to 8mm in first run ~~strut~~
= minimum 10mm.

Minimum size of triangle of fillet weld is called size of the fillet weld.

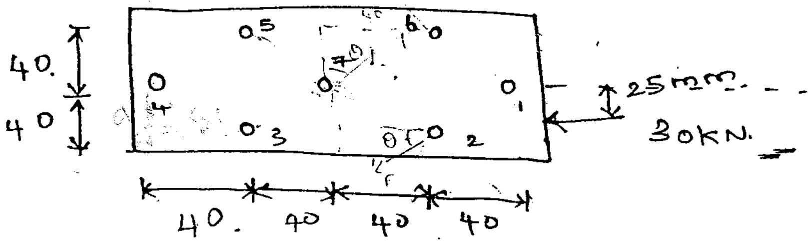
* value of throat thickness is the 1st distance from the corner of the weld to the hypotenuse of the triangle of the weld.

* The value of throat thickness is dependent on fusion angle.

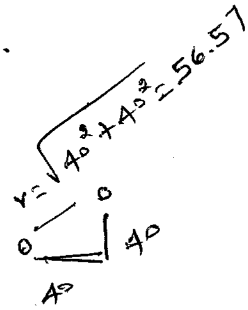
* case(a): In case of flat plate or member. $S_{max} = t_{plate} - 1.5$ mm.

* Case(b) In case of rounded corner (Angle section)
$S_{max} = \frac{3}{4} \times t_{round\ corner}$.

3.)



Determine suitable ϕ of bolt of grade 4.6 Thickness of plate = 10mm.

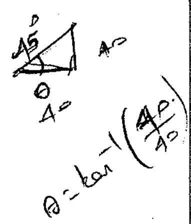


$$P_s = \frac{30\times10^3}{7} = \underline{4.29\ kN}$$

$$F_{m_1} = \frac{(30\times10^3\times25)}{\Sigma r^2}\times r$$

$$= \frac{30\times10^3\times25\times80}{(80^2+(56.57^2+56.57^2+80^2+56.57^2+56.57^2)}$$

$$\boxed{F_{m_1} = 2.34\ kN}$$



$$F_{m_2} = \frac{30\times10^3\times25\times56.57}{(80^2+56.57^2+56.57^2+80^2+56.57^2+56.57^2)}$$

$F_{m_2} = 1.666\ kN$ $\qquad \cos\theta_1 = \underline{0.707}$

$$R_1 = \sqrt{F_{m_1}^2+P_s^2}$$

$$= \sqrt{2.34^2+4.29^2}$$

$$\boxed{R_1 = 4.89\ kN}$$

$$\theta_1 = \tan^{-1}\left(\frac{40}{40}\right) = 45^\circ$$

$$R_2 = \sqrt{F_{m_2}^2+P_s^2+2F_{m_2}P_s\cos\theta_1}$$

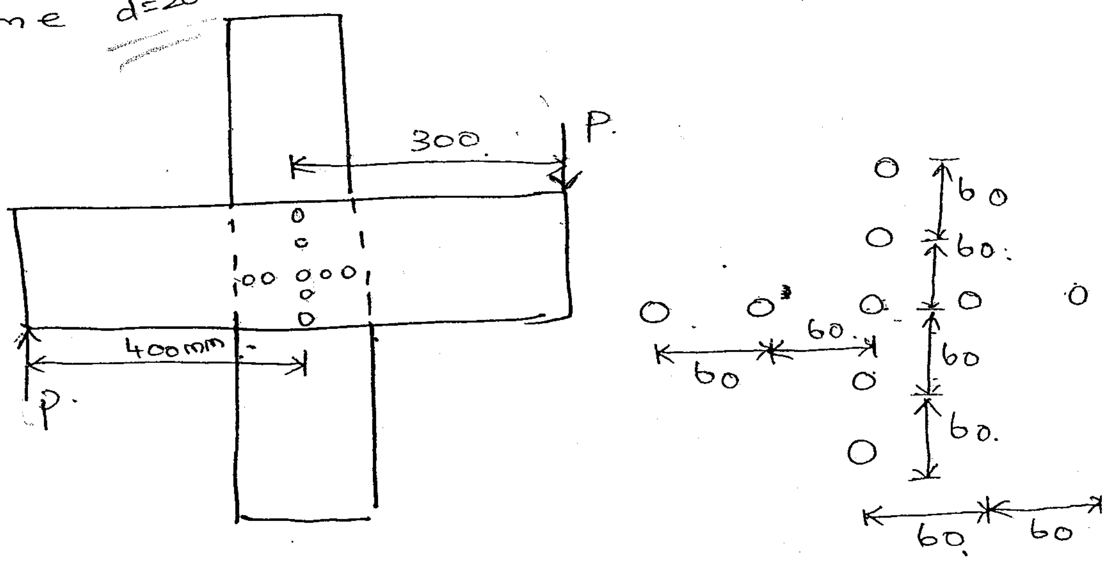
$$= \sqrt{1.66^2+(4.29)^2+2(1.66)(4.29)\cos45^\circ}$$

$$\boxed{R_2 = 5.59\ kN}$$

4.) Determine the load carrying Capacity of the system. (1)

$P = g = 60\,mm$

Assume $d = 20\,mm$.



Direct shear:

$$P_s = \frac{-P+P}{09} = 0.$$

$$\boxed{P_s = 0}$$

Torsional shear.

$$= \frac{\left[(P \times e_1) + (P \times e_2)\right] \times r_1}{\Sigma (r^2)}$$

$$= \frac{\left[(P \times 300) + (P \times 400)\right] \times 120}{(4(120^2) + 4(60^2))}$$

$$R_{max} = 1.17\,P.$$

$$R_{max} \leq V_{dp}$$

$$1.17P \leq 45.3 \times 10^3$$

$$P \leq \frac{45.3 \times 10^3}{1.17}$$

$$P \leq 38.72\,KN.$$

$$\boxed{P = 38\,KN.}$$

BOLT

INPLANE MOMENT:

Direct shear

$$P_s = \frac{P}{N}$$

Shear due to Torsion

$$F_{m_1} = \frac{Pe}{\sum r^2} \times r_{max}$$

Resultant $R = \sqrt{P_s^2 + F_m^2 + 2P_s F_m \cos\theta_{max}}$

$$\cos\theta_{max} = \frac{b}{r_{max}}$$

According to LSM

$$R \leq V_{db}$$

According to WSM

$$\tau_{vf} > \tau_{y}\ \text{Calculated}$$

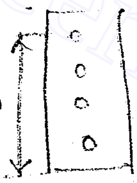
$$\tau_{vf} > \frac{R_{max}}{\left(\frac{\pi d^2}{4} \times 0.78\right)}$$

Unwin's formula

$d = 6\sqrt{t}$ mm

$$\eta = \frac{\text{Strength of bolted joint}}{\text{Strength of solid joint}}$$

Out Plane Moment



Case I $V_{sb} = \frac{P}{N}$ Direct shear

Case II Axial stress

$$T_b = \frac{P_1}{N} \quad \text{(or)} \quad T_b = \frac{M_1 \times y_1}{2\sum y^2}$$

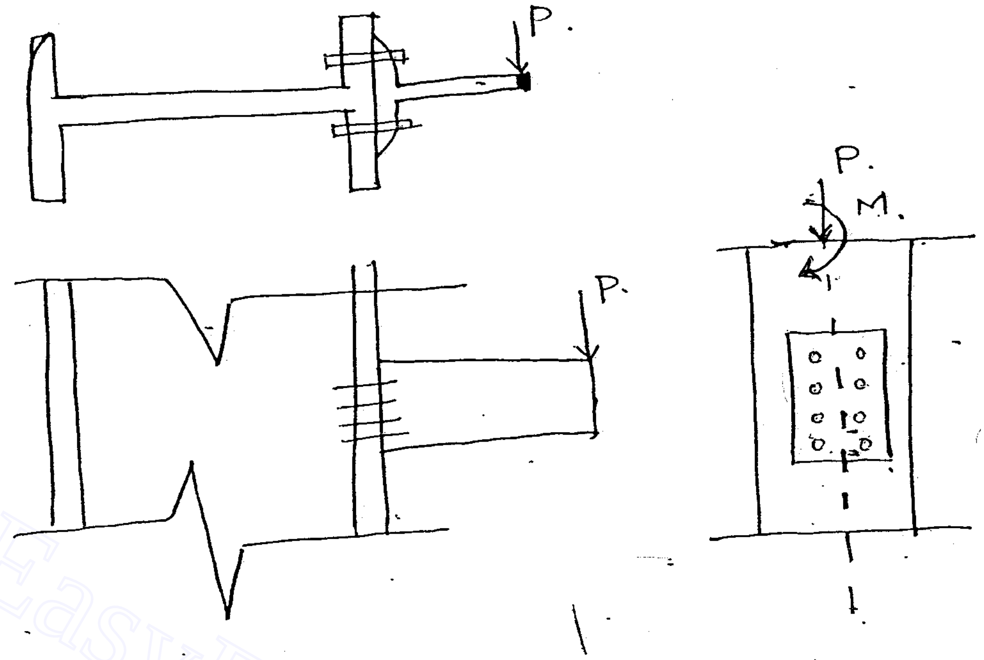
$$M_1 = \frac{P \times e \times y_1}{\ldots}$$

N.A $= \frac{h}{7}$

21/11/2015

OUT PLANE MOMENT

* Connection subjected to shear and tension



Case 1: Direct stress.

$$P_s = \frac{P}{N} = \frac{P}{n+n}$$

N – Total No. of bolts
n – No. of bolts in one row

Case 2: Axial stress.

$$T_b = \frac{P_t}{N}.$$

$$T_{db} = \frac{f_y A_g}{\gamma_{m0}} \text{ (or) } \frac{0.9 f_u A_n}{\gamma_{m1}}$$

Interaction Equation

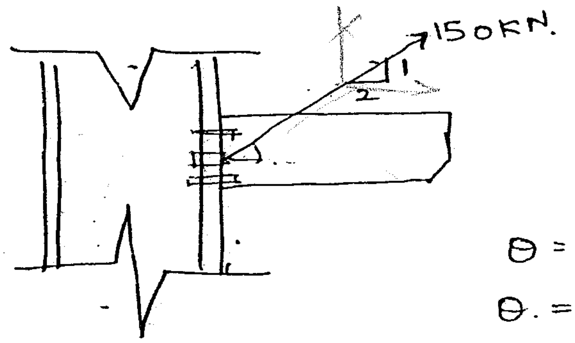
According to IS 800: 2007 pg: 76.

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left[\frac{T_b}{T_{db}}\right]^2 \leq 1.0.$$

According to IS 800 : 1984.

$$\left(\frac{\tau_{vf}\ cal}{\tau_{vf}}\right) + \left(\frac{\sigma_t\ cal}{\sigma_{tf}}\right) \leq 1.4.$$

1.) Determine interaction value of a given (2) bolt connection. 20mm ϕ bolt and grade 4.6 pitch 60mm.



$\theta = \tan^{-1}\left(\frac{1}{2}\right)$

$\theta = 26.56.$

Interaction equation.

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.$$

$$V_{sb} = \frac{150 \sin 26.56}{6} = 11.18 \text{ KN}.$$

$$T_b = \frac{150 \cos 26.56}{6} = 22.36 \text{ KN}.$$

V_{db} :

$$V_{dsb} = 0.462 f_{ub} \times \frac{\pi}{4} \times d^2 \times 0.78$$

$$= 0.462 \times 400 \times \frac{\pi}{4} \times 20^2 \times 0.78$$

$$V_{dsb} = 45.28 \text{ KN}.$$

T_{db}

$$T_{db} = \frac{0.9 f_u A_{nb}}{\gamma_{m1}} = \frac{0.9 \times 400 \times \frac{\pi}{4} \times 20^2 \times 0.78}{1.25}$$

$$= 70.57 \text{ KN}.$$

$$T_{db} = \frac{f_y A_g}{\gamma_{m0}} = \frac{240 \times \frac{\pi}{4} \times 20^2}{1.1}$$

$$= 68.54 \text{ kN.}$$

$$\left(\frac{11.18}{45.28}\right)^2 + \left(\frac{22.36}{68.54}\right)^2 \leq 1$$

$$0.167 \leq 1$$

Hence safe.

2:) Determine the interaction ratio for the

Load = 100 kN e = 200 mm. p = 80 mm.

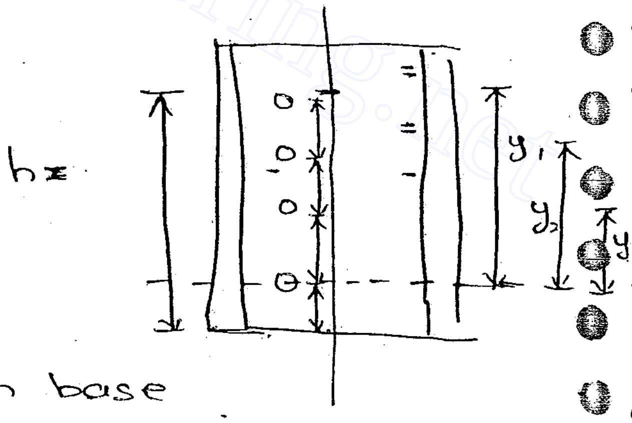
gauge = 40 mm.

[valid for In plane & out plane] $n = \sqrt{\dfrac{6M}{R.\,pm}}$

M – Moment.
R – Rivet value.
p. – ~~80~~ pitch.
m – No. of. rows.

$$n = \sqrt{\frac{6 \times 100 \times 10^3 \times 200}{45.28 \times 10^3 \times 80 \times 2}}$$

$$\boxed{n = 4}$$



h = 80+80+80+40.

h = 280 mm.

Depth of N.A from base

$$= \frac{h}{7}$$

$$= \frac{280}{7}$$

$$= 40 \text{ mm.}$$

$y_1 = 240$ mm $y_2 = 160$ mm. $y_3 = 80$ mm.

$$\boxed{T_b = \frac{M_t \times y_1}{2(y_1^2+y_2^2+y_3^2)}}$$

$$\boxed{M_t = \frac{M}{1+\frac{2b}{21}\frac{\Sigma y}{\Sigma y^2}}}$$

$$M = P\times e = 100\times10^3\times200 = 20\times10^6\ N.m$$

$$\Sigma y^2 = 2\left[240^2+160^2+80^2\right]$$

$$= 179200\ mm^2$$

$$\Sigma y = 2(240+160+80)$$

$$= 960\ mm$$

$$M_t = \frac{20\times10^6}{1+\frac{2\times280}{21}\left(\frac{960}{179200}\right)}$$

$$M_t = 17.5\times10^6\ N.mm$$

$$T_b = \frac{M_t \times y_1}{2(y_1^2+y_2^2+y_3^2)}$$

$$= \frac{17.5\times10^6\times240}{2(80^2+160^2+240^2)}$$

$$\boxed{T_b = 23.4375\ kN}$$

$$V_{dsb} = 45.28\ kN$$

$$T_{db} = 68.51\ kN$$

$$\left(\frac{12.5}{45.28}\right)^2+\left(\frac{23.44}{68.51}\right)^2 \le 1$$

$0.193 \le 1$ Hence safe

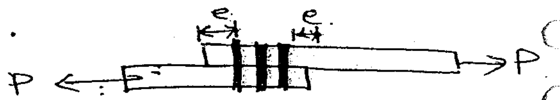
Unwin Formula

Dia of bolt $\boxed{\phi = 6\sqrt{t_{min}}}$

t_{min} - Minimum thickness of plate.

3.) Determine no. of. bolt. 2 bolts 100 x 8mm and 100 x 10mm is connected by lap joint with single row of bolt along the length. use 20mm ϕ and grade 4.6 (M

P = 60mm e = 30mm.



$$\text{No. of. bolt} = \frac{\text{Load}}{\text{Bolt value}}$$

Load = Load carrying capacity of plate

~~Load carryi~~ $T_{dp} = \frac{0.9 f_u}{\gamma_{m1}} \times A_n$.

$$= \frac{0.9 \times 410}{1.25} \times [100 - (20+2)] \times 8$$

$$= 184.20 \text{ kN}$$

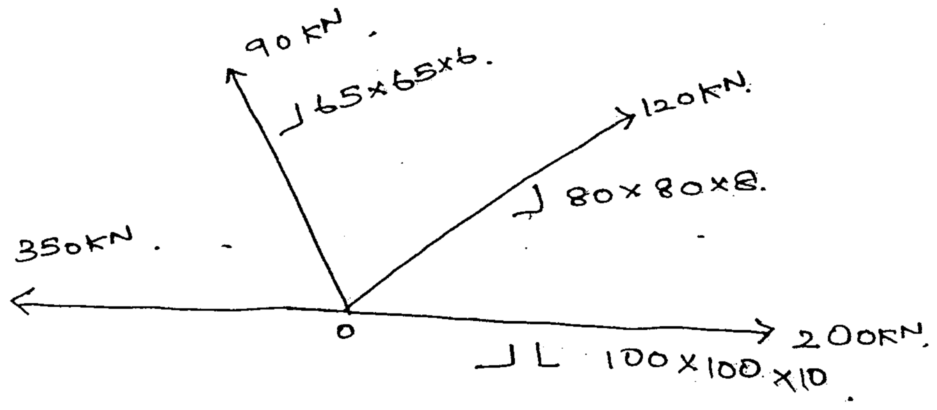
$$\text{No. of. bolt} = \frac{184.20 \times 10^3}{45.28 \times 10^3}$$

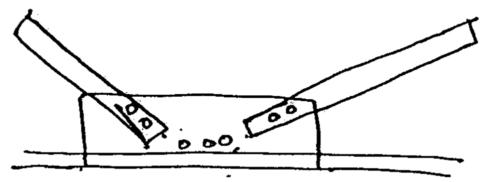
$$= 4.068$$

$\simeq 4$ bolt.

5. Design the joint O as shown in figure

Take P = 60mm, edge = 30mm. use 16mm ϕ bolt. M 4.6. use 10mm thick gusset plate.





Design of Gusset plate.

Net Load = 350 − 200

= 150 kN

Bolt value.

$$V_{dsb} = \frac{0.462 \times f_{ub} \times A_n \times 2}{\gamma_{mb}}$$

$$= \frac{0.462 \times 400 \times \frac{\pi}{4} \times 16^2 \times 2 \times 0.78}{(1.25)}$$

$$= 46.37 \text{ kN}$$

$k_b = \frac{e}{3d_0}, \frac{P}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u},$

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{1.25}$$

$$= \frac{2.5 \times 1 \times 410 \times 16 \times 10}{1.25}$$

$$= 164 \text{ kN}$$

1. Determine Efficiency of the Lap joint connecting two plates 100 x 10 and 100 x 8 mm use 16mm ϕ bolt and M4.6. use 5 bolts.

Solution:

$$\eta = \frac{\text{strength of bolted joint}}{\text{strength of solid plate}}$$

(i) Design Shear strength of bolt. $\frac{f_u / \sqrt{3}}{\gamma_{mb}} \times f_u$

$$V_{dsb} = 0.462 f_u . (n_n A_{nb})$$

$$= 0.462 \times 400 \times 1 \times \left[\left(\pi \times \frac{16^2}{4}\right) \times 0.78\right]$$

$$V_{dsb} = \cancel{37.16}\ 28.98 \text{ kN}.$$

~~Stren~~

(ii) Design bearing strength of bolt.

$$V_{dpb} = 2.5 k_b d \times t \times \frac{f_u}{\gamma_{mb}}.$$

$e = 24mm$. $p = 40mm$

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

$$= \frac{24}{3 \times 18}, \frac{40}{3 \times 18} - 0.25, \frac{400}{410}, 1$$

$$= 0.44, 0.49, 0.975, 1.$$

$$V_{dsb}. = 45.056 \text{ kN}.$$

(iii) ~~Stres~~ Tension strength of bolt

$$V_{dtb} = \frac{0.9 f_{ub} . A_{nb}}{\gamma_{mb}}$$

$$= \frac{0.9 \times 400 \times \frac{\pi \times 16^2}{4}}{1.25}$$

$$= 57.9 \text{ KN.}$$

(iv) Tension in plate.

$$P_t \text{ (plate)} = \frac{0.9 f_u}{\gamma_{mb}} A_{nb}$$

$$= \frac{0.9 \times 410}{1.25} (100 - 18) \times 8$$

$$= 193.65 \text{ KN.}$$

Bolt value $= 5 \times 28.98$

$$= 144.9 \text{ kN.}$$

Strength of solid plate.

$$= \frac{0.9 f_u}{\gamma_{mb}} \times A_g$$

$$= \frac{0.9}{1.25} \times 410 \times 100 \times 8$$

$$= 236.16 \text{ KN.}$$

$$\eta = \frac{144.9}{236.6} \times 100$$

$$\eta = 61.35 \%$$

DESIGN OF FILLET WELD:

* Size of fillet weld

It is distance from root to toe of the fillet weld.

* $S_{min} \nless 3mm$.

*.

Thickness Of Thicker Part		Minimum size.
Over	Upto.	
—	10.	3
10	20	5
20	32	6
32	50.	8 (1st run), & 10mm

Effective Throat Thickness:

It is ⊥r distance from right angle corner of fillet weld to the hypotenuse

Minimum throat thickness for fillet weld $\nless$ 3mm.

$$t_e = k \times \text{Size of weld}.$$

k = constant depending upon fusion angle

Angle	K.
60 - 90	0.7
91 - 100	0.65
101 - 106.	0.6
107 - 113	0.55
114 - 120.	0.5.

END RETURN:

* The fillet weld terminating @ the end or side of the member should be return around the corner whenever practicable for a distance not less than 2S.

* End return are made twice the size of the weld to relieve high stress concentration @ the ends

Welding:

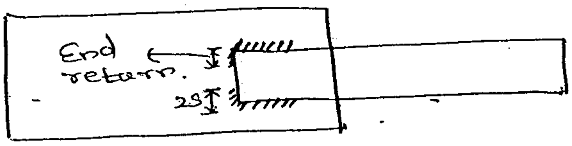
* The effective length of fillet weld

= Total length of weld − 2(weld size

$$\boxed{l_{eff} = l_{total} - 2S}$$

* The minimum end return of the weld is 2 times the weld size.

$$\boxed{\text{End return} = 2S}$$



* The minimum effective length of fillet weld = 4S.

for Fillet weld $\boxed{l_{eff} = 4S}$

* The minimum effective length of fillet weld intermittent welding shall be 4S (or) 40mm whichever is greater. (Discontinous welding).
* The clear spacing of intermittent weld shall not exceed $16t_{min}$ (or) 200mm whichever is less. In case of tension members.

shall not less than. $12t_{min}$ (or) 200mm whichever is less for compression members.

* The maximum permissible stress in fillet weld is 108 MPa − WSM.

$0.462 f_u$ − LSM (shop weld)

$0.385 f_u$ − LSM (Field weld).

* The maximum permissible tensile or compressive stress in welding in fillet weld is 165 MPa.
* The effective thickness of butt weld is $\frac{5}{8} t_{min}$ − single v-joint.

* $l_{eff} = l_{total} - 2S$.

* End return : 2S.

* Weld type | l_{eff} min. | spacing

Weld type	l_{eff} min.	spacing	
Fillet	4S		
Intermittent weld	4S (or) 40 mm whichever is greater.	Comp: 12t min (or) 300 mm	Tens: 16t min (or) 200

* Max permissible stress in weld

- WSM: 108 MPa.
- LSM
  - Shop: 0.462 fu.
  - Field: 0.385 fu
- Tension: 165 MPa
- Compression: 165 MPa.

* Butt Weld.

- Effective thickness
  - Single-V: $\frac{5}{8}t_{min}$
  - Double-V: t_{min}
- fusion angle 60°
- Max permissible stress (same as parent metal)
- width of slot or plug weld: 3t min (or) 25 mm
- Effective area: $t_e \times l_{eff}$
- Design
  - axial: $T_{dw} = \frac{f_y\, l_{eff}\, t_{eff}}{\gamma_{mw}}$
  - Shear: $V_{dw} = \frac{f_{ywd}\, l_{eff}\, t_e}{\gamma_{mw}}$

$f_{ywd} = \frac{f_{yw}}{\sqrt{3}}$ (or) $\frac{f_y}{\sqrt{3}}$

 Double V joint.

* The angle for V-butt weld should be 60
* The max. permissible stress of butt weld is same as that of parent metal.
* The width of the slot (or) plug shall not be less than 3 t mm (or) 25 mm whichever is greater.
* Effective area = (Effective throat thickness) × (Effective length).

* Design strength of butt weld:

a) Design axial strength:

The design strength of butt weld in tension or compression is governed by yield stress.

$$T_{dw} = \frac{f_y \cdot L_{eff} \cdot t_e}{\gamma_{mw}}$$

where.

f_y = smaller of yield stress of weld (or) parent metal

(b) Design shear strength of butt weld:

$$V_{dw} = \frac{f_{yw1}\, L_{eff}\, t_e}{\gamma_{mw}}$$

f_{yw1} is smaller of.

$$\frac{f_{yw}}{\sqrt{3}} \text{ (or) } \frac{f_y}{\sqrt{3}}.$$

Fillet Weld

Size

- $S_{min} \nless 3\,mm$.
- S based thick of thicker plate

thicker part	S
0 - 10	3
10 - 20	5
20 - 32	6
32 - 50	8 (1st run) 10 min

Thickness

- $t_{min} \nless 3\,mm$.
- $t_e = k \times$ size of weld

Angle	k
60 - 90	0.7
90 - 100	0.65
101 - 106	0.6
107 - 113	0.55
114 - 120	0.5

End Return

- $E \nless 2S$.
- weld subjected to e, $\sigma_{reversal}$, Impact Load, Tension. End carrying Bending Load
- Fusion angle. 60 - 120

Design

- $P_{dw} = f_u \times L_{eff} \times t / (\sqrt{3}\,\gamma_{mw})$
- $\gamma_{mw} = 1.25$ (shop), 1.5 (field)
- Reduction
  - → condition $l_j > 150\,t$
  - → $\beta_{lw} = 1.2 - 0.2\,l_j / 150\,t$

* End returns must be provided for welded joints which are subjected to eccentricity. / Stress reversal. (or) impact load.
* Fillet weld is in effective when fusion angle is beyond 60° - 120°.
* This is particularly important in tension end of part carrying bending load.

OVER LAP:

The over lap of plates to be welded ≮ 4 t_{min} (or) 40mm whichever is more.

INTERMEDIATE. (or) Intermittent weld:

It is used when length of fillet weld required to transmitting the force less than the continuous fillet weld.

DESIGN STRENGTH OF FILLET WELD:

$$P_{dw} = \frac{f_u \, t_w \times l_{eff}}{\sqrt{3}\, \gamma_{mw}}$$

f_u - smaller of ultimate strength of the weld (or) parent metal.

γ_{mw} - 1.25 shop weld
1.5 Field weld

Reduction factor for long joint:

If the max. length of side weld transfering shear along its length exceeds 150 times the throat size, the reduction in weld strength as per long joint. β

The design capacity of the weld is is reduced by a factor.

$$\beta_{tw} = 1.2 - \frac{0.2\, l_j}{150\, t_w}.$$

l_j - length of joint in force transfer.

26|11|2015

D.S.S

WELD.

FILLET WELD.

1) Two 16mm thick plate are joined in a shop single U butt weld, Double U butt weld. l_{eff} = 300mm. Determine design strength of welded joint f_y = 250MPa and f_u = 410MPa of the weld and steel.

Solution:

(i) Single U butt

$$T_{db} = \frac{f_y \times l_{eff} \times t_e}{\gamma_{mb}}$$

$$t_e = \frac{5}{8} \times t_{min} = \frac{5}{8} \times 16$$

$$t_e = 10\,mm$$

$$= \frac{250 \times 300 \times 10}{1.25}$$

$$= 600\,kN.$$

(ii) Double U butt.

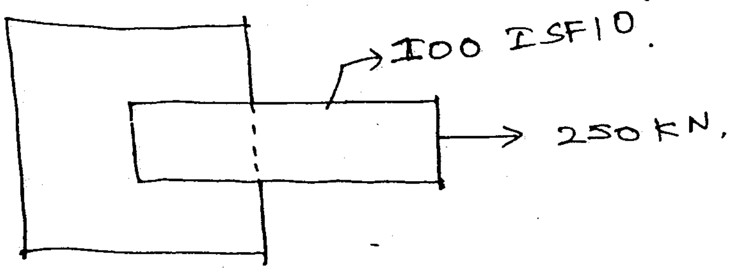
$$t_e = t_{min}$$

$$T_{db} = \frac{f_y \times l_{eff} \times t_e}{\gamma_{mb}}$$

$$= \frac{250 \times 300 \times 16}{1.25}$$

$$= 960\,kN,$$

2. Determine s, l and overlap b/w plates of the fillet weld lap connection to transmit P = 250 kN. f_{y_ω} = 250 MPa f_{u_ω} = 410 MPa. Assume shop weld (γ_{mb} = 1.25). & width of plate 100 mm.



$t_e = 0.7 \times S$

$$P_{dw}' = \frac{f_u \times l_e \times t_e}{\sqrt{3} \cdot \gamma_{mb}}$$

$S_{min} = 3mm$

$S_{max} = t_{min} - 1.5$

$S_{max} = 10 - 1.5$

$= 8.5mm$

$\boxed{S = 6mm}$

$$250 \times 10^3 = \frac{410 \times l_e \times 4.2}{\sqrt{3} \times 1.25}$$

$l_e = 314.32$

$\boxed{l_e = 315 \text{ mm}}$

$t = 0.7 \times 6$

$\boxed{t = 4.2mm}$

over lap length $= \frac{315 - 100}{2}$

$\boxed{\text{over lap length} = 107.5mm}$

Check:

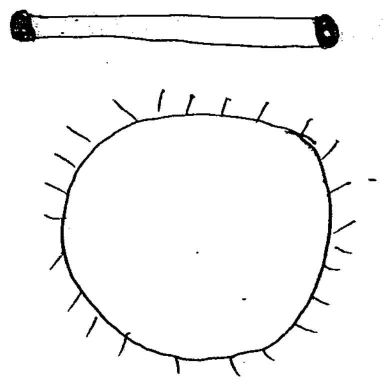
Min over lap = $4 t_{min}$ (or) 40mm whichever is great

$= 4 \times 10$

min over lap = 40 mm.

Hence safe.

3. A circular plate 100mm ϕ is welded to another plate by means of $S_{weld} = 6mm$. Fillet weld. Calculate twisting Moment capacity that can be resisted by fille welded connection. Use steel of grade 410 and shop welding.



$l_{eff} = \pi d$

$= \pi \times 100$

$\boxed{l_{eff} = 100\pi}$

$t_e = 0.7S$

$= 0.7 \times 6$

$\boxed{t_e = 4.2mm}$

$$P_{dw} = \frac{f_u \times l_{eff} \times t_e}{\sqrt{3} \times \gamma_{mb}}$$

$$= \frac{410 \times 100 \times \pi \times 4.2}{\sqrt{3} \times 1.25}$$

$P_{dw} = 249.869\ kN.$

Twisting Moment $= P_{dw} \times C.G$

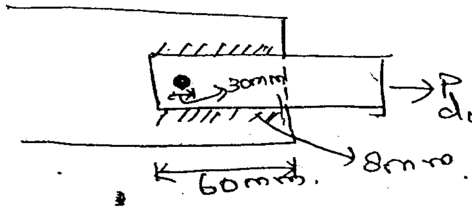
$= 249.86 \times 10^3 \times \frac{d}{2}$

$= 249.86 \times 10^3 \times \frac{100}{2}$

$= \cancel{24}\ \cancel{124}\ 12.493\ kN\,m$

$\boxed{\text{Twisting Moment} = 12.493\ kN.m}$

4. Determine service load permitted on connection as shown in the figure. Use grade 410.



solution:

$$P_{dw} = P_{dwf} + P_{dwp}$$

$$= \frac{f_u \times l_e \times t_e}{\sqrt{3} \times \gamma_{mb}} + \frac{f_u \times A}{\sqrt{3} \times \gamma_{mb}}$$

$$= \frac{410 \times 120 \times 0.7 \times 8}{\sqrt{3} \times 1.5} + \frac{410 \times \pi \times 30^2}{4 \times \sqrt{3} \times 1.5}$$

$$P_{dw} = 217.59\,kN$$

$$\text{Service load} = \frac{P_{dw}}{1.5} = \underline{145.06\,kN}$$

5. Determine "P" on a butt weld joint size 150 × 10mm and 150 × 6mm. $f_y = 250\,MPa$

Use (i) single V ~~butt~~ weld

(ii) Double V butt weld. Shop weld

$$T_{db} = \frac{f_y\, l_{eff} \times t_e}{\gamma_{mb}}$$

$f_u = 410$

$f_y = \frac{410}{1.5}$

$= 273.33$

$f_y = 250\,MPa$

$t_e = \frac{5}{8} \times 6$

$= 3.75\,mm$

~~27.33~~

$$= \frac{273.33 \times 150 \times 3.75}{1.25}$$

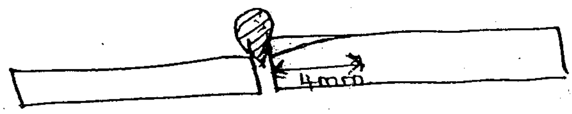
$$\boxed{T_{db} = 122.99\,kN}$$

$$T_{db} = \frac{273.33 \times 150 \times 6}{1.25}$$

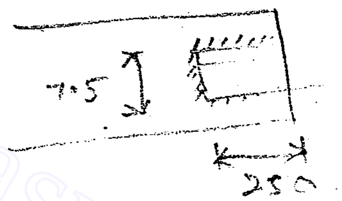
$$\boxed{T_d = 196.8\,kN}$$

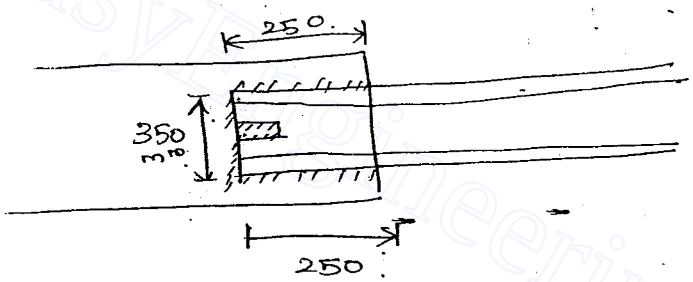
Note :

* welding in case of unequal thickness. slope in provide in 1 in 5.



6.) Determine length of slot with gusset plat where c/s area of channel is 3200 mm^2 t_w = 7.5 mm and lap length = 250 mm. depth of channel 350 mm.





$f_u = 410$ MPa.

$t_e = 0.7 \times S$.

$S_{min} = 3$ mm

$S_{max} = 7.5 - 1.5$

$S_{max} = 6$ mm.

$\boxed{S = 6\text{mm}}$

$t_e = 0.7 \times 6 = 4.2$ mm.

$$P_{dw} = \frac{f_u \, l_{eff} \, t_e}{\sqrt{3}\, \gamma_{mb}}.$$

$$P_{dw} = P_{channel}.$$

$$P_c = \frac{f_y}{\gamma_{mo}} A.$$

$$= \frac{250}{1.1} \times 3200.$$

$$\boxed{P_c = 727.272 \text{ kN}}$$

$$727.272 \times 10^3 = \frac{410 \times l_{eff} \times 4.2}{\sqrt{3} \times 1.25}.$$

$$l_{eff} = 914.39 \text{ mm}.$$

$$l_{eff} \simeq 915 \text{ mm}.$$

~~7207~~.27

$l_{eff} = 250 + 250 + 350 + \text{slot length}_{\text{total}}$

$915 = \cancel{250}\ 850 + S_{total}$

$S_{total} = 65\ mm.$

length in one side of slot $= \frac{65}{2}$

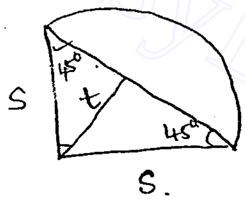
$= 32.5mm$

width of slot $= 25\ mm$ or $3t_{min}$

$= 3 \times 7.5$

$= 22.5\ mm.$

Note:



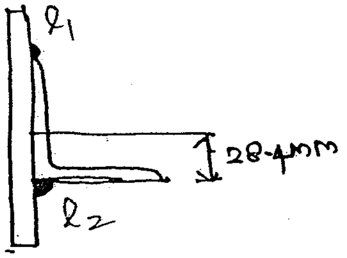
$\sin 45° = \frac{t_{weld}}{S}$

$t_w = \frac{1}{\sqrt{2}} S$

$\boxed{t_w = 0.7S}$

7. Determine length of weld @ top & bottom

$S = 6mm$ ISA $100 \times 100 \times 10$. $P = 250 kN$.



$$P_{dw} = 0.462 \times f_u \times l_e \times t_e$$

$$250 \times 10^3 = 0.462 \times 410 \times l_e \times 0.7 \times 6$$

$$l_e = 314.24mm.$$

$$l_1 y_1 = l_2 y_2$$

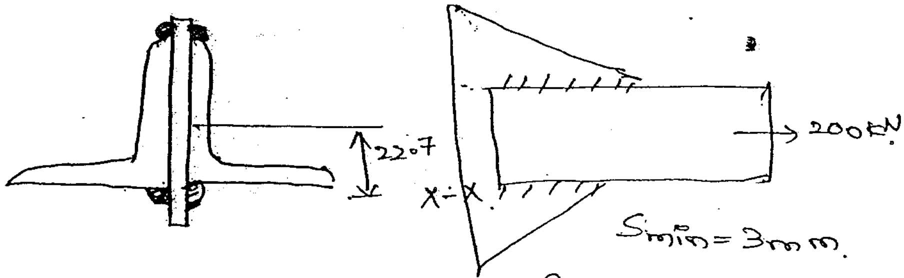
$$l_1 = l_2 \frac{28.4}{71.6}.$$

$$l_e = l_1 + l_2.$$

$$314.24 = \frac{28.4}{71.6} l_2 + l_2.$$

$$\boxed{\begin{aligned} l_2 &= 225.54mm. \\ l_1 &= \cancel{88.7}\ 89.46\ mm. \end{aligned}}$$

8. A tie member of truss consists of double angle section 80×80×8 mm welded on opposite side of 12mm thick gusset plate. Design a fillet weld to make the joint at P = 200KN.



$S_{min} = 3mm.$

$S_{max} = 8-1.5$
$= 6.5mm.$

$l_w = 2(x_1+x_2).$

$$\text{Strength of weld/mm} = 0.462 \times f_u \times t_e$$
$$= 0.462 \times 410 \times 0.7 \times 6.$$
$$= 795.584 \text{ N/mm}.$$

Strength of weld = Load

$$2 \times 795.584 (x_1+x_2) = 200 \times 10^3.$$
$$x_1+x_2 = 125.69 mm.$$

Moment about X-X.

$$2 \times 795.584 \times x_1 \times 80 = 200 \times 10^3 \times 22.7.$$
$$x_1 = 35.66 mm$$
$$x_2 = 90.02 mm.$$

$$L_w = 2(x_1+x_2) = 251.36 mm.$$

Note:

A Large size weld requires more than 1 run of welding which means that after first run of welding chipping and cleaning will be required for proper bond for successive run. This increase the cost

* A smaller weld size will be cheaper than larger ~~load~~ one for the same strength, considering volume of the weld.

For (eg:) A 300mm length of 5mm size weld will have the same strength as a 150mm long 10mm size fillet weld. (115.5kN)

However the volume of weld metal for a 10mm size will be twice as that of 5mm size.

l	S
300 mm	5mm
150 mm	10mm.

Volume of 5mm $= \frac{1}{2} \times 5 \times 5 \times 300 = 3750 \, mm.$

V of 10mm $= \frac{1}{2} \times 10 \times 10 \times 150 = 7500 \, mm.$

Eccentric connection.

In plane moment.

$$F_S = \frac{P}{(b+c+l)}$$

$$\tau = \frac{P \times e \quad x \cdot r_{max}}{I_{xx} + I_{yy}}$$

$$e = a + (b - \bar{x})$$

$$\cos\theta = \left(\frac{b - \bar{x}}{r}\right)$$

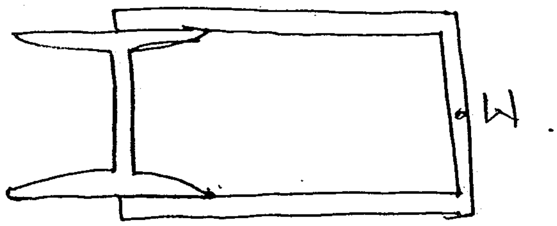
$$f_r = \sqrt{f_s^2 + \tau^2 + 2 f_s \tau \cos\theta}$$

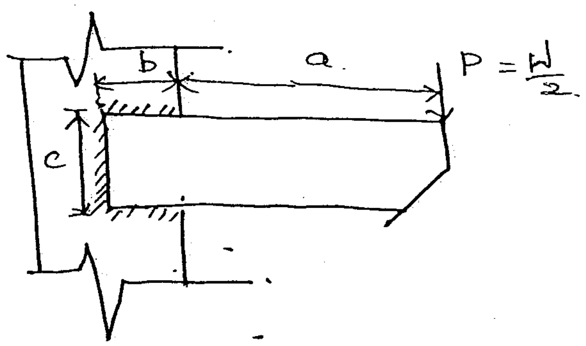
$$f_r \leq 0.462 f_u$$

$$\leq 0.385 f_u$$

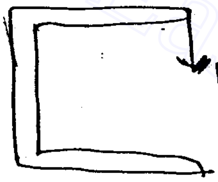
Eccentric Connection

In plane Moment (Torsional shear)



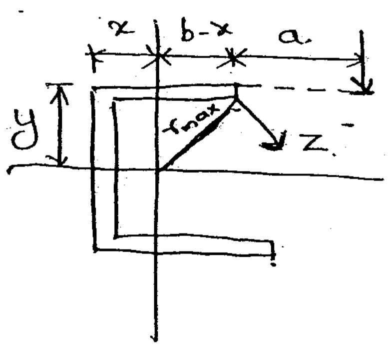


1.) Direct Shear.



$$F_s = \frac{\text{Load}}{\text{welded area}}$$

$$\boxed{F_s = \frac{P}{(b+c+b)\times t}}$$



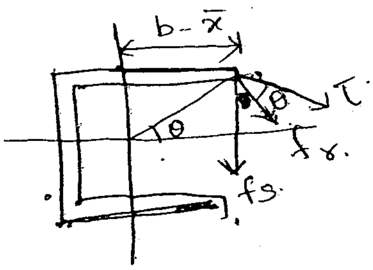
$$\frac{T}{J} = \frac{\tau}{R}$$

$$\tau = \frac{T}{J} \times R$$

$$\boxed{\cos\theta = \frac{(b-\bar{x})}{r}}$$

$$\boxed{\tau = \frac{P\times e}{I_{xx}+I_{yy}} \times r_{max}}$$

$$\boxed{e = a + (b-x)}$$



$$f_r = \sqrt{f_s^2 + \tau^2 + 2 f_s \tau \cos\theta}$$

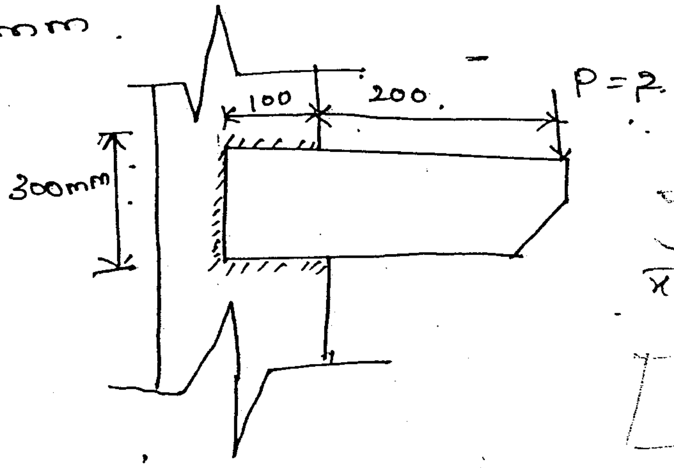
$f_r \le$ Permissible stress

$\le 0.4b2 f_u$ (Shop)

$\le 0.385 f_u$ (Field)

1. Determine Load carrying capacity use.

$S = 6mm$.



$I_{xx} = 28.3 \times 10^6 mm^4$

$I_{yy} = 1.96 \times 10^6 mm^4$

$y = 150mm$

$\bar{x} = 20mm$

$\boxed{P = 102.7KN}$

Solution:

$S = 6mm$. $t = 6 \times 0.7 = 4.2mm$. $\boxed{t = 4.2mm}$

Total Area of weld $= (300 \times 4.2) + 2(100 \times 4.2)$

$= 2100mm^2$

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2 + a_3 y_3}{A_{total}}$$

$$= \frac{(100 \times 4.2 \times 297.9) + (300 \times 4.2 \times 150) + (100 \times 4.2 \times 2.1)}{2100}$$

$$\boxed{\bar{y} = 150mm}$$

$$\bar{x} = \frac{a_1 x_1 + a_2 x_2 + a_3 x_3}{2100}$$

$$= \frac{2(100 \times 4.2 \times 50) + (300 \times 4.2 \times 0)}{2100}$$

$\bar{x} = 20mm$

$$I_{xx} = 2\left[\frac{b_1 d_1^3}{12} + A(\bar{y} - y_1)^2\right] + \frac{b_2 d_2^3}{12} + A(\bar{y} - y_1)^2$$

$$= 2\left[\frac{100 \times 4.2^3}{12} + 100 \times 4.2 \times 150^2\right] + \frac{300 \times 4.2^3}{12} + 300 \times 4.2 \times (0)$$

$$\boxed{I_{xx} = 28.35 \times 10^6 mm^4}$$

$$I_{yy} = \frac{300 \times 4.2^3}{12} + 300 \times 4.2(20 - 0)^2$$

$$+ 2\left[\frac{100^3 \times 4.2}{12} + 100 \times 4.2(50 - 50)^2\right]$$

$$\boxed{I_{yy} = 1.96 \times 10^6 mm^4}$$

$$I_{zz} = 28.35\times10^{6} + 1.96\times10^{6}$$

$$= 30.31\times10^{6}.$$

$$f_s = \frac{P\times1000}{A_{weld}} = \frac{P\times}{2100} = 4.762\times10^{-4} = 0.04762$$

$\cancel{0.047}$

$$\tau = \frac{P\times e}{I_{zz}}\times r_{max}.$$

$e = 80+200$,

$$\boxed{e = 280mm}$$

$$r_{max} = \sqrt{150^2+80^2}$$

$$\boxed{r_{max} = 170mm}$$

$$= \frac{P\times280\times170}{30.31\times10^{6}}$$

$$\tau = 1.57\times10^{-3}P.$$

$$= 1.57P.$$

$$\theta = \tan^{-1}\left(\frac{80}{150}\right)$$

$$\theta = 28.072.$$

$$f_r = \sqrt{(1.57\times10^{-3}P)^2 + (4.762\times10^{-4}P)^2 + 2(1.57\times10^{-3}\times4.762\times10^{-4}\cos28.072}$$

$$f_r = 2.0026\times10^{-3}P.$$

$$f_r = \frac{f_u}{\sqrt{3}\times1.25} = \frac{410}{\sqrt{3}\times1.25}$$

$$= 189.37\ N/mm^2.$$

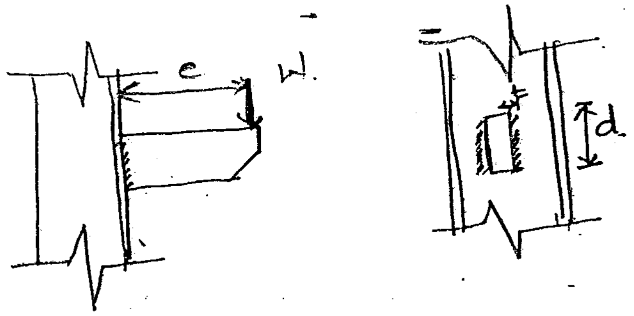
$$2.0026\times10^{-3}P = 189.37.$$

$$P =$$

5/12/2014

STEEL STRUCTURES

OUTPLANE MOMENT.



Direct shear stress.

$$\boxed{q = f_s = \frac{P}{2(d \times t)}}$$

Bending stress $\frac{M}{I} = \frac{f}{y}$.

$$f_a = \frac{M}{I} \times y.$$

$$\boxed{f_a = \frac{P \times e}{2\left(\frac{t \times d^3}{12}\right)} \times \left(\frac{d}{2}\right)}$$

Resultant stress:

$$\boxed{f_r = \sqrt{f_s^2 + f_a^2}}$$

(According to WSM)

$$\boxed{f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{(f_u/\sqrt{3})}{\gamma_{mb}}}$$

(According to LSM)

* out plane Moment · ⟶ Direct stress.
  ⟶ Bending stress

*· Direct Stress

$$\boxed{f_s = \frac{P}{2(d t)}}$$

* Bearing Stress

$$f_a = \frac{P \times e}{2\left(\frac{t \times d^3}{12}\right)} \times y.$$

* Equivalent stress:

$$f_e = \sqrt{3(f_s)^2 + f_a^2} \quad \text{LSM} \leq 0.450 f_u \quad \frac{\frac{f_u}{\sqrt{3}}}{1.1}$$

WSM

$$f_r = \sqrt{f_s^2 + f_a^2}.$$

*· Butt weld.

thickness of weld t = thickness size of plate

length of weld d = depth of plate.

$$f_e = \sqrt{3 f_s^2 + f_a^2} \leq \frac{f_y}{1.1}.$$

1. Determine Load carrying capacity of 300 mm B and 16 mm t is connected to a flange of a column with 6 mm fillet weld on both side. Load is applied 200 mm from the face of the column.

Given:

$t = 0.7 \times S$
$= 0.7 \times 6$
$\boxed{t = 4.2\,mm}$

$$q = f_s = \frac{P}{2(d \times t)}$$

$$= \frac{P}{2 \times 4.2 \times 300} = 3.97 \times 10^{-4}.P.$$

$$\boxed{f_s = 3.97 \times 10^{-4} P}$$

$$f_a = \frac{P \times e}{2\left(\frac{t \times d^3}{12}\right)} \times \frac{d}{2}.$$

$$= \frac{P \times 200}{2\left(\frac{4.2 \times 300^3}{12}\right)} \times \frac{300}{2}.$$

$$\boxed{f_a = 1.587 \times 10^{-3}.P.}$$

$$f_e = \sqrt{3f_s^2 + f_a^2}.$$

$$= \sqrt{3(3.97 \times 10^{-4}P)^2 + (1.587 \times 10^{-3}P)^2}.$$

$$f_e = 1.7298 \times 10^{-3}.P.$$

$$f_e \leq 0.462 f_u.$$

$$1.7298 \times 10^{-3} P \leq 0.462 f_u$$

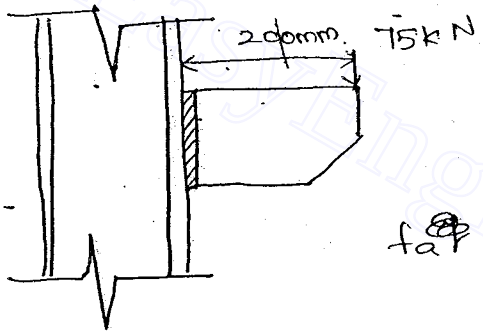
$$P \leq \frac{0.462 \times 410}{1.7298 \times 10^{-3}}$$

$$P \leq 109.5\ KN.$$

$$\boxed{P = 109\ KN}$$

2. An I - Section is connected to a flange on a col. cantilever - I section @ a ~~d~~ $e = 300\,mm$. $b_f = 150\,mm$, $D = 300\,mm$. Length of ~~f~~ weld of each flange 100mm ad 150mm either side of web. $S = 6\,mm$ for flange $S = 5\,mm$ for web. what is the load carrying capacity.

3. Determine the max. stress developed in a butt weld. connecting flange plate with a flange of a column. $P = 75\,kN$ is applied on the plate @ $e = 200\,mm$. Size of plate 300×12.



$$f_e = \sqrt{3q^2 + f_s^2}$$

$$f_a = \frac{P \times e}{\frac{t \times d^3}{12}} \times \frac{y}{2}$$

$$= \frac{75 \times 10^3 \times 200}{\frac{12 \times 300^3}{12}} \times \frac{300}{2}$$

$$f_a = 83.33\ N/mm^2$$

$$q = f_s = \frac{75 \times 10^3}{12 \times 300} = 20.833$$

$$f_e = \sqrt{3(83.33)^2 + (20.833)^2} = \sqrt{3(20.833)^2 + (83.33)^2}$$

$$\boxed{f_e = 145.82\ N/mm^2} \qquad f_e = 90.81\ N/mm^2$$

$$\text{Max stress} = \frac{f_y}{1.1} = \frac{\cancel{218}\ 240}{1.1}$$

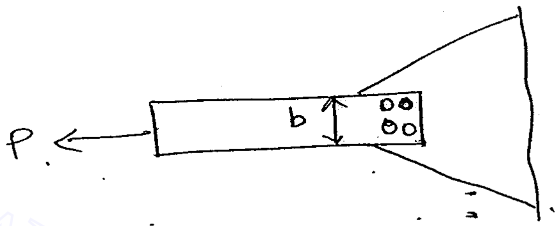
$$= 218.18 > \cancel{145.82}\ 90.81\ N/mm^2$$

Hence safe.

TENSION MEMBER.

* Member subjected to axial tension.
* Eg: Tie of a roof truss.
* The load carrying capacity of a tension member is of two criteria.
  - → yield criteria (Based on A_g)
  - → Rupture criteria (Based on A_n).

*



Yielding criteria:

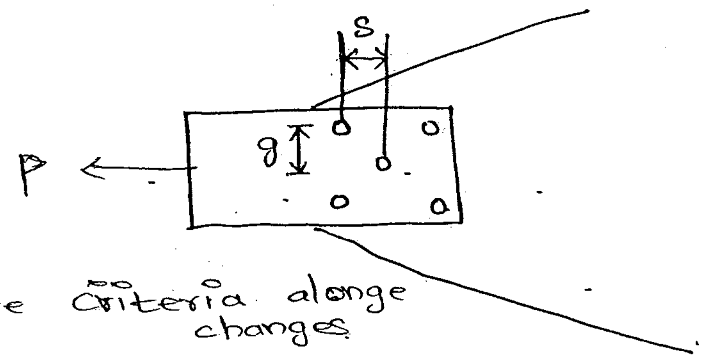
$$P_{at} = \frac{f_y}{1.1} \times A_g.$$

Rupture criteria.

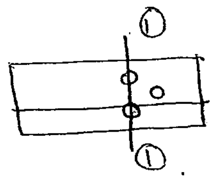
$$P_{at} = \frac{0.9 f_u}{1.25} \times A_n.$$

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

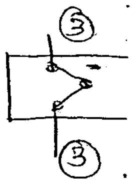
*. For Staggered bolting.



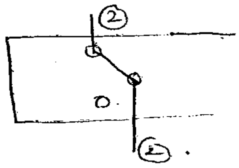
case 1:



case ③



case 2:



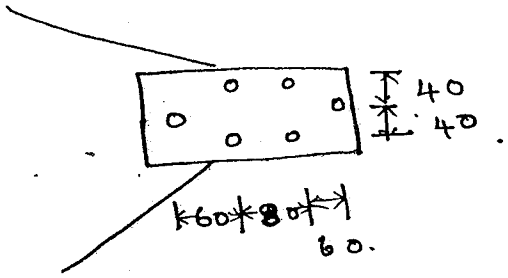
Rupture criteria alonge changes

$$P_{at} = \frac{0.9 f_u}{1.25} \times A_n.$$

$$A_n = \left[b - n d_h + \frac{S_1^2}{4g} + \frac{S_2^2}{4g} + \cdots \right] \times t.$$

1. A tension member is of flat plate 200mm x 10mm. It is connected to a gusset plate with 6 bolt in 2rows in chain bolting. $\phi_{bolt} = 16mm$ $g = 60mm$. $p = 80mm$. Determine load carrying capacity.

(ii) In the above numerical if it staggered bolting Determine P_{at}



case(i):

Yield criteria:

$$P_{at} = \frac{f_y}{1.1} \times A_g$$

$$= \frac{240}{1.1} \times (200 \times 10)$$

$$= 436.36 \text{ kN}.$$

Rupture criteria.

$$P_{at} = \frac{0.9 f_u}{1.25} \times A_n.$$

$$= \frac{0.9 \times 410}{1.25} \left[[200 - 3(18)] \times 10 \right]$$

$$= 430.992 \text{ kN}.$$

Load carrying capacity = 430 kN.

case(ii)

Yielding criteria:

$$P = \frac{f_y}{1.1} \times A_g = 436.36 \text{ kN}.$$

Rupture criteria:

$$A_{n1} = (200 - 18) \times 10 = 1820 mm^2$$

$$A_{n2} = \left(200 - 2(18) + \frac{60^2}{4 \times 40}\right) \times 10 = 1865 \ mm^2$$

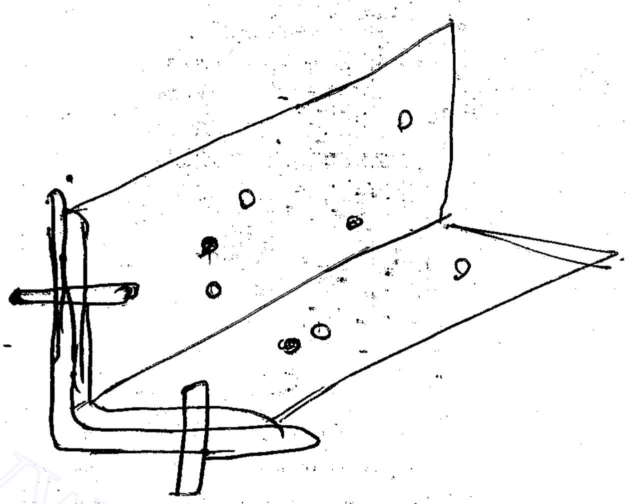
$$A_{n3} = \left[200 - 3(18) + 2\left(\frac{60^2}{4 \times 40}\right)\right] 10 = 1910 mm^2$$

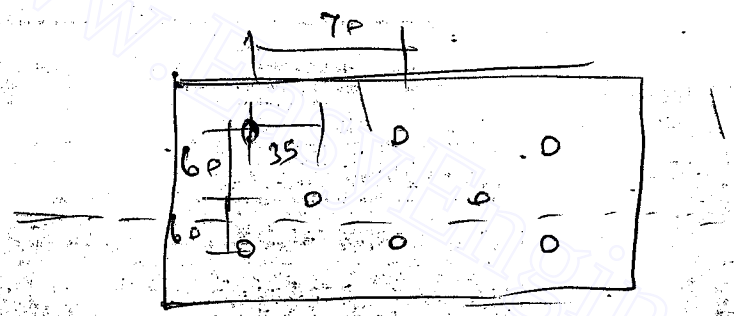
$$P = \frac{0.9 f_u}{1.25} \times A_n.$$

$$= \frac{0.9 \times 410}{1.25} \times 1820.$$

$$P = 537.26 \text{ kN}.$$

Design Load = 435.2 kN.




70
35
60
60

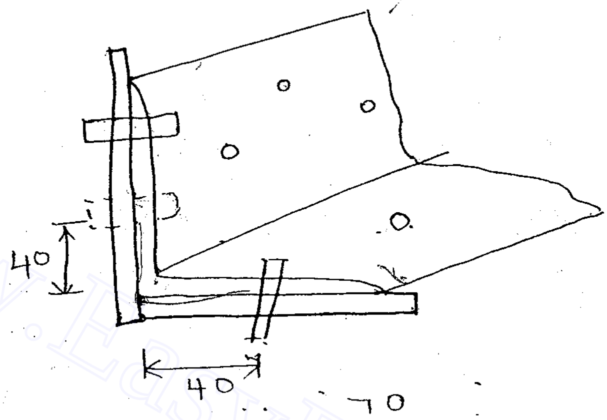
6/12/2015

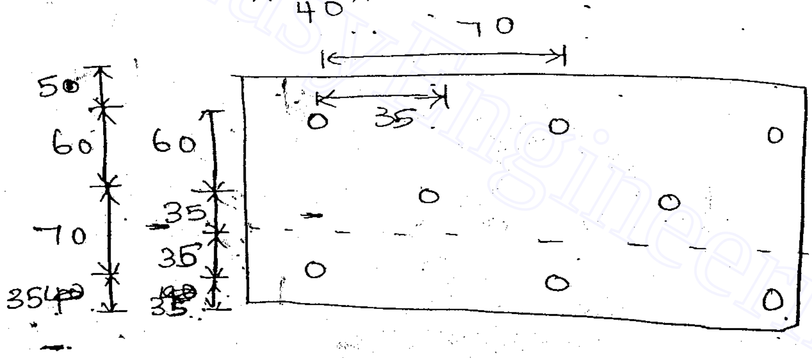
D.S.S

Tension Member.

1. A tension member consists of an angle. 150 x 75 x 10 is connected with gusset plate on both the sides. The longer leg consists of 2 rows of bolt spaced @. g = 60mm. p = 70mm. position of the bolt is 40mm from the corner of the both the leg. Determine Anet and ultimate load.

d = 16.





$$A_{n1} = 150 - \frac{10}{2} + [215 - 2(18)] \times 10$$

$$= 1790 \text{ mm}^2$$

$$A_{n2} = 215 - 2(18) + \frac{35^2}{4 \times 60}$$

$$= 1841.04 \text{ mm}^2$$

$$A_{n3} = 215 - 3(18) + \frac{35^2}{4 \times 60} + \frac{35^2}{4 \times 70}$$

$$= 1884.79 \text{ mm}^2$$

$$= 1704.79 \text{ mm}^2$$

Yield

$$P_{at} = \frac{f_y \times \overset{215\times10}{\cancel{245\times10}}\left[(150\times10)+(75\times10)\right]}{1.1}$$

$$= 488.63 \text{ kN}.$$

Rupture

$$P_{at} = \frac{0.9 f_u A_{n1}}{1.25}$$

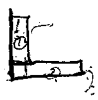
$$= \frac{0.9 \times 410 \times 1704.79}{1.25}$$

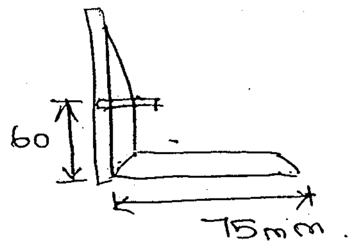
$$= 503.25 \text{ kN}.$$

2. Single angle 125 × 75 × 10 mm, $\phi_{bolt} = 20mm$
In single row the gauge is 60mm from corner. Determine the ultimate load carrying capacity and no. of. bolt use yielding and rupture criteria.

Solution:

Note: If no. of. bolt is not given Assume no. of. bolt and proceed atlast actual no. of. bolt can be calculated.





1.) Yielding crieteria.

$$P_{at} = \frac{f_y}{1.1} \times A_g.$$

$$= \frac{250}{1.1} \times (125+75-10)\,10.$$

$$P_{at} = \cancel{488.64\,kN}\ 431.82\,kN.$$

2.) Net section ruptare.

$$P_{at} = \frac{0.9 f_u A_{nc}}{1.25} + \beta \frac{f_y \times A_g}{1.1}.$$

$$= 0.9 \times$$

$$\beta = 1.4 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right).$$

$\omega = 75\,mm$ $\quad t = 10\,mm$

$b_s = 60 + 75 - 10 = 125\,mm$

$L_c = P_1 + P_2 + P_3 + P_4 \quad 4(P)$

$= \cancel{4(2.5 \times \ldots)} = 4(2.5d)$

$$\boxed{L_c = 200\,mm}$$

$\beta \geq 0.7$

$\beta \leq \frac{(f_u/\gamma_{m1})}{(f_y/\gamma_{m0})}$

$$\beta = 1.4 - 0.076\left(\frac{75}{10}\right)\left(\frac{250}{410}\right)\left(\frac{125}{200}\right)$$

$$\boxed{\beta = 1.182}$$

$$P_{at} = \frac{0.9 \times 410 \times (125-5-22) \times 10}{1.25} + \frac{250 \times (75-5) \times 10}{1.1} \times 1.182$$

$P_{at} = 477.34\,KN.$

Design load = ~~477.34 KN~~ 430 KN.

No. of bolt $= \frac{430}{45.6}$

$= 9.43$

$\simeq 10$ bolts.

finished (1)

19/12/2015

R.C.C.

COLUMN. (COMPRESSION MEMBER).

* Types..

- Transverse Reinforcement
- Based on Shape.
- Based on $\frac{l}{b}$ ratio.
- Based on loading.

*. Based on $\frac{l}{b}$ ratio.

a) $\frac{l}{b} < 3$. pedestal.

b) $3 < \frac{l}{b}$ 12. Short column

c) $60 \not> \frac{l}{b} \geq 12$ long (or) slender column.

* Based on shape:

(a) Rectangle

(b) square

(c) circular.

* Based on. transverse reinforcement

(a) Lateral ties

(b) circular ties

(c) Helical (or) hoop reinforcement

*. Based on Loading.

(a) Axial Loaded column.

(b) Axial with uniaxial bending

(c) Axial with biaxial bending.

* Minimum no. of. bar.

Square & Rectangle. – 4 bars.

Circular. – 6 bars

hexagon – 6 bars (one @ each corner)

* Min dia of bar for column 12mm (to avoid buckling)

* Load carrying capacity of column.

For $P_u = P_{concrete} + P_{steel}$

For ~~lateral~~ transverse reinforcement with lateral ties.

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}.$$

For helical reinforcement, $P_u = 1.05 \left[0.4 f_{ck} A_c + 0.67 f_y A_{sc}\right]$

* E_{min}. (check for minimum Eccentricity).

$$e_{min} = \frac{l_x}{500} + \frac{b}{30} \text{ (or) } 20\,mm.$$ whichever is greater.

Axial Load condition $e_{min} < 0.05b$.

$$e_{min} = \frac{l_y}{500} + \frac{d}{30} \text{ (or) } 20\,mm.$$

Axial load condition $e_{min} < 0.05d$.

$$e_{min} = \frac{(\text{unsupported length})}{500} + \left(\frac{\text{least lateral dimension}}{30}\right)$$

Least value of $e_{min} = 20\,mm$.

* Assumption:

→ The Max. axial compressive strain in concrete is taken as 0.002 $[\varepsilon_{cac} = 0.002]$

→ The max. compressive strain @ highly compressed extreme fibre in concrete subjected axial compression and bending and when there is no tension in the section shall be.

$$0.0035 - 0.75 \left[\text{strain in least compressive extreme fibre}\right]$$

* Slenderness limits:

→ when both ends are restrained $\quad l \not> 60b$

→ when one end is restrained. $\quad l \not> \frac{100b^2}{D}$

* Cover concrete.

when size of column < 200mm clear cover = 25mm

when size of column ≥ 200mm clear cover = 40mm

* Min % of reinforcement = 0.8% of Ag.

* Max % of reinforcement

→ 6% of Ag. (without overlapping)

→ 4% of Ag (with overlapping)

* Min % of reinforcement = 0.15% Ag. for pedestal.

* Spacing of longitudinal bars. ≯ 300 mm.

* ϕ of lateral ties. = $\frac{1}{4} \times \phi$ largest main bar.

= 6 mm (whichever is greater)

* Pitch of Lateral ties.

→ ≯ b. (least lateral dimension)

→ ≯ 16 ϕ smaller main bar.

→ ≯ 300 mm.

whichever is smaller.

* Pitch for helical ties.

→ ≮ 25mm.

→ ≮ 3. ϕ_{helix} (which ever is small)

→ ≯ 75 mm.

→ ≯ $\frac{1}{6}$ D_{core}.

* short column with Biaxial Bending

The load contour method given by. breseller. in 1960.

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n} \leq 1$$

M_{ux}, M_{uy} = External moment.

M_{ux1}, M_{uy1} = Moment ~~capa~~ capacity.

α^n depends on $\left(\frac{P_u}{P_{uz}}\right)$.

* slender compression. Method.

The addition moment is given by the formula.

$$M_{ax} = \left(\frac{PD}{2000}\right)\left(\frac{L_{ex}}{D}\right)^2.$$

$$M_{ay} = \left(\frac{PB}{2000}\right)\left(\frac{L_{ey}}{B}\right)^2.$$

* A bore moment should be added to. eccentric loads.

* why ~~to~~ do we provide lateral ties?
  - → To avoid buckling.
  - → to hold longitudinal reinforcement
  - → to provide confined surface

* Load carrying Capacity of column

→ If $e = 0$. (Ideal (or) Purely axialy)

$$P_u = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}.$$

→ If $e \leq 0.05b$

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}.$$

(11% reduction of ideal column)

1. Determine the concentric load carrying capacity of circle column. D = 500mm. Use M_{20} and Fe 415. 8 bars 25mm ϕ is applied.
   (a) If small eccentricity what is P_u.
   (b) If helical reinforcement is provided for above two cases;

Solution:

$$A_g = \frac{\pi}{4} \times 500^2 = 196.35 \times 10^3 \text{ mm}^2$$

$$A_{st} = 3926.99 \text{ mm}^2.$$

$$A_c = 196.35 \times 10^3 - 3926.99.$$

$$A_c = 192.42 \times 10^3 \text{ mm}^2$$

case (i) $e = 0$.

$$P_u = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$$

$$= (0.45 \times 20 \times 192.42 \times 10^3) + (0.75 \times 415 \times 3926.99)$$

$$\boxed{P_u = 2954.06 \text{ kN}}$$

case (ii) $e \leq 0.05b$.

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{st}$

$= (0.4 \times 20 \times 192042 \times 10^3) + (0.67 \times 415 \times 3926.9$

$= \cancel{2063}$ $\boxed{P_u = 2631.26 \text{ kN}.}$

case (iii) For helical reinforcement.

$e = 0$.

$P_u = 1.05\ (2954.06)$.

$\boxed{P_u = 3101.763 \text{ kN}.}$

$e \leq 0.05b$.

$P_u = 1.05\ (2631.26)$.

$\boxed{P_u = 2762.82 \text{ kN}.}$

* longitudinal bars. must be tied by single. 48ϕ ties must be tie and beyond that additional ties are provide.

Max angle of hook. = 135°.

1. A reinforced concrete column contains longitudinal steel = 1% net c/s Area of the column. Assume $m = 10$. the load carried ~~and~~ by steel and concrete is P_s and P_c. The ratio of $\frac{P_s}{P_c}$, expressed in percentage is _____. (use elastic theory).

Solution:

$$P = \sigma_c A_c + \sigma_{sc} A_{sc}.$$

$$\boxed{A_{sc} = \frac{1}{100} \times A_g.} \qquad \boxed{A_c = \frac{99}{100} A_g.}$$

$$P_s = \frac{\sigma_{sc} A_g}{100}. \qquad P_c = \frac{\sigma_c \, 99 A_g}{100}.$$

$$\frac{P_s}{P_c} = \frac{\left(\frac{\sigma_{sc} \cancel{A_g}}{\cancel{100}}\right)}{\left(\frac{\sigma_c \, 99 \cancel{A_g}}{\cancel{100}}\right)}$$

$$\varepsilon_{sc} = \varepsilon_c.$$

$$\sigma_{sc} / E_s = \sigma_c / E_c$$

$$\boxed{\frac{P_s}{P_c.} = \frac{\sigma_{sc}}{\sigma_{cc} \; 99.}}$$

$$\frac{\sigma_{sc}}{\sigma_c} = \frac{E_s}{E_c}.$$

$$\frac{P_s}{P_c.} = \frac{m}{99.}$$

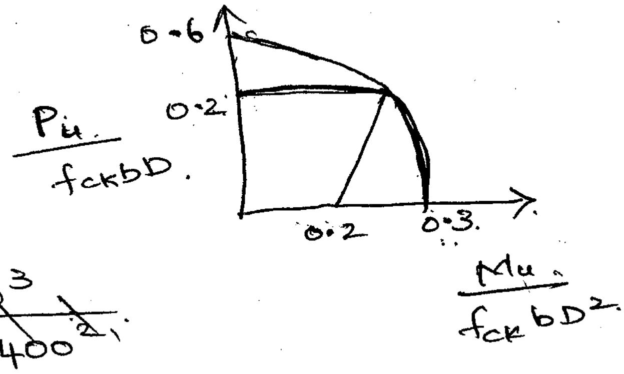
$$\frac{\sigma_{sc}}{\sigma_c.} = m$$

$$= \frac{10}{99.}$$

$$\boxed{\frac{P_s}{P_c} = 10\%}$$

Downloaded From : www.EasyEngineering.net

2. R.C.C column of square c/s 400mm size. has its load moment interaction diagram as shown in figure. what is the maximum uniaxial eccentricity if $P_u = 640\ kN$. $f_{ck} = 20\ N/mm^2$.



Solution:

$$\cancel{\frac{P_u}{f_{ck}bD} = \frac{640\times10^3}{20\times400^2}}$$

$$\frac{P_u}{f_{ck}bD} = \frac{640\times10^3}{20\times400^2} = 0.2.$$

$$\frac{M_u}{f_{ck}bD^2} = 0.3.$$

$$M_u = 0.3\times20\times400^3.$$

$$M_u = 384\ kN.m.$$

$$e = \frac{M_u}{P_u} = \frac{384}{640} = 0.6$$

$$\boxed{e = 0.6m}$$

3. Find Asc. for circle column. $\phi = 400mm$. $P_u = 1500kN$. total length 3.2mm. Adopt M25. and Fe415. Use helical tie bar.

Solution:

$$\frac{l}{D} = \frac{3200}{400} = 8 < 12mm.$$

It is a short column.

$$e_{min} = \frac{l}{500} + \frac{D}{30}$$

$$= \frac{3200}{500} + \frac{400}{30}$$

$$e_{min} = 19.73 \leq 20mm.$$

$$e_{min} = 20mm.$$

$$0.05D = 0.05 \times 400 = 20mm. = e_{min}$$

Hence design as axial column.

$$P_u = 1.05[0.4 f_{ck} A_c + 0.67 f_y A_{sc}].$$

$$= 1.05\left[0.4 \times 25 \times \left(\frac{\pi}{4} \times 400^2 - A_{sc}\right) + 0.67 \times 415 \times A_{sc}\right]$$

$$1500 \times 10^3 = 1.05 [1.256 \times 10^6 - 10 A_{sc} + 278.05 A_{sc}]$$

$$184500 = 268.05 A_{sc} \times 1.05$$

$$A_{sc} = 613.95 m$$

$$\boxed{A_{sc} = 641 \, mm^2}$$

$$\text{Min } A_{st} = 0.8\% \, A_g$$

$$= 0.8 \times \frac{\pi \times 400^2}{4}$$

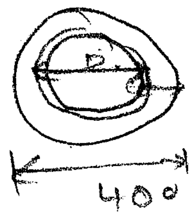
$$= 1005.3 \, mm^2.$$

~~ϕccep~~

Hence . provide .

$$A_{sc} = 1005\,mm^2.$$

ϕ of bar :



$$6 \times \frac{\pi \times d^2}{4} = 1005.$$

$$d = \sqrt{\frac{1005 \times 4}{6 \times \pi}}.$$

$$\boxed{d = 16\,mm.}$$

Lateral ties:

1.) $\frac{\phi\,\text{main bar}}{4} = \frac{16}{4} = 4\,mm$

2.) 6mm.

Provide 8mm ϕ bar.

Pitch.

$$\frac{\text{Volume of helical reinforcement}}{\text{Volume of core.}} \geq 0.36\left(\frac{A_g}{A_{core}} - 1\right)\frac{f_{ck}}{f_y}.$$

* $3\,\phi_{helix} = 3 \times 6\,mm = 18\,mm.$

* 25 mm .

* ~~75mm~~. ~~Volume~~

$$\boxed{\text{Dia of core} = \text{outer to outer dia of helix.}}$$

$$D_{core} = 400 - 32 - 32.$$

$$\boxed{D_{core} = 336\,mm.}$$

Volume of core $= A_{core} \times p.$

$$= \frac{\pi}{4} \times 336^2 \times p.$$

$$= 88.67 \times 10^3 p.$$

Volume of helix. $= \frac{\pi}{4} \times 8^2 \times l_{tie}.$

$$\boxed{l_{tie} = \pi\left[D_{core} - \frac{\phi_{helix}}{2} - \frac{\phi_{helix}}{2}\right]}$$

$$= \frac{\pi}{4} \times 8^2 \times 328 \times \pi$$

$$= 51.79 \times 10^3.$$

$$\frac{\text{Volume of helix}}{\text{Volume of core}} \geq 0.36\left(\frac{A_g}{A_{core}} - 1\right]\frac{f_{ck}}{f_y}.$$

$$\geq 0.36\left(\frac{125.66 \times 10^3}{88.67 \times 10^3} - 1\right)\frac{25}{415}.$$

$$\frac{51.79 \times 10^3}{88.67 \times 10^3 p.} \geq 9.048 \times 10^{-3}.$$

$$p \leq 64.55\,mm.$$

p
(i) $> 25\,mm.$
(ii) $< 75\,mm.$
(iii) $\geq 3 \times 8 = 24\,mm$

(iv) $\geq \frac{1}{6} \times D_{core} = \frac{1}{6} \times 336 = 56\,mm.$

Hence provide 8mm ϕ @ 56mm c/c spacing.

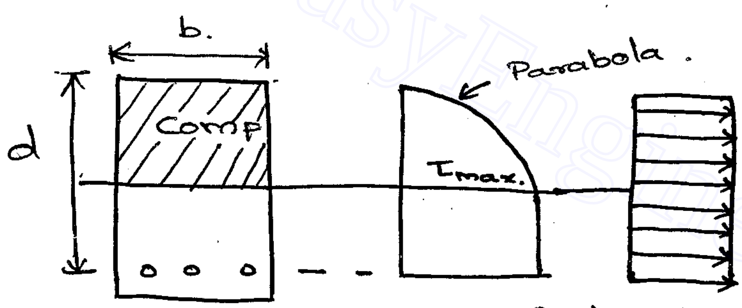
3/1/2015

R.C.C.

SHEAR BOND TORSION

* The stress distribution in R.C.C based on elastic theory (conventional Theory) is parabolic having values @ zero @ max. compressive fibre and max. shear stress @ neutral axis and then it becomes rectangle from N.A to centre of tension steel

* According to IS 456 : 2000 the shear stress distrubition is assumed as uniform (rectangle) having value as given below.



Conventional method. IS 456 2000.

* $$\boxed{\tau_{max} = \frac{3}{2}\tau_{avg}}$$

* $\tau_v = \dfrac{V_u}{bd}$ (for rectangluar beam).

$$\tau_v = \frac{V_u \pm \frac{M_u}{d}\tan\beta}{bd.}$$ [for beam of varying depth).

$$\tau_v = \frac{V_u - \frac{M_u}{d}\tan\beta}{bd.}$$

$$\tau_v = V_u + \frac{M_u}{d}\tan\beta.$$

* Shear resistance ~~without~~ by concrete without stirrups:

(i) compressive force. – 20–40% of total shear force

(ii) Interlocking of aggregate – 33–50%.

(iii) Dowel Action – 15–20%.

$$\tau_{cmax} = 0.85 \times 0.75 \sqrt{f_{ck}} = 0.637\sqrt{f_{ck}}.$$

(Table 20)

Grade of concrete.	M_{15}	M_{20}	M_{25}	M_{30}	M_{35}	M_{40}
τ_{cmax} (LSM).	2.5	2.8	3.1	3.5	3.7	4.0
τ_{cmax} WSM.	1.67	1.87	2.06	2.33	2.47	2.67

* τ_c depends upon. f_{ck} and P_t

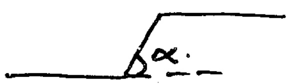
* $\tau_v = \tau_c$. No design but nominal stirrups is provided.

* $\tau_v < \tau_c$ "

* $\tau_v > \tau_c$. Shear reinforcement is required.

* Shear reinforcement Types.
  - Bent-up bars (for S.S 50% of bars are bent up).
  - Vertical (or) Inclined stirrups

* $$\boxed{V_{bent\,up\ bars} = 0.87 f_y A_{sv} \times \sin\alpha}$$



* shear resistance by combined bent up & stirrups.

$$V_{shear\ resistance} = (\tau_v - \tau_c)\, bd.$$

* Max shear resistance by bentup bar alone.

$$\boxed{0.87 f_y A_{sv} \sin\alpha \le 50\% \, (\tau_v - \tau_c) bd}$$

* Shear resistance by stirrups.

$$V_{stirrups} = V_{\substack{shear \\ resistance}} - V_{bentup}$$

* ϕ of stirrups.

6mm, 8, 10, $\underbrace{12 \text{ \& } 16\text{mm}}$ → Heavy torsion.

Theoretically.

* No. of legs.

2, 4, 6, 8, 10, 12.

used for structural element such as beam, column.

* Area of stirrups.

$$A_{sv} = \frac{\pi}{4} \times (dia)^2 \times \text{No. of. legs}.$$

Clause 40.4 pg: 70

* Shear reinforcement shall be provided to carry a shear force equal to.

$$V_{us} = V_u - \tau_c bd.$$

The strength of shear reinforcement V_{us} shall be calculated as below.

$$V_{us} = \frac{0.87 f_y . A_{sv} d}{S_v} \quad \text{(vertical stirrups)}$$

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v} (\sin\alpha + \cos\alpha) \quad \text{(Inclined Stirrups)}$$

* For single bars and (or) single group of parallel bars all bent up @ same c/s.

$$V_{us} = 0.87 f_y A_{sv} . \sin\alpha.$$

where S_v - spacing of stirrups (or) bent up bars. ~~only~~ along ~~the~~. length of member.

f_y shall not be greated than 415 MPa.

~~angle~~ α - angle between the Inclined. stirrups ~~and~~ (or) bent up bar. and axis of the rmebe~~r~~.

$\boxed{\alpha \not< 45^\circ}$.

* The complete development lengths and ~~ac~~ anchorage shall be demined to have been provided when the bar is bent through an angle of atleast 90° round, a bar of atleast its own diameter and its continued beyond the end of the curve of length for atleast 8ϕ.

* when the bar is bent through an angle of 135° and beyond the end of the curve for a length of atleast 6ϕ.

* when the bar is bent through an angle of 180° and it is continous~~ed~~ ~~as~~ beyond the end of the curve for a length of atleast 4ϕ.

\+ check for maximum spacing of stirrups.

$S_{v\,max} \not> 0.75\,d$. (normal vertical stirrups)

$\not> d$. (Inclined stirrups)

$\not> 300$ mm.

$\not> \dfrac{0.87 f_y A_{sv}}{0.4 b}$.

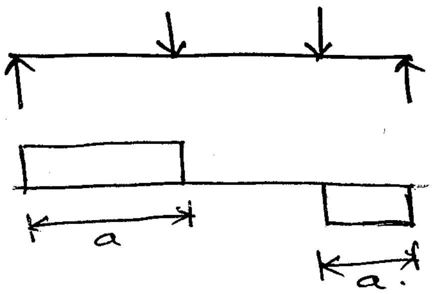
$\not>$ calculated. $\longrightarrow S_v = \dfrac{0.87 f_y A_{sv} d}{V_{us}}$

* The. min. percentage of shear reinforcement is ~~d for inclined stirrups~~.

$$\boxed{100 \times \frac{A_{sv}}{bd} = \frac{\cancel{0.87 f_y}}{0.4b} \cdot \frac{0.4}{0.87 f_y} \times 100.}$$

Grade	Strain
Fe 215	0.184%
Fe 415	0.11%
Fe 500	~~0.09%~~

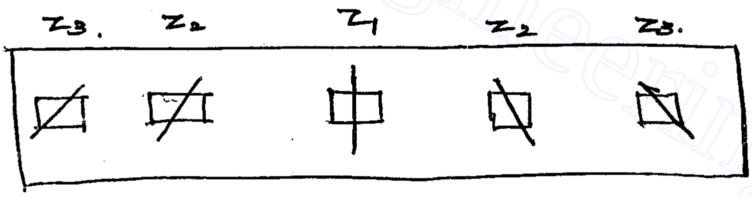
*

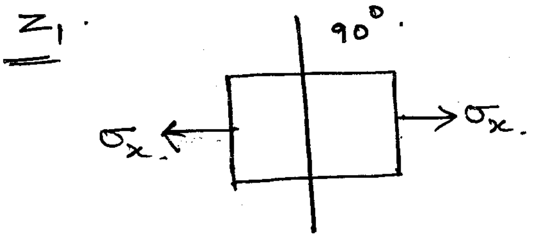


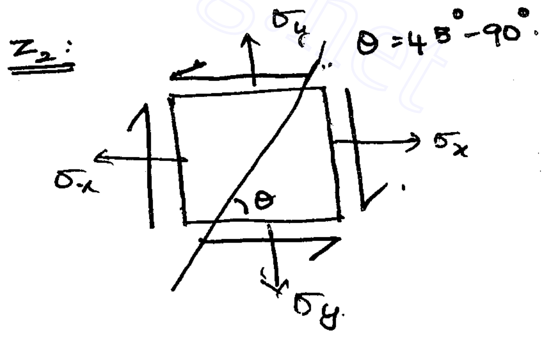
$$\boxed{\text{Shear span, } a = \frac{M}{V}}$$

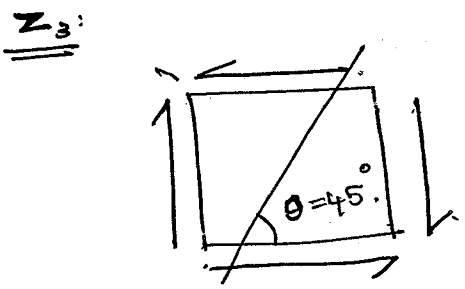
* The shear failure pattern is dependent on $\frac{a}{d}$ ratio.

$$\frac{a}{d} = \frac{\text{Shear span}}{\text{Effective depth}}.$$









zone 1	Flexural zone $\sigma_1 = \sigma_x$.	Flexural Failure.
zone 2.	Combined zone $\sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2}\sqrt{\sigma_x^2 + 4\tau^2}$	combined Failure.
Zone 3.	Daigonal Tension Crack zone (or) Pure shear. failure zone. where $\sigma_1 = \tau$.	~~where~~ shear failure

* If $\frac{a}{d} < 1$ (deep beam) – Tension or compression failure.

In this case internal tied arch developes.

* If $\frac{a}{d}$ b/w 1 to 2.5 – shear tension (or) shear compression crack.

* If $\frac{a}{d}$ b/w 2.5 to 6 – Diagonal tension crack.

* If $\frac{a}{d} > 6$. – Flexural crack.

1. A beam of rectanglura section of 230 x 400 (effective) is subjected to max shear force 120 kN. Use M_{20} and Fe415 and Fe 250. The design shear strength (τ_c) = 0.48 MPa.

(a) Spacing of 2 legged 8 mm strippus.

(b) In addition beam is subjected to a torque of 10.9 kN.m Determine the shear Force caused by the stirrups.

Solution:

$$S_v = \frac{0.87 f_y A_{sv} \times d}{V_{us}}$$

$$= \frac{0.87 \times 250 \times \frac{\pi}{4} \times 8^2 \times 2 \times 400}{[V_c - (\tau_c bd)]}$$

$$= \frac{8.746 \times 10^6}{[120 \times 10^3 - (0.48 \times 230 \times 400)]}$$

$$\boxed{S_v = 115.32\text{ mm}}$$

Check:

$S_v \not> 0.75d = 0.75 \times 400 = 300\text{mm}$

$\not> 300\text{mm}$.

$$S_v \not> \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 250 \times 2 \times \frac{\pi}{4} \times 8^2}{0.4 \times 230}$$

$= 237.67\text{ mm}$

$$\boxed{S_v = 100\text{ mm}}$$

$$V_s = V_u - \tau_c bd.$$

since torsion is acting

$$V_u = V_e.$$

$$V_s = V_e - \tau_c bd$$

$$= \left(V_u + \frac{1.6 T_u}{b}\right) - \tau_c bd.$$

$$= \left(120\times10^3 + \frac{1.6\times10.9\times10^6}{230}\right) - \left(0.48 \times 230 \times 400\right)$$

$$\boxed{V_s = 151.67 \text{ kN.}}$$

2\. A R.C.C beam is fixed @ one end and free @ other end it is subjected to. $V_u = 300kN$. and $l = 3m$.

Reinforcement - 6 # 20 mm ϕ.

c/s @ Fixed → 300 × 500.

c/s @ free → 300 × 300.

Determine spacing of 2 legged 8 mm stirrups use M_{20} and Fe 415 of Effective Cover 40 mm.

$\tan\beta = \frac{200}{3000}$

$= 0.067$

$$\tau_v = \frac{V_u - \frac{M_u}{d}\tan\beta}{bd}$$

$$\tau_v = \frac{(300\times10^3) - \frac{450\times10^6}{460}\times 0.067}{300 \times 460}$$

$\boxed{\cancel{\tau_v = 2.2217 \ N/mm^2}}$

$\boxed{\tau_v = 1.70 \ N/mm^2}$

$\tau_c = 0.69$.

$$S_v = \frac{0.87\times415\times 2\times\frac{\pi}{4}\times 8^2 \times 460}{V_u - \tau_c bd}$$

$$= \frac{0.87\times4150\times2\times\frac{\pi}{4}\times8^2\times460}{(1.7-0.69)\,460\times380}$$

$S_v = 119.79$ mm

3. Consider a bar of $\phi = d$ embedded in large concrete block as shown in fig. with a pull out force "P" being applied. Let σ_b and σ_{st} be bond and tensile strength of the bar. If the block is held in position and it is assumed that material of the block does not fall which of the following option represents max. value of "P".

Max value of. P_s & P_b.

Ans: Max value of. $\frac{\pi}{4} D^2 \sigma_{st}$ & $\pi D L \sigma_b$.

4. Consider 2 beams P&Q each having 400 x 750 effective $\tau_{cmax} = 2.8$ MPa for the reinforcement provide and grade of concrete M_{20}. $\tau_c = 0.75$ MPa.

The design shear in beam P is 400 kN. Q - 750 kN. considering the IS 456 : 2000 which of the following statement is true.

P $\tau_v = \frac{400 \times 10^3}{400 \times 750} = 1.33$ $\tau_v > \tau_c$.

$\tau_v < \tau_{cmax}$.

$V_{us} = (\tau_v - \tau_c) bd$

$\boxed{V_{us} = 175 \text{ kN}}$

Q $\tau_v = \frac{750 \times 10^3}{400 \times 750} = 2.5$. $\tau_v > \tau_c > \tau_{cmax}$.

Hence the section must be revised.

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5. The ~~width~~ beam size 250 x 450 (effective) The beam is reinforced by 20mm TOR bars in TOR bar, $V_u = 150\,kN$. Check the requirement of shear reinforcement and provide if required. use M_{20}, Fe 415. use 8mm ϕ stirrups.

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}.$$

$$\frac{A_{sv}}{S_v} \geq \frac{0.4\,b}{f_y}.$$

Above conditioned must be satisfied.

% of steel.	1	1.25	1.5.
τ_c N/mm^2	0.62	0.67	0.72

TORSION. PRINCIPLE STRESSES AND THEORIES OF FAILURE.

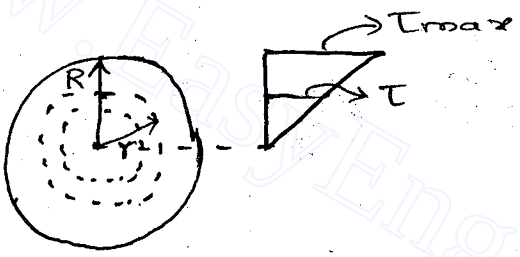
* Torque:

Torque is force x radius $T = F \times r$ and the phenomenon is called torsion which is B.M about its own axis or Z-Z axis.

Unit - N.m (or) Joule. $\frac{kg\,m}{s^2} \cdot$ $kg\,m^2 s^{-2}$

Dimension - ML^2T^{-2} ML^2T^{-2}

* Torsional Equation:



By similar Δ^{le} property.

$$\frac{\tau_{max}}{\tau} = \frac{R}{r}$$

$$\boxed{\tau = \frac{\tau_{max}}{R} \times r}$$

$$T = F \times r.$$

$$dT = dF \times r.$$

$$dT = \tau \times dA \times r.$$

$$= \tau \times 2\pi r\, dr \times r.$$

$$= \frac{\tau_{max}}{R} \times r \times 2\pi r^2 \times dr.$$

$$dT = 2\pi \frac{\tau_{max}}{R} \times r^3 dr.$$

$$\int dT = 2\pi \frac{\tau_{max}}{R} \int_0^R r^3 dr.$$

$$T = 2\pi \frac{\tau_{max}}{R} \left(\frac{r^4}{4}\right)_0^R.$$

$$= \frac{2\pi}{4} \frac{\tau_{max}}{R} R^4$$

$$= \frac{\pi}{2} \frac{\tau_{max}}{R} \left(\frac{D}{2}\right)^4.$$

$$= \frac{\pi}{2} \frac{\tau_{max}}{R} \left(\frac{D^4}{16}\right)$$

22/12/2015

Torsion.

* Torque:
  - → It is the B.M about its own axis (or) z-z axis
  - → The pheonomenon is called Torsion
  - → ~~Torque~~ moment T = Force × radius. ~~twodo~~

* Torsional Equation:

$$\frac{T}{J} = \frac{\tau}{R} = \frac{G\theta}{l}.$$

$$\boxed{dT = dF \times r.} \qquad \boxed{\phi = R\theta/l}$$

$$\boxed{G = \frac{\tau}{\phi}}$$

* Polar moment of Inertia

$J = I_p = I_{xx} + I_{yy}$.

$J = \frac{\pi D^4}{32}$ for solid shaft.

$J = \frac{\pi (D^4 - d^4)}{32}$ for hollow shaft

$J = \frac{\pi D^3 t}{4}$ for thin circular shaft

* Strain Energy Due to Torsion:

$U = \frac{\tau^2}{2G} \times \text{volume}$.

$U = \frac{\tau_{max}^2}{4G} \times \text{volume}$.

$U = \frac{T^2 \times l}{2GJ}$.

* Equivalent Torque and Equivalent Bending Moment.

$T = P\bar{x}$ $\qquad M = P\bar{y}$. $\qquad \sigma_{1,2} = \frac{16}{\pi D^3}\left[M \pm \sqrt{M^2+T^2}\right]$

$M_{eq} = \frac{1}{2}\left[M + \sqrt{M^2+T^2}\right]$ $\qquad \sigma_1 = \frac{32 M_{eq}}{\pi D^3}$

$T_{eq} = \left[\sqrt{M^2+T^2}\right]$. $\qquad \tau_{max} = \frac{16}{\pi D^3}\left[\sqrt{M^2+T^2}\right]$

$= \frac{\sigma_1 - \sigma_2}{2}$.

$$T = \frac{\tau_{max}}{R} \times \left(\frac{\pi D^4}{32}\right).$$

$$= \frac{\tau_{max}}{R} \times J.$$

$$\boxed{\frac{T}{J} = \frac{\tau_{max}}{R.}}$$ Torsional equation

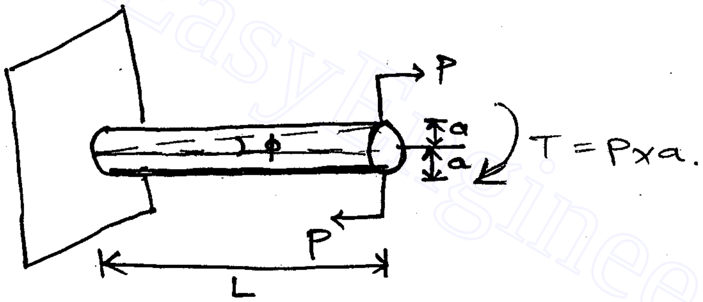
* Polar moment of inertia (I_p (or) J).

For solid circular shaft.

$$\boxed{J = \frac{\pi}{32} D^4}.$$

For thin circular shaft.

$$\boxed{J = \frac{\pi D^3 t}{4}}$$



$$\tan\phi = \frac{R\theta}{l}.$$

$$\boxed{\phi = \frac{R\theta}{l}}.$$

$\boxed{\theta - \text{in radians.}}$

ϕ = shear strain.

$$\theta = \frac{\text{Arc length.}}{\text{Radius.}}$$

$$\theta = \frac{\hat{l}}{R.}$$

$$\boxed{l = R\theta.}$$

$$G = \frac{\tau}{\phi.}$$

$$G = \frac{\tau}{\frac{R\theta.}{l.}}.$$

$$\boxed{\frac{\tau}{R} = \frac{G\theta}{l.}}$$

$$\boxed{\frac{T}{J} = \frac{\tau}{R} = \frac{G\theta}{l}.}$$ Torsional Equation.

* Expression Of Strain Energy Due to Torsion.

$$U = \frac{\tau^2}{2G} \times \text{volume.}$$

$$\boxed{\tau = \frac{\tau_{max}}{R} \times r}$$

$$du = \frac{\tau^2}{2G} \times dV.$$

$$= \frac{\left(\frac{\tau_{max} \times r}{R}\right)^2}{2G} \times l \times dA.$$

$$= \frac{\tau_{max}^2 \times r^2}{2R^2 G} \times l \times 2\pi\, r\, dr.$$

$$du = \frac{\tau_{max}^2}{2R^2 G} \times 2\pi l \times r^3 dr.$$

$$\int du = \frac{\tau_{max}^2}{\cancel{2}R^2 G} \cancel{2}\pi l \times \int_0^R r^3 dr.$$

$$U = \frac{\tau_{max}^2}{R^2 G} \times \pi l \times \left(\frac{r^4}{4}\right)_0^R$$

$$= \frac{\tau_{max}^2}{\cancel{R^2} G} \times \pi l \times \frac{R^4}{4}.$$

$$= \frac{\tau_{max}^2}{4G} \times \pi R^2 l.$$

$$\boxed{U = \frac{\tau_{max}^2}{4G} \times \text{volume}}$$

$$\tau_{max} = \frac{T}{J} R$$

$$U = \frac{\left(\frac{TR}{J}\right)^2 \times A \times l.}{4G.}$$

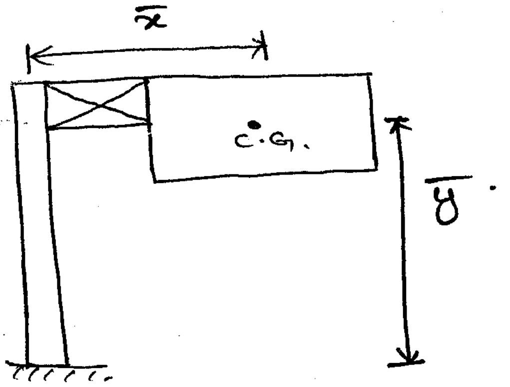
$$= \frac{T^2 \times \frac{D^2}{4}}{\left(\frac{\pi D^4}{32}\right)^2 \times 4G} \times \frac{\pi \times D^2}{4} \times l.$$

$$= \frac{T^2 \times \cancel{8}\ \pi\left(\frac{\cancel{D^4}}{\cancel{32}}\right) \times l.}{\left(\frac{\pi D^4}{32}\right)^{\cancel{2}} \times 2G.}$$

$$\boxed{U = \frac{T^2 \times l}{2GJ}}$$

(4)

* Expression for Equivalent Torque and Equivalent B.M when ~~the~~ it is Subjected to combined Torque and B.M.



$$\boxed{\begin{aligned} T &= P \times \bar{x} \\ M &= P \times \bar{y} \end{aligned}}$$

$$\boxed{P = P_\omega \times A}$$

$$\frac{\tau}{R} = \frac{T}{J}$$

$$\tau = \frac{T \times R}{J}$$

$$= \frac{T \times D/2}{\left(\frac{\pi D^4}{32}\right)}$$

$$\boxed{\tau = \frac{16T}{\pi D^3}}$$

$$\frac{M}{I} = \frac{f}{y}$$

$$\frac{M \times y}{I} = \sigma_x$$

$$\sigma_x = \frac{M}{\left(\frac{\pi D^4}{64}\right)} \times \frac{D}{2}$$

$$\boxed{\sigma_x = \frac{32M}{\pi D^3}}$$

$$\sigma_{1,2} = \frac{\sigma_x}{2} \pm \frac{1}{2}\sqrt{\sigma_x^2 + 4\tau^2}$$

$$= \frac{\left(\frac{32M}{\pi D^3}\right)}{2} \pm \frac{1}{2}\sqrt{\left(\frac{32M}{\pi D^3}\right)^2 + 4\left(\frac{16T}{\pi D^3}\right)^2}$$

$$= \frac{\left(\frac{32M}{\pi D^3}\right)}{2} \pm \frac{1}{2}\sqrt{\left(\frac{32M}{\pi D^3}\right)^2 + \left(\frac{32T}{\pi D^3}\right)^2}$$

$$= \frac{\frac{32M}{\pi D^3}}{2} \pm \frac{\frac{32}{\pi D^3}}{2}\sqrt{M^2 + T^2}$$

$$= \frac{32}{\pi D^3} \times \frac{1}{2}\left[M \pm \sqrt{M^2 + T^2}\right]$$

$$\sigma_{1,2} = \frac{16}{\pi D^3}\left[M \pm \sqrt{M^2 + T^2}\right]$$

For Resultant Bending.

$$\frac{M_{equ}}{I} = \frac{f}{y}$$

$$\frac{M_{eq}}{\left(\frac{\pi \times D^4}{64}\right)} = \frac{\sigma_1}{(D/2)}$$

$$\sigma_1 = \frac{M_{eq}}{\frac{\pi D^3}{32}}$$

$$\boxed{\sigma_1 = \frac{32 M_{eq}}{\pi D^3}}$$

$$\frac{16}{\pi D^3}\left(M + \sqrt{M^2+T^2}\right) = \frac{32 M_{eq}}{\pi D^3}$$

$$\boxed{M_{eq} = \frac{1}{2}\left(M + \sqrt{M^2+T^2}\right)}$$

$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2}$$

$$= \frac{1}{2} \times \frac{16}{\pi D^3} \times \left(M + \sqrt{M^2+T^2} - M + \sqrt{M^2+T^2}\right)$$

$$= \frac{1}{2} \times \frac{16}{\pi D^3} \times 2\left(\sqrt{M^2+T^2}\right)$$

$$\boxed{\tau_{max} = \frac{16}{\pi D^3}\sqrt{(M^2+T^2)}}$$

Resultant Torsion,

$$\frac{\tau_{max}}{R} = \frac{T_{eq}}{J}$$

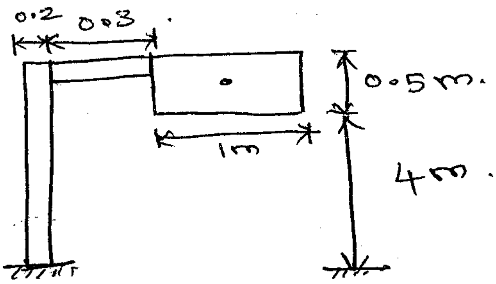
$$\frac{\frac{16}{\pi D^3}\sqrt{(M^2+T^2)}}{\frac{D}{2}} = \frac{T_{eq}}{\frac{\pi D^4}{32}}$$

$$\frac{32}{\pi D^4}\sqrt{M^2+T^2} = \frac{32}{\pi D^4} T_{eq}$$

$$\boxed{T_{eq} = \sqrt{M^2+T^2}}$$

(6)

1. A vertical post $D_o = 200mm$. $D_i = 100mm$. supports a sign board of $1m \times 0.5m$ subjected wind pressure 1.5KPa. board is connected to the post ~~30c~~ by 30cm long angles. the under side of board is 4m above G.L. Determine M_{eq}, T_{eq}, σ_1, τ_{max}, Bending stress due to ~~Mending~~ B.M only, τ due to Torsion only.



$$\bar{x} = \frac{0.2}{2} + 0.3 + \frac{1}{2}$$

$$\boxed{\bar{x} = 0.9m}$$

$$\bar{y} = \frac{0.5}{2} + 4$$

$$\boxed{\bar{y} = 4.25m}$$

$$P = 1.5 \times 1 \times 0.5$$

$$\boxed{P = 0.75 \text{ kN}}$$

$$M = P \times \bar{y}$$

$$= 0.75 \times 4.25.$$

$$\boxed{M = 3.1875 \text{ kN m.}}$$

$$T = P \times \bar{x}.$$

$$= 0.75 \times 0.9.$$

$$\boxed{T = 0.675 \text{ kN.m}}$$

$$M_{eq} = \frac{1}{2}\left[M + \sqrt{M^2+T^2}\right]$$

$$= \frac{1}{2}\left[3.1875 + \sqrt{(3.1875)^2 + (0.675)^2}\right.$$

$$\boxed{M_{eq} = 3.22 \text{ kN.m}}$$

$$T_{eq} = \sqrt{M^2+T^2}.$$

$$= \sqrt{(3.1875)^2 + (0.675)^2}$$

$$\boxed{T_{eq} = 3.26 \text{ kN.m}}$$

$$\sigma_1 = \frac{32\, M_{eq}}{\pi D^3}.$$

$\sigma_1 = $ ~~16~~

$$\sigma_x = \frac{M}{I} \times y$$

$$= \frac{3.1875 \times \cancel{\frac{200}{2}}}{\cancel{\frac{\pi}{64}(200 - 100)^4}}$$

$$= \frac{3.1875 \times \left(\frac{0.2}{2}\right)}{\frac{\pi}{64}(0.2^4 - 0.1^4)}$$

$\sigma_x = \cancel{64.93}\ \cancel{N/mm^2}$ $\boxed{\sigma_x = 4.33 N/mm^2}$

$$\tau = \frac{T}{J} \times R$$

$$= \frac{0.675 \times 10^6 \times \frac{200}{2}}{\frac{\pi}{32}(200^4 - 100^4)}$$

~~$\tau = 6.88\ N/mm^2$~~ $\boxed{\tau = 0.46\ N/mm^2}$

$$\sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2}\sqrt{\sigma_x^2 + 4\tau^2}$$

$$= \frac{4.33}{2} + \frac{1}{2}\sqrt{(4.33)^2 + 4(0.46)^2}$$

$$= 2.165 + 2.213$$

$= \cancel{65.65}$

$$\boxed{\sigma_1 = 4.38\ N/mm^2}$$

$$\sigma_2 = 2.165 - 2.213$$

$$\sigma_2 = -0.02$$

$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2} = \frac{4.38 + 0.02}{2}$$

$$\boxed{\tau_{max} = 2.202\ N/mm^2}$$

2.) A solid shaft is subjected to torque 4.5 kNm $\theta = 1^\circ$. $l = 20$ dia

$\tau_{max} = 80$ MPa. Take $G = 0.8 \times 10^5$ N/mm²

Determine the dia.

Solution:

$$\frac{T}{J} = \frac{\tau}{R}$$

$$\frac{4.5 \times 10^6}{\frac{\pi}{32} \times D^4} = \frac{80}{D/2}$$

$$D^3 = \frac{4.5 \times 10^6}{\frac{\pi}{16} \times 80}$$

$$\boxed{D = 65.92 \text{ mm}}$$

$$\frac{T}{J} = \frac{G\theta}{l}$$

$$\frac{4.5 \times 10^6}{\frac{\pi}{32} \times D^4} = \frac{0.8 \times 10^5 \times \frac{1 \times \pi}{180}}{20 \times D}$$

$$D^3 = \frac{4.5 \times 10^6 \times 20}{0.8 \times 10^5 \times \frac{\pi}{180} \times \frac{\pi}{32}}$$

$$\boxed{D = 86.91 \text{ mm}}$$

Adopt the greater value.

$$\boxed{D = 87 \text{ mm}}$$

* Power :

$$P = \frac{2\pi N T}{60}$$ P - Watts

T - kN·m.

% of saving material $= \frac{W_S - W_H}{W_S} \times 100$.

$= \frac{D_S^2 - (D_o^2 - D_i^2)}{D_S^2} \times 100$.

% of wastage of material

$= \frac{W_H - W_S}{W_H} \times 100$.

* Hoop stress, longitudinal stress

Hoop stress $(\sigma_x) = \frac{Pd}{2t}$.

Longitudinal stress $(\sigma_y) = \frac{Pd}{4t}$

Absolute Max stress $= \frac{\sigma_1}{2}$

* If the Torque is acting @ the middle of the bar then $\theta_{net} = 0$.

$\theta_1 = \theta_2$.

* If two Torque is acting in opposite direction in two different bars then θ_{net}

$\theta_{net} = \theta_1 - \theta_2$.

39) A solid shaft has to transfer a mean power of 1000 kW @ 300 r.p.m. τ_{max} = 60 MPa. determine dia of shaft. If T_{max} = 25% more than T_{mean}. Also Determine % saving in material, if hollow shaft of inner dia 0.6 D_o is used

Solution:

$$P = \frac{2\pi N T}{60}.$$

$$1000 \times 10^3 = \frac{2\pi N T}{60}.$$

$$\boxed{T_{mean} = 31.83 \text{ kN·m}}.$$

$$T_{max} = 1.25 \times 31.83.$$

$$\boxed{T_{max} = 39.78 \text{ kN·m}.}$$

$$\frac{T}{J} = \frac{\tau_{max}}{R}.$$

$$\frac{39.78 \times 10^6}{\frac{\pi}{32} \times D^4} = \frac{60}{D/2}.$$

$$D^3 = \frac{39.78 \times 10^6}{\left(\frac{\pi}{16} \times 60\right)}.$$

$$\boxed{D = 150 \text{mm}}$$

For hollow shaft.

$$\frac{39.78 \times 10^6}{\frac{\pi}{32} \times (D^4 - (0.6D)^4)} = \frac{60}{D/2}.$$

$$\frac{39.78 \times 10^6}{\frac{\pi}{32} \times 0.8704\ D^4} = \frac{60 \times 2}{D}.$$

$$D^3 = 3.879 \times 10^6.$$

$$\boxed{D = 157 \text{mm}.}$$

~~Mat~~

$$\%\text{ of saving material} = \frac{W_S - W_H}{W_S} \times 100.$$

$$= \frac{D_s^2 - (D_o^2 - D_i^2)}{D_s^2} \times 100$$

$$= \frac{150^2 - (157^2 - (0.6 \times 157)^2)}{150^2} \times 100.$$

$$\boxed{\%\text{ of saving of material} = 29.89\,\%.}$$

$$\left.\begin{array}{l}\%\text{ of wastage of}\\ \text{material}\end{array}\right\} = \frac{W_H - W_S}{W_H} \times 100.$$

1. A thin cylinder $d = 300mm$ $t = 10mm$ $T = 100 N\cdot m$. inner fluid pressure $= 1 MPa$. Determine shear stress due to torque. Max. principle stress, absolute maximum stress.

$$R = \frac{300}{2} = 150mm$$

$$\frac{T}{J} = \frac{\tau}{R}.$$

$$J = \frac{\pi}{4} d^3 t$$

$$\tau = \frac{T \times R}{J}.$$

$$= \frac{100 \times 150 \times 10^3}{\frac{\pi}{4} \times 300^3 \times 10}.$$

$$\boxed{\tau = 0.07 N/mm^2}$$

Hoop stress $= \frac{Pd}{2t}$.

$$\sigma_x = \frac{1 \times 300}{2 \times 10}.$$

$$\sigma_x = 15 N/mm^2$$

Longitudinal stress $= \frac{Pd}{4t}$

$$= \frac{1 \times 300}{4 \times 10}.$$

$$= 7.5 N/mm^2.$$

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \frac{1}{2} \sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2}$$

$$= \left(\frac{15 + 7.5}{2}\right) + \frac{1}{2} \sqrt{(15 - 7.5)^2 + 4(0.07)^2}$$

$$\boxed{\sigma_1 = 15 N/mm^2}$$

$$\sigma_2 = \left(\frac{\sigma_x + \sigma_y}{2}\right) - \frac{1}{2} \sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2}.$$

$$\boxed{\sigma_2 = 7.5 N/mm^2}$$

$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2}$$

$$= \frac{15 - 7.5}{2}$$

$$\boxed{\tau_{max} = 3.75 \ N/mm^2}$$

$$\left.\begin{matrix}\text{Absolute max.}\\ \text{stress}\end{matrix}\right\} = \frac{\sigma_1}{2}.$$

$$= \frac{15}{2}.$$

$$= 7.5 \ N/mm^2.$$

2. Simply supported beam span = 4 m subjected to central point load 2 kN. $T = 1.5$ kN·m $\phi = 100$ mm. Find. M_{eq}, σ_1, T_{eq}, τ_{max},

Solution:

$$M = \frac{wl}{4} = \frac{2 \times 4}{4} = 2 \text{ kN·m}.$$

$$M_{eq} = \frac{1}{2}\left[M + \sqrt{M^2 + T^2}\right]$$

$$= \frac{1}{2}\left[2 + \sqrt{2^2 + 1.5^2}\right].$$

$$\boxed{M_{eq} = 2.25 \text{ kN·m}.}$$

$$T_{eq} = \sqrt{M^2 + T^2}$$

$$= \sqrt{2^2 + 1.5^2}.$$

$$\boxed{T_{eq} = 2.5 \text{ kN·m}.}$$

$$\frac{M_{eq}}{I} = \frac{\sigma_{max}}{y}. \qquad y = \frac{D}{2}$$

$$\sigma_{max} = \frac{M_{eq} \times y}{I}$$

$$= \frac{2.25 \times 10^6 \times 50}{\frac{\pi}{64} \times 100^4}.$$

$$\boxed{\sigma_{max} = 22.92 \text{ N/mm}^2}$$

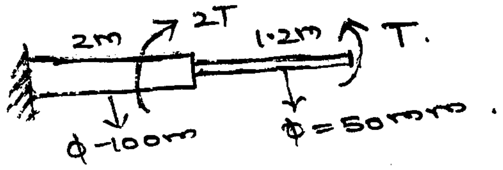
$$\frac{T_{eq}}{J} = \frac{\tau_{max}}{R}.$$

$$\tau_{max} = \frac{T_{eq} \times R}{J}.$$

$$= \frac{2.5 \times 10^6 \times 50}{\frac{\pi}{32} \times 100^4}.$$

$$\boxed{\tau_{max} = 12.73 \text{ N/mm}^2}$$

3. A shaft is shown in figure below determine value of T if τ_{max} = & shear stress must not exceed 80 MPa. $G = 0.8 \times 10^5$ MPa. Determine angle of twist also.



solution:

Hint : If Torque is safer for smaller dia then sure it will be safer for larger dia.

$$\frac{T}{J} = \frac{\tau_{max}}{R}.$$

$$T = \frac{80.}{\frac{50}{2}} \times \frac{\pi}{32} \times 50^4.$$

$$\boxed{T = 1.96 \times 10^6 \cdot N.mm}.$$

$$\frac{T}{J_1} = \frac{G\theta}{l}.$$

$$\theta_1 = \frac{1.96 \times 10^6 \times 1.200}{0.8 \times 10^5 \times \left(\frac{\pi \times 50^4}{32}\right)}$$

$$\theta_1 = 0.0479.$$

$$\theta_2 = \frac{1.96 \times 10^6 \times 2000}{0.8 \times 10^5 \times \left(\frac{\pi \times 100^4}{32}\right)}.$$

$$\theta_2 = 0.005.$$

$$\theta_{net} = \theta_1 - \theta_2.$$

$$= 0.0479 - 0.005.$$

$$\boxed{\theta_{net} = 0.043.}$$

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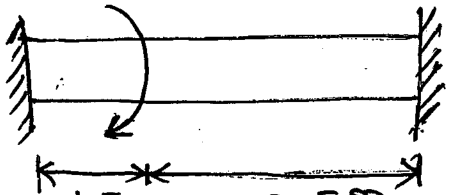
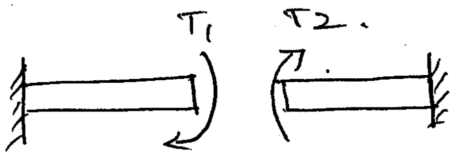
4. Solid shaft $\phi = 50mm$. $T = 1 kN \cdot m$ @ 1.5m from left support. Determine the Torque developed @ fixed end.

~~Max~~ τ_{max}, θ,





$$T = T_1 + T_2.$$

$$1 = T_1 + T_2.$$

$$1 = 1.67T_2 + T_2.$$

$$T_2 = \frac{1}{2.67}.$$

$$\boxed{T_2 = 0.375 \text{ kN·m}}$$

$$\theta_1 = \theta_2.$$

$$\frac{T_1 \times l_1}{GJ} = \frac{T_2 l_2}{GJ}$$

$$T_1 = T_2 \frac{l_2}{l_1}.$$

$$= \frac{2.5}{1.1} T_2$$

$$T_1 = 1.67 T_2.$$

$$T_1 = 1 - 0.375.$$

$$\boxed{T_1 = 0.625 \text{ kN·m}.}$$

$$\theta_1 = \frac{T_1 l_1}{GJ.} = \left(\frac{0.625 \times 10^6 \times 1500}{0.8 \times 10^5 \times \frac{\pi}{32} \times 50^4}\right)$$

$$\boxed{\theta_1 = 0.019.}$$

$$\frac{\tau_{max}}{R.} = \frac{T_{max}}{J.}$$

$$\tau_{max} = \frac{0.625 \times 10^6 \times \frac{50}{2}.}{\frac{\pi}{32} \times 50^4}$$

$$\boxed{\tau_{max} = 25.46 \text{ N/mm}^2}$$

27/12/2015.

STRENGTH OF MATERIAL

1. A hollow shaft with Inner dia ~~Inner dia~~ 60% of outer dia. is compared to the solid shaft of same material. and equal weight. Determine the rate of torque resistance. $T = \frac{\tau}{R} \times J$.

Solution:

$$\frac{W_H}{W_s} = \frac{\gamma \times \text{Volume of hollow shaft}}{\cancel{\gamma} \times \text{Volume of solid shaft}}$$

$$= \frac{\text{Area of hollow shaft} \times L}{\text{Area of solid shaft} \times L}$$

$$= \frac{\frac{\pi}{4}(D_H^2 - D_i^2)}{\frac{\pi}{4} D_o^2}$$

$$\frac{W_H}{W_s} = \frac{\cancel{\frac{\pi}{4}}\ 0.64 D_H^2}{\cancel{D_o^2}} = \frac{0.64 D_H^2}{D_o^2}$$

$$\boxed{D_o = 0.8 D_H.}$$

~~$W_H = 0.64 W_s$~~

$$\frac{T_H}{T_s} = \frac{\frac{\tau}{R_H} \times J}{\frac{\tau}{R_s} \times J}$$

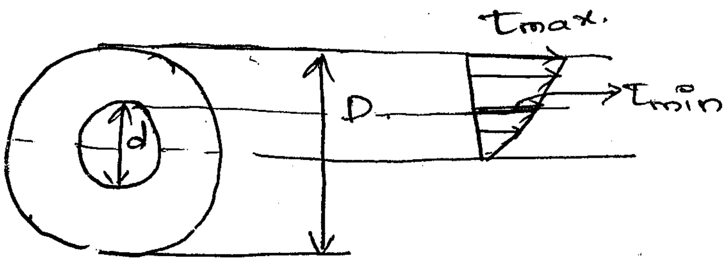
$$= \frac{D_S \times (D_H^4 - D_i^4)\pi/64}{D_H \times \frac{\pi}{64}(D_s^4)}$$

$$= \frac{D_S\ (0.8704\ D_H^4)}{0.8 D_H\ (0.8 D_H)^4}$$

$$= \frac{0.8\ D_H.\ (0.8704\ D_H^4)}{D_H.\ (0.8 D_H)^4}$$

$$\boxed{\frac{T_H}{T_s} = 1.7}$$

Isotropic State of Stress is independent & on frame of refrence (Refrence Axis)



Note : If

Note:

If 2 different shafts are connected in series then the torque will be same for both the to shafts and the angle of twist will not be same

If 2 shafts are in parallel then angle of twist will be same but torque will not be same. (i.e) the torque is shared by both the shaft

2. A steel shaft of dia 40 mm is coaxially fitted with hollow bronze shaft hollow shaft is fitted. The inner dia of hollow shaft equal to dia of steel shaft. The maximum permissible shear stress is 40 MPa for bronze and 60 MPa for steel. The total length of the shaft is 4 m. Determine outer dia of hollow shaft. Total torque applied on combined shaft. Angle of twist

$G_{steel} = 0.8 \times 10^5 \, N/mm^2$ $G_{bronze} = 0.4 \times 10^5 \, N/mm^2$

$$\theta_s = \theta_b \qquad \frac{\tau}{R} = \frac{G\theta}{l}$$

$$\left(\frac{\tau \times l}{R \times G}\right)_s = \left(\frac{\tau \times l}{R \times G}\right)_B \qquad \theta = \frac{\tau \times l}{R\,G}$$

$$\left(\frac{60 \times 4}{\frac{40}{2} \times 0.8 \times 10^5}\right) = \frac{40 \times 4}{\frac{D_b}{2} \times 0.4 \times 10^5}$$

$D_b = 53.33$ mm (outer dia)

$D_b = 40$ mm (Inner dia)

$$\frac{T_b}{J} = \frac{\tau}{R}.$$

$$T_b = \frac{40}{D_b/2} \times \frac{\pi}{32}\left(D_{bo}^4 - D_{bi}^4\right).$$

$$= \frac{40}{\left(\frac{53.33}{2}\right)} \times \frac{\pi}{32}\left(53.33^4 - 40^4\right)$$

$$\boxed{T_b = 814.24 \times 10^3 \ N.mm.}$$

$$T_s = \frac{\tau_s}{R} \times J.$$

$$= \frac{60}{(40/2)} \times \frac{\pi}{32} \times 40^4.$$

$$\boxed{T_s = 753.98 \times 10^3 \ N.mm}$$

$$\frac{T_s}{J} = \frac{G\theta}{l}.$$

$$\theta = \frac{753.98 \times 10^3}{\frac{\pi}{32}(40^4)} \times \frac{4000}{0.8 \times 10^5}$$

$$\boxed{\theta = 0.15.}$$

$$\frac{T_b}{J} = \frac{G\theta}{l}.$$

$$\theta = \frac{814.24 \times 10^3 \times 4000}{\frac{\pi}{32}(53.33^4 - 40^4) \times 0.4 \times 10^5}$$

$$\boxed{\theta = 0.15.}$$

Total torque $= T_1 + T_2$

$= 814.24 \times 10^3 + 753.98 \times 10^3$

$= 1568.22$ kN.mm.

28/12/2015

STRENGTH OF MATERIAL.

STRAIN ENERGY OF HOLLOW SHAFT.

$$U = \frac{T^2 l}{2GJ} \qquad T = \frac{\tau}{R} \times J.$$

$$= \frac{\frac{\tau^2 J^{\cancel{2}}}{R^2} \times l}{2G\cancel{J}}$$

$$= \frac{\tau^2 \cdot \frac{\pi}{32}(D^4 - d^4) \times l}{\left(\frac{D}{2}\right)^2 \times 2G}$$

$$= \frac{\tau^2 \frac{\pi}{4}(D^2 - d^2) \times \frac{1}{8}(D^2 + d^4) \times l}{\left(\frac{D}{2}\right)^2 \times 2G}$$

$$= \frac{\tau^2 \times A \times l \times \frac{1}{8}}{2G \times \frac{D}{4}} \frac{(D^2 + d^4)}{D^2}$$

$$\boxed{U = \frac{\tau^2}{4G} \times \text{volume} \left(\frac{D^2 + d^4}{D^2}\right)}$$

$$\boxed{\frac{U}{\text{Volume}} = \frac{\tau^2}{4G} \cdot \left(\frac{D^2 + d^4}{D^2}\right)}$$

1. Determine torsional energy per unit volume of a hollow shaft of $D_o = 100mm$ and $d_i = 60mm$. torsional stress 60MPa and $G = 84GPa$.

$$\frac{U}{V} = \frac{\tau^2}{4G} \times \frac{D_o^2 + d_i^2}{D^2}$$

Pa – N/m²

$$= \frac{(60)^2}{4 \times \frac{84 \times 10^9}{10^6}} \times \left(\frac{100^2 + 60^2}{100^2}\right) =$$

$$= 0.0146 \frac{N}{mm^2} \times \frac{mm}{mm}$$

$$= 0.0146 \frac{N.mm}{mm^3}$$

$$= 0.0146 \times 10^{-3} \frac{Joule}{mm^3}$$

$$\boxed{\frac{U}{V} = 0.0146 \times 10^6 \frac{Joule}{m^3}}$$

2. A shaft d = 75mm l = 1m is connected to another shaft of d = 50mm, of l = 2m. a power of 45 kW is given to 75mm dia shaft and power of 15 kW is taken away @ the junction of two shaft. Determine τ_{max} any where in the shaft and ~~total~~ angle of twist. Take revolution of shaft as 200 rpm.

$$P = \frac{2\pi NT}{60}$$

$$45 \times 10^3 = \frac{2 \times \pi \times 200 \times T_1}{60}$$

$$T_1 = 2148.59 \ N.m$$

$$\boxed{T_1 = 2148.59 \times 10^3 \ N.mm}$$

$$\frac{T_1}{J} = \frac{\tau}{R}$$

$$\tau = \frac{2148.59 \times 10^3}{\frac{\pi}{32} \times 75^4} \times \frac{75}{2}$$

$$\boxed{\tau = 25.96\ N/mm^2}$$

$$\frac{T}{J} = \frac{G\theta}{l}$$

$$\theta_1 = \frac{2148.59 \times 10^3}{\frac{\pi}{32} \times 75^4} \times \frac{1000}{0.8 \times 10^5}$$

$$\theta_1 = 8.646 \times 10^{-3} \text{ radian}$$

$$P_2 = \frac{T_2 \, 2N\pi}{60}$$

$$(45-15) \times 10^3 = \frac{2 \times 200 \times T_2 \pi}{60}$$

$$T_2 = 1432.39 \ Nm$$

$$T_2 = 1432.39 \times 10^3 \ N \cdot mm$$

$$\frac{T_2}{J} = \frac{\tau}{R}$$

$$\tau = \frac{1432.39 \times 10^3}{\frac{\pi}{32} \times (50^4)} \times \frac{50}{2}$$

$$\boxed{\tau = 58.36\ N/mm^2}$$ ✓

$$\frac{T}{J} = \frac{G\theta}{l}$$

$$\theta_2 = \frac{1432.39 \times 10^3 \times 2000}{0.8 \times 10^5 \times \frac{\pi}{32}(50^4)}$$

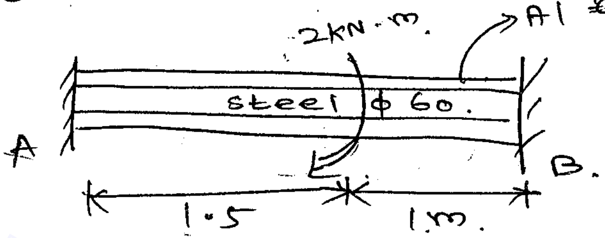
$$\boxed{\theta_2 = 0.0583} \text{ radians}$$

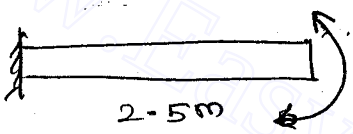
$$\theta = \theta_1 + \theta_2 = 8.646 \times 10^{-3} + 0.0583$$

$$= 0.06701 \text{ radians}$$

3. A steel shaft of 60.50 mm ϕ is coaxially fitted with aluminium shaft of 80mm. Both ends of shaft are fixed. Torque of 2kN.m is applied at a distance 1m. from right support and 1.5m from left support. Determine the torque developed @ support and maximum T. develops in shaft. $G_s = 84$ GPa

$G_{Al} = 28$ GPa.





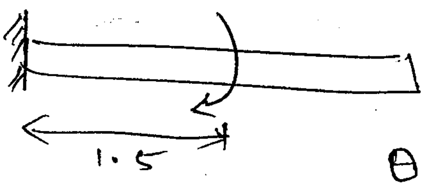
$$\theta_B = \frac{T_B \times l}{G J}$$

$$= \frac{T_B \times 2500}{} \left(\frac{1}{G_s J_s} + \frac{1}{G_{Al} J_{Al}} \right)$$

$$= T_B \times 2500 \left(\frac{32}{84\times10^3 \times \pi \times 60^4} \right) + \left(\frac{32}{28\times10^3 \times \pi (80^4 - 60^4)} \right)$$

$$\boxed{\cancel{\theta_B = 1.12\times10^{-14} T_B}}$$

$$\boxed{\theta_B = 5.587 \times 10^{-8} T_B}$$



$$\theta_A = \frac{2\times10^6 \times 1500}{} \left(\frac{1}{G_s J_s} + \frac{1}{G_{Al} J_{Al}} \right)$$

$$\boxed{\theta_A = 0.0670}$$

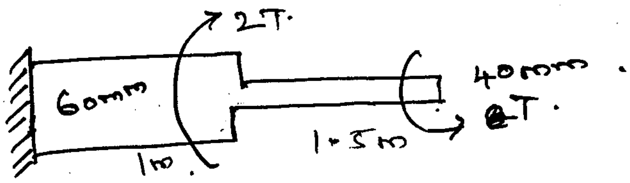
$$\theta_A = \theta_B.$$

$$0.0670 = 5.587\times10^{-8} T_B$$

$$T_B = 1.2 \times 10^6 \text{ N.mm}$$

3.) A steel shaft of 16mm ϕ is coaxially
fitted with. aluminium. ~~another~~ shaft of $\phi = 80$mm Both ends of shaft fix. 2kN-m
is applied at a distance 1m from.

4.) Figure shows a stepped steel shaft subjected to torque "T" @ free end. and 2T. @ in the opp. direction @ the junction. Determine the total angle of twist if. τ_{max} = 80MPa. G = 80GPa.



$$\frac{T}{J} = \frac{\tau}{R}.$$

$$\frac{T}{J} = \frac{\tau_{max}}{R}.$$

$$= \frac{80}{(40/2)} \times \frac{\pi}{32} 40^4.$$

$$T = 1.005 \times 10^6 \text{ N.mm}.$$

$$T = 1.005 \text{ KN.m}.$$

$$\frac{G\theta}{l} = \frac{T}{J}. \qquad \theta = G$$

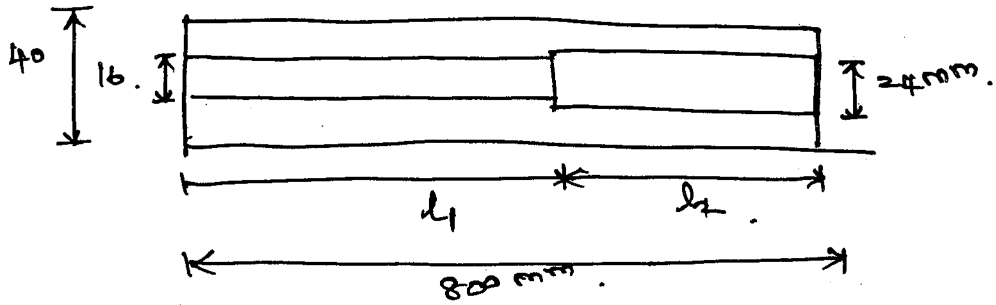
$$= \frac{1.005}{\pi} \qquad 3.73$$

$$\theta = \frac{T}{G}\left(\frac{l_1}{J_1} - \frac{l_2}{J_2}\right).$$

$$= \frac{1.005 \times 10^6}{80 \times 10^3} \left[\frac{1000}{\frac{\pi}{32} 60^4} - \frac{1500}{\frac{\pi}{32} 40^4}\right]$$

$$\boxed{\theta = 3.71^\circ}$$

5. Figure shows a hollow shaft. Determine the power transmitted @ 200 rpm $\tau_{max} = 70\text{ MPa}$. Find the length of the shaft portions. If twist produced in two shafts are equal.



Solution:

$$\tau = \frac{16T}{\pi}\left(\frac{D}{D^4-d^4}\right)$$

The maximum value of shear stress in which will reach in the portion "d" is higher.

$$\tau = \frac{16T}{\pi}\left(\frac{D}{D^4-d^4}\right)$$

$$P = \frac{2\pi NT}{60}$$

$$70 = \frac{16\times T}{\pi}\left(\frac{40}{40^4-24^4}\right)$$

$$\boxed{T = 765.64\times 10^3\ \text{N.mm}}$$

$T = 765.64\ \text{N.m}$

$$P = \frac{2\pi NT}{60} = \frac{2\pi\times 200\times 765.64}{60}$$

$$\boxed{P = 16.03\text{ kW}}$$

$$\frac{Tl}{JG} = \theta$$

$$\theta_1 = \theta_2$$

$$\frac{\cancel{T}l_1}{J\cancel{G}} = \frac{\cancel{T}l_2}{J\cancel{G}}$$

$$l_1 = l_2\cdot\frac{\frac{\pi}{32}(40^4-16^4)}{\frac{\pi}{32}(40^4-24^4)}$$

$$l_1 = 1.12 l_2$$

$l_1 + l_2 = 800$

$1.12 l_2 + l_2 = 800$

$$\boxed{l_2 = 377.45\text{ mm}}$$

$$\boxed{l_1 = 422.55\text{ mm}}$$

S.O.M.

* Σ Normal stresses acting @ any 2 $\perp^{r}$ plane remains constant.

i.e $\sigma_x + \sigma_y = \sigma_1 + \sigma_2$.

$$\sigma_n = \frac{\sigma_x + \sigma_y}{2} = \frac{\sigma_1 + \sigma_2}{2}$$

$$\tau = \frac{P_1 - P_2}{2}.$$

$$\sigma_r = \sqrt{\frac{P_1^2 + P_2^2}{2}}.$$

$$\sigma_n = \left(\frac{\sigma_x + \sigma_y}{2}\right) + \left(\frac{\sigma_x - \sigma_y}{2}\right)\cos 2\theta + \tau \sin\theta.$$

$$\tau_\theta = -\left(\frac{\sigma_x - \sigma_y}{2}\right)\sin 2\theta + 2 \cos 2\theta.$$

Complex stress.

$\sigma_n = \sigma_x \sin^2\theta + \sigma_y \cos^2\theta + \tau \sin 2\theta$

$\tau_\theta = \left(\frac{\sigma_x - \sigma_y}{2}\right)\sin 2\theta + \tau\cos 2\theta.$

$\sigma_R = \sqrt{\sigma_n^2 + \tau_\theta^2}.$

$\sigma_r = \tan^{-1}\left(\frac{\tau_\theta}{\sigma_n}\right)$ w.r.to σ_n.

P. in

$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2}$

$\tan(180 - 2\theta) = \left(\frac{2\tau}{\sigma_x - \sigma_y}\right)$ $\theta = 90 - \frac{1}{2}.$

$\theta = \frac{\sigma_x - \sigma_y}{2}\sin 2\theta + \tau\cos 2\theta.$

$-\tau\frac{\cos 2\theta}{\sin 2\theta} = \left(\frac{\sigma_x - \sigma_y}{2}\right)$

$\sigma_x > \sigma_y$. $\theta = 90° - 45$

$\sigma_x = \sigma_y = \theta = 45°$ $\quad -\tau$ ~~tan~~ $= \frac{\sigma_x - \sigma_y}{2}$

$\sigma_x < \sigma_y$ $\quad \theta = -45°$ $\quad \cot 2\theta$.

$\tan 2\theta = \frac{-2\tau}{\sigma_x - \sigma_y}$

$\tan(180 - 2\theta) = \frac{2\tau}{\sigma_x - \sigma_y}$

$\tau_{max} = \frac{\sigma_x - \sigma_y}{2}$ $\quad \theta_{max} = 45°$

θ $\tan 2\theta = \frac{2\tau}{\sigma_x - \sigma_y}$

Theories of Failure

R, S, G, H, V

$\sigma_t < \sigma_c$ $\quad \sigma_1 = \frac{1}{2}\left(\frac{\sigma_x + \sigma_y}{2}\right)$ $\quad \sqrt{\frac{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}{2}}$

G c m.

Ruling

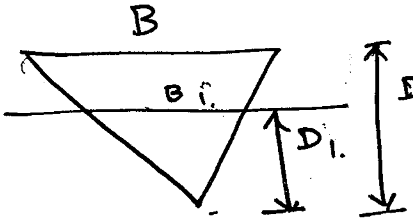
$(0, 0, \sigma_1, \sigma_2)$

σ_t / F.O.S $\quad \geq \sigma_t$

08/11/2016

STEEL

1.)



$$\frac{B}{D} = \frac{B_1}{D_1}$$

$$B_1 = \frac{B D_1}{D}$$

$Z_p = ?$

$$A_1 = A_2$$

$$(B+B_1) \times \frac{(D-D_1)}{\cancel{2}} = \frac{1}{\cancel{2}} \times B_1 \times D_1$$

$$\left(B + \frac{BD_1}{D}\right) \times (D - D_1) = \frac{BD_1}{D} \times D_1$$

$$\frac{\cancel{B}}{\cancel{D}}(D+D_1) \times (D-D_1) = \frac{\cancel{B}}{\cancel{D}} \times D_1^2$$

$$D^2 - D_1^2 = D_1^2$$

$$2D_1^2 = D^2$$

$$\boxed{D_1 = \frac{D}{\sqrt{2}}}$$

$$Z_p = A_1 y_1 + A_2 y_2$$

$$\cancel{= (B+B_1) \times \frac{(D-D_1)}{2} \times}$$

$$\cancel{A_1 = \left(B + \frac{BD_1}{D}\right) \times \left(B + \frac{BD_1}{D}\right)} \times \frac{\left(D - \frac{D}{\sqrt{2}}\right)}{2}$$

$(B + 2B$

LUG ANGLES.

* In order to increase the efficiency of outstanding leg in single angle and to decrease the ~~strength~~ length of the end connection. Sometimes a short length angle at the ends are connected to the gusset and the outstanding leg of the main angle directly.

* By using lug angle there will be saving in gusset plate but additional fasteners and angle member are required. Hence nowadays it is not preferred.

* As per IS 800 2007. specification of lug angle are

a) minimum of two bolt (or) equivalent weld should be used for attached lug angle to the gusset, if the main member is an angle.

(b) The whole area of member shall be taken as effective area rather than net section.

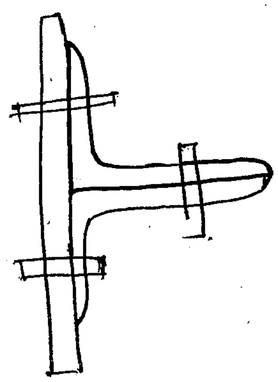
(c) The load on the lug angle with gusset plate shall be 20% excess of load acting on outstanding leg of the main angle

(d) The connection ~~and~~ with lug angle main angle shall be based on 40% excess of load acting on outstanding leg of main angle.

(e) In case of channel as main tension member. → Lug angle – Gusset = 10% excess. of load acting on outsanding leg of channel section.

(f) Lug angle - Main channel member } = 20% of load of outstanding leg of main channel.

1. Design the lug angle and its connection for a angle 125 x 75 x 10 carrying a $P_u = 300$ kN. Use 20mm ϕ bolt.



Step 1:

$P = 300$ kN.

$A_g = (125 + 75 - 10) \times 10 = 1900 \, mm^2$

$A_1 = (125 - 5) \times 10$.

$A_1 = 1200 \, mm^2$

$A_2 = (75 - 5) \times 10$.

$A_2 = 700 \, mm^2$

$$P_1 = \frac{P}{A_g} \times A_1.$$

$$= \frac{300 \times 1200}{1900}.$$

$$\boxed{P_1 = 189.47 \text{ kN.}}$$

$$P_2 = 300 - 189.47.$$

$$\boxed{P_2 = 110.53 \text{ kN.}}$$

Step 2: Find the area of lug angle.

$$1.2P = f \times A.$$

$$1.2 \times 110.53 = \frac{250}{1.1} \times A.$$

$$\boxed{A = 583.6 \, mm^2}$$

Assume.

ISA 75 x 75 x 8 mm.

Step 3 : Design of lug angle - angle section.

$$n = \frac{1.4 \times 110.53}{45.26}$$

$= 3.418$

$n \simeq 4$ bolt.

Step 4 : Design of lug angle - gusset plate

$$n = \frac{1.2 \times 110.53}{45.26}$$

$= 2.93$

$\boxed{n \simeq 3 \text{ bolt}}$

Step 5 : Design of connected leg.

$$n = \frac{189.47}{45.26}$$

$= 4.19$

$\boxed{n = 5 \text{ bolt}}$

2. An angle section 150 x 75 x 10 mm is connected to gusset plate with 5 bolt of 20mm ϕ bolt take g = 100mm from the corner Determine ultimate Load. (8)

Solution:

100.

Yielding criteria.

$$P_{at} = \frac{f_y}{1.1} \times A_g.$$

$$= \frac{250}{1.1} \times (150+75-10) \times 10.$$

$$\boxed{P_{at} = 488.64 \text{ kN}}$$

Rupture criteria.

$$P_{at} = \frac{0.9 f_u}{1.25} A_{nc} + \beta\left(\frac{f_y}{1.1}\right) \times A_g.$$

$$= \frac{0.9 \times 410}{1.25} \times \left(150 - \frac{22}{5}\right) \times 10.$$

$$P_{at} = 337.86 \text{ kN} + 1.11\left(\frac{250}{1.1}\right) \times (75-5) \times 10$$

Block shear.

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L}\right).$$

$$= 1.4 - 0.076\left(\frac{75}{10}\right)\left(\frac{250}{410}\right)\left(\frac{165}{4 \times 50}\right).$$

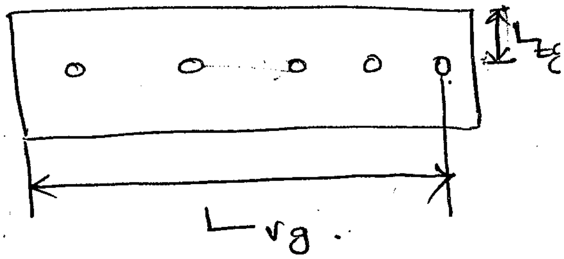
$$\beta = 1.11.$$

$$\boxed{P_{at} = 538.096 \text{ kN}}$$

$$T_{db} = \frac{A_{vg}}{\sqrt{3}} \frac{f_y}{\gamma_{m0}} + A_{tn} \frac{0.9 f_u}{\gamma_{m1}}$$

$$\left(\frac{A_{vn}}{\sqrt{3}} \cdot \frac{0.9 f_u}{\gamma_{m1}} \right) + A_{tg} \frac{f_y}{\gamma_{m0}}$$

least.



$A_{vg} = L_{vg} \times t$

$= [30 + 4(50)] \times 10$

$A_{vg} = 2300\ mm^2$

$A_{vn} = [230 - 4.5(22)] \times t$

$A_{vn} = 1310$

$A_{tg} = L_{tg} \times t$

$= 50 \times 10$

$= 500\ mm^2$

$A_{tn} = (50 - \frac{22}{2}) \times 10$

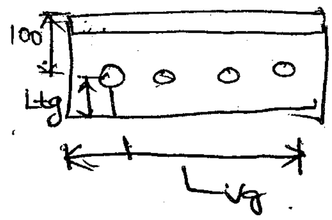
$\cancel{A_{tn} = 890\ mm^2}$

$A_{tn} = 390\ mm^2$

$$T_{db} = \left(\frac{2300}{\sqrt{3}} \times \frac{250}{1.1} \right) + \left(\frac{\cancel{890}\ 390 \times 0.9 \times 410}{1.25} \right)$$

$= \cancel{564.052\ kN} = 416.92\ kN$

$$T_{db} = \left(\frac{1310 \times 0.9 \times 410}{\sqrt{3} \times 1.25} \right) + \left(\frac{\cancel{1000}\ 500}{\cancel{\sqrt{3}}} \times \frac{250}{1.1} \right)$$



$T_{db} = 336.9\ kN$

$$\boxed{\text{Ultimate Load} = 336\ kN}$$

18/12/2015

STEEL.

TENSION MEMBER.

* For preliminary design of tensio member IS 800 recommends following formula for design shearing strength of net section.

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

where

$\alpha = 0.6$ for one or 2 bolt.

$\alpha = 0.7$ for 3 bolt

$\alpha = 0.8$ for 4 and more no. of. bolt

$\alpha = 0.8$ for weld length.

* Maximum Slenderness Ratio: (Stiffness requirement)

→ A tension member in which reversal of direct ~~stress~~ due ~~to~~ other than wind or seismic load. (due to live load and Dead load)

$$\lambda \not> 180$$

→ Members subjected to reversal of stress due to wind load or Seismic load $\lambda \not> 350$.

→ For any other than except Pre stress (pre - ~~ten~~sion) $\lambda \not> 400$.

1. A flat ties 150ISF 16 is carrying load subjected to reversal of stress other than wind load. Determine limiting length of flange.

Solution:

$$\frac{l}{r_{min}} \leq 180$$

$$r_{min} = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{(\cancel{b}t^{3}/12)}{\cancel{b} \times \cancel{t} \times t}}$$

$$= \sqrt{\frac{t^2}{12}}$$

$$r_{min} = \sqrt{\frac{16^2}{12}}$$

$$\frac{l}{\sqrt{\frac{16^2}{12}}} \leq 180.$$

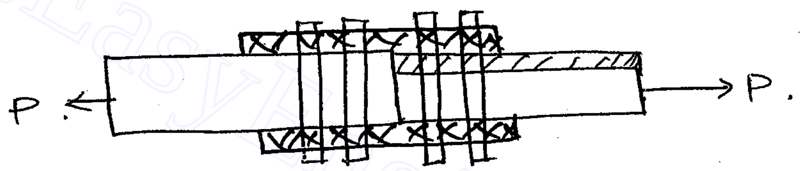
$$l \leq 180 \times \frac{16}{\sqrt{12}}.$$

$$l \leq 831.38$$

$$\boxed{l = 831 mm}$$

Tension splice:

* It is joint for tension member.
* ~~It~~ Tension splice is provided when length of the member required is lesser than the available length from Indian rolling mill. or factory. (or)
* when two length of a tension member has different thickness are to be connected with fillet plate.
* Tension splices are provided on the both sides of a member jointed in the form of cover plates.



* The strength of splice plate and bolt/weld connecting them should have strength atleast equal to design load.
* The design shear capacity of bolt carrying shear through packing plate in excess of 6mm ~~can~~ shall be decreased by a factor.

$$\boxed{\beta_{pkg} = 1.0 - [0.0125 t_{pkg}]}$$

1. 2 plates 8mm and 16mm are to be jointed by a double cover butt joint with a packing plate and cover plate of t = 8mm. what is the ~~factor~~. effect of packing on design shear ~~strength~~ of. bolt. The factor is.

$$\beta_{pkg} = 1 - 0.0125(8).$$

$$\boxed{\beta_{pkg} = 0.9}$$

Inference: The design shear strength is reduced by 10%.

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8/12/2015

COMPRESSION MEMBERS.

* A compression member is a structural member which is subjected to two equal and opposite compressive forces applied at its ends.

* Example:
  Top chords of trusses, Bracing members, boom in a crane, compression flanges of built-up beams, and rolled beams.

* Types.

Short column.

$$\lambda < \sqrt{\frac{\pi^2 E}{f_p}}$$

Long column.

$$\lambda \geq \sqrt{\frac{\pi^2 E}{f_p}} \qquad \boxed{f_p = f_y}$$

* short columns subjected to axial compression fails by yielding or crushing.

* Very long column fails by elastic buckling by Eluer's Load.

* Intermediate column generally fails by inelastic buckling.

Design compressive strength of member (P_d)

* slenderness ratio and yield stress & are the factors affecting ultimate strength of axially loaded column.

$$\boxed{P_d = A_e\, f_{cd}}$$

A_e - effective cross section of column.
f_{cd} - design stress in compression.

* IS 800 propose 4 multiple curves for column namely a, b, c, d. based on Perry Robertson approach.

* code reccommends

$$\boxed{f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + (\phi^2 - \lambda^2)^{0.5}}}$$ 7.1.2.1

$$\boxed{f_{cd} = \chi\,(f_y/\gamma_{mo})} \leq \frac{f_y}{\gamma_{mo}}.$$

$$\boxed{\chi = \frac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}}}$$

where
$$\boxed{\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]}$$

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

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

α - imperfection factor

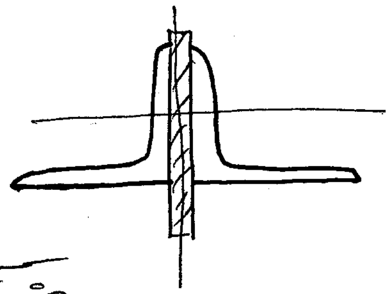
χ - stress reduction factor for residual stresses.

Table-7 of IS:800:

Buckling class.	a	b	c	d.
α.	0.21	0.34	0.49	0.76.

f_{cc} - Euler buckling stress.

1. A principle strut of a roof truss composed of 2 equal angle ISA 75x75x6 ~~or~~ are connected back to back, on either side of 10mm thick gusset plate the c/s area of each angle is 866 mm^2 M.I $I_{zz} = I_{yy} = 457000\ mm^4$. the distance of centroid $C_{zz} = C_{yy} = 20.6\ mm$. what is the r_{min}.



$$I_z = 2[I_{zz} + Ah^2]$$

$$= 2[457000]$$

$$I_z = 914 \times 10^3\ mm^4.$$

$$I_y = 2\left[I_{yy} + A\left(20.6 + \frac{10}{2}\right)^2\right]$$

$$= 2\left[457000 + 866(25.6)^2\right]$$

$$\boxed{I_y = 958.33 \times 10^3\ mm^4}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$

$$= \sqrt{\frac{914 \times 10^3}{866 \times 2}}$$

$$\boxed{r_{min} = 22.97\ mm.}$$

A tubular column section $D_o = \sqrt{3}D$, $D_i = D$. The column is effectively held in position and unrestrained against rotation. $\lambda_{eff} = 200$ what is $\frac{l}{d}$ ratio.

Solution:

$$r_{min} = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{\frac{\pi}{64}\left((\sqrt{3}d)^4 - d^4\right)}{\frac{\pi}{4}\left((\sqrt{3}d)^2 - d^2\right)}}$$

$a^4 - b^4$

$(a^2)^2 - (b^2)^2$

$(a^2+b^2)(a^2-b^2)$

$$= \sqrt{\frac{1}{16} \cdot \frac{\cancel{\left((\sqrt{3}d)^2 - d^2\right)}\left((\sqrt{3}d)^2 + d^2\right)}{\cancel{(\sqrt{3}d)^2 - d^2}}}$$

$$= \sqrt{\frac{1}{16}\left(3d^2 + d^2\right)}$$

$$= \sqrt{\frac{4d^2}{16}}$$

$$= \sqrt{\frac{d^2}{4}}$$

$$r_{min} = \frac{d}{2}$$

$$\lambda_{eff} = \frac{l}{r_{min}}$$

$$200 = \frac{l}{(d/2)}$$

$$\frac{l}{d} = \left(\frac{200}{2}\right)$$

$$\boxed{\frac{l}{d} = 100}$$

2. Determine the design axial load on the column ISMB ~~600~~ 350 height = 3m. both ends are pinned. $f_y = 250\ N/mm^2$. $f_u = 410\ N/mm^2$. $E = 2\times10^5\ N/mm^2$.

Properties of ISMB 350 are:

$A = 6670\ mm^2$. $t_f = 14.2mm$

$t_w = 8.1\ mm$ $b_f = 140mm$

$r_{zz} = 143$ $r_{yy} = 28.4mm$.

Solution:

Buckling about z-z axis is a.

Buckling about y-y axis is b.

$$\boxed{P_d = A_e \times f_{cd}}$$

$$f_{cd} = \chi\,(f_y/\gamma_{mo}) \qquad \chi = \frac{1}{\phi+(\phi^2-\lambda^2)^{0.5}}$$

$$\phi = 0.5\,[1+\alpha(\lambda-0.2)+\lambda^2) \qquad \lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2\times2\times10^5}{\left(\frac{1\times3000}{28.4}\right)^2}$$

$$\boxed{f_{cc} = 176.9\ N/mm^2}$$

$$\lambda = \sqrt{\frac{250}{176.9}} = 1.19. \qquad \boxed{\lambda = 1.19}$$

$$\phi = 0.5\,[1+0.34(1.19-0.2)+1.19^2]$$

$$\boxed{\phi = 1.37}$$

$$\chi = \frac{1}{1.37+(1.37^2-1.19^2)^{0.5}}$$

$$\boxed{\chi = 0.48}$$

$$f_{cd} = \frac{0.48 \times 250}{1.1}$$

$$\boxed{f_{cd} = 109.09 \ \ N/mm^2}$$

$$P = A_e \times f_{cd}.$$

$$= 6670 \times 109.09$$

$$\boxed{P = 727.63 \ kN}$$

* fcd shall be assumed as following

for angle strut, $f_{cd} = 90$ MPa.

for rolled steel beam section $\}$, $f_{cd} = 135$ MPa.

column with heavy factored load $\}$, $f_{cd} = 200$ MPa.

* Maximum slenderness ratio for compression member:

→ A member carrying D.L & L.L ⊁ 180.

→ Member subjected to W.L or S.L ⊁ 250.

→ Compression flange of a beam restrained against torsional buckling ⊁ 300.

3.) A strut of roof truss 60x60x6 are connected to 10mm gusset plate. is subjected compressive load resulting from wind Load c/s Area of each angle. 684 mm² $I_{zz} = I_{yy} = 226\,000$ mm⁴. $I_{uu} = 360000$ mm⁴ $I_{vv} = 91,000$ mm⁴. Determine the maximum length of a strut of a truss.

$$\frac{l}{r_{min}} \leq 250.$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{91000}{684}} = 11.53.$$

$$\frac{l}{11.53} \leq 250.$$

$$l \leq 250 \times 11.53.$$

$$l \leq 2883.58$$

$$\boxed{l = 2880 \text{ mm}}$$

18/12/2015.

STEEL COLUMN.

BUILT UP COLUMNS:

* It is used when rolled steel section do not provide required sectional area (or). large radius of gyration of column is required to different direction.
* when ever two members are used as column. then they must be connected by lacings or battered. systems.

Lacing and Built-up section:

The different components of built up section are placed in such a way that the build-up section has same radius of gyration about both the axis.

Different component of built up section are laced up or connected together so that they act as a single column.

Lacing is preffered in eccentric load and battens are preffered in concentric load.

LACING SPECIFICATIONS:

* Flate bars, angle, channel and tublar section are used for lacing.
* It should be continued upto the top of the column.
* It should be shadow of each other.
* It should not be varied about the length.
* Tie plates should be provided at the ends and where lacing systems are interprupted.

Lacing (Design specification):

* Effective slenderness ratio can be increased by 5% to account for shear deformation due to unbalanced horizontal forces.
* Angle of inclination 40°-70°.
* $\lambda_{eff} \not> 145$.

* $t_{min} = \frac{l}{40}$. – single lacing.

$t_{min} = \frac{l}{60}$ – double lacing.

* Effective length:

$l_e = l$ – single lacing.

$l_e = 0.7l$ – Double lacing

$l_e = 0.7l$ – welded lacing.

* Spacing:

$\frac{S_{max}}{r_{min}} \not> 0.7 \times \lambda_{column}$.

$\left(\frac{S_{max}}{r_{min}}\right) \not> 50$.

* Minimum width:

$b_{min} = 3 \times d$

* Load on lacing:

The lacing should be designed to resist transerve shear of 2.5% of column load.

V (transerve shear load) = 2.5% of degin axial load.

* Lacing should be designed to resiste addition shear due to bending if the column carries bending.

* For single lacing the force in the lacing bar $F = \frac{V}{N \sin\theta}$. (If N = 2)

~~Double lacing~~ $F = \frac{V}{2 \sin\theta}$.

Double lacing, $F = \frac{V}{4 \sin\theta}$

1. A build colum made up of ISMC 450. Channels placed back to back. carries a factored load of 2250 kN single lacing with $\theta = 45^\circ$. with longitudinal axis. what is design ~~load~~ transverse shear on lacing.

$$V = \frac{2.5}{100} \times 2250$$

$$\boxed{V = 56.25\,kN}$$

2. A built column consists of ISMC 300 placed back to back @ a spacing = 200 mm and carries an working axial load of 1500 kN the Double lacing provided with angle 45° with longitudinal axis. what is the design axial load of lacing.

Ultimate load = 1500 × 1.5.

Ultimate load = 2250 kN.

$$V = 2.5\,\%\ 2250.$$

$$= \frac{2.5}{100} \times 2250.$$

$$\boxed{V = 56.25\,kN}$$

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

$$= \frac{56.25 \times 10^3}{4 \sin 45}.$$

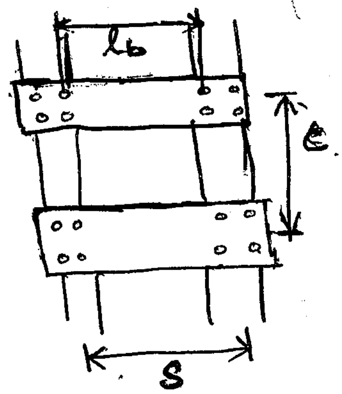
$$\boxed{F = 19.89\,kN.}$$

19/12/2015

STEEL

BATTEN

* Minimum ~~3~~ bay there should be 3 bays of battening. (i.e) the no. of. battens should be such that the member should be divided not less than 3 parts longitudinally.
* Flat plates are used ~~by~~ as battens.
* λ_{eff} should be increased by 10%.
* Spacing



$$\frac{C}{r_{min}} \ngtr 50.$$

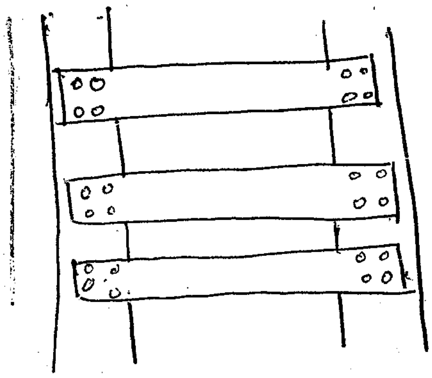
$$\frac{C}{r_{min}} \ngtr 0.7 \times \lambda_{column}.$$

* S is transverse distance between centroid of the bolt group.
* Battens will be designed to carry B.M and S.F arising from a transverse shear, 2.5% of Design axial load where.

$$\boxed{V_1 = \frac{VC}{NS}}$$

N = No. of. parallel plates of battens.

$$\boxed{M = \frac{VC}{2N}}$$



* . thickness :

$$t > \frac{l_b}{50}.$$

* Effective depth. (d)

$d > a$ for end batten.

$d > 2b$ for any batten.

$d > \frac{3a}{4}$ for intermediate batten.

a - distance b/ω. centre to centre ~~Centre~~ of flangle ~~b/w~~ of the column section.

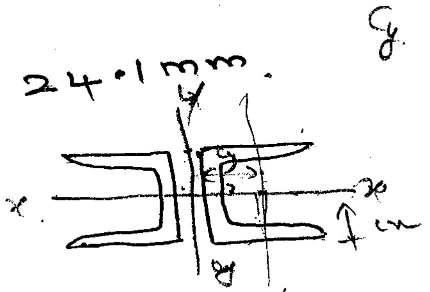
b - width of the flange of column section.

10 2 channels ISLC 350 are placed. are placed back to back total length = 6m one end of the restrained against rotation. ~~es~~restrained only. but the other end is ~~translation~~. Determine against rotation and slenderness ratio if spacing of channel, slenderness is provided, safe batten system and LSM compressive load as per WSM

Pro

$A = 4949\ mm^2 \quad I_{xx} = 9312.6 \times 10^4\ mm^4.$

$I_{yy} = 394.6 \times 10^4\ mm^4. \quad C_y = 24.1\ mm.$



Solution:

$$r_y \geq r_x.$$

$$\sqrt{\frac{I_y}{A}} \geq \sqrt{\frac{I_x}{A}}$$

$$I_y \geq I_x.$$

$$\cancel{7}\left(I_{yy} + Ah^2\right) \geq \cancel{g} I_{xx}.$$

$$\cancel{2}\left(394.6\times10^4 + 4947\left[\begin{matrix}24.1\\+S/2\end{matrix}\right]^2\right) \geq 9312.6\times10^4.$$

$$\boxed{S = 220\,mm}$$

Guided roller $K = 1.2$

$$\lambda = \frac{KL}{r_{min}} = \frac{1.2\times6000}{\sqrt{\frac{I_{min}}{A}}}.$$

$$= \frac{1.2\times6000}{\sqrt{\frac{2(9312.6\times10^4)}{2\times4947}}}$$

$$\lambda = 52.48.$$

$$\lambda_{batten} = 1.1\times\lambda.$$

$$= 52.48\times1.1.$$

$$\boxed{\lambda = 57.7\,mm.}$$

L.S.M.

171.42.

$$P_a = A_e \cdot f_{cd}.$$

1696.

$$= 2\times4947 \times 171.42.$$

$$\boxed{P_a = 1.628\times10^6\,N}$$

W.S.M

$$P_c = \sigma_{ac}\times A_{total}.$$

$$\sigma_{ac} = 0.6\frac{f_{cc}\,f_y}{\left(f_{cc}^n + f_y^n\right)^{1/n}}$$

$n = 1.4$ constant

$$f_{cc} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2\times2\times10^5}{(57.7)^2} = \underline{592.89}\ N/mm^2$$

$$\sigma_{ac} = \frac{592.89\times250\times0.6}{\left((592.89)^{1.4} + (250)^{1.4}\right)^{1/1.4}} = \cancel{207.45}\ N/mm^2$$

$$\boxed{\sigma_{ac} = 124.47\ N/mm^2}$$

1. Design lacing and ~~batten~~ batten for axial load of 1000kN. longitudinal. ~~to~~ Load, Transverse load, Moment no. of batten plate. 4, S = 400mm

Solution:

Design of lacing:

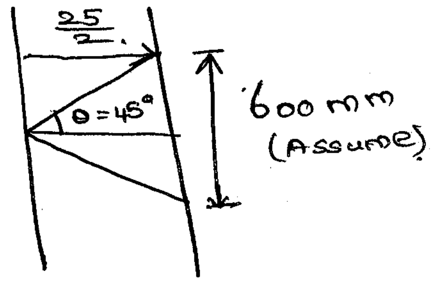
(i) Longitudinal load = 2.5 % load on column

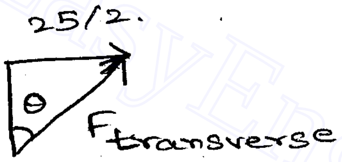
$= \frac{2.5}{100} \times 1000.$

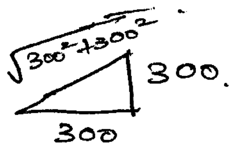
$= 25\,kN.$

(ii) Transverse load.

Assume $\theta = 45^\circ$.







$\sin\theta = \frac{(25/2)}{F_{transverse}}.$

$F_{transverse} = \frac{12.5}{\sin 45^\circ}.$

$\boxed{F_{transverse} = 17.68\,kN}$

(iii) Width:

b = 3 × d of bolt.

= 3 × 20.

$\boxed{b = 60mm}$

(iv) thickness:

Assume $l_{eff} = \sqrt{(300^2 + 300^2)}$

$l_{eff} = 424.26mm$

$t = \frac{1}{40} l_{eff}$

$= \frac{1}{40} \times 424.26.$

$\boxed{t = 10.61\,mm}$

$\boxed{t = 12mm}$

(v) . $I_{xx} = \frac{bt^3}{12}$ $\qquad I_{yy} = \frac{b^3 t}{12}.$

$= \frac{60 \times 12^3}{12}$ $\qquad = \frac{12 \times 60^3}{12}.$

$\boxed{I_{xx} = 8640 \text{ mm}^4}$ $\qquad \boxed{I_{yy} = 216 \times 10^3 \text{ mm}^4}$

$r_{min} = \sqrt{\frac{8640}{60 \times 12}}.$

$\boxed{r_{min} = 3.464.}$

$\lambda_{lacing} \leq 145.$

$\frac{l_{eff}}{r_{min}} = \left(\frac{424.26}{3.464}\right).$

$= 122.48 \leq 145 \text{ mm}$

Hence safe.

Batten:

10/12/2015

STEEL

COLUMN BASES

* The main function of base plate is to spread the column load over a sufficiently wide area and keep the footing from over stressed.

* Types of column bases:
  - → slab base
  - → Gusseted base.

* **Slab base :**
  For purely axially load, a plane square steel plate or a slab base attached to the column is adequate.

* Gusseted Base
  when there is a large moment in relation to the vertically applied a gusseted base may be required.

* The ~~minimu~~ minimum thickness (t_s) of rectangular slab bases under axial compression shall be for I, H, channel Box,

LSM.

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

(thickness of column flange)

WSM

$$t_s = \sqrt{\frac{3w\,(a^2 - 0.3b^2)}{0.75\,f_y}} > t_f.$$

(As per IS 800:2007 Annexure G

Column Base:

Main Function:

→ To spread the column load over a wide area

→ To avoid footings from over stressed

* Types → Slab base.
→ Gusseted Base.

* Slab base
Suitable.

$$w = \frac{\text{load in column}}{\text{size of base plate}}$$

w - pressure on underside of the base plate.

a, b - larger and smaller projection of the slab base beyond the column.

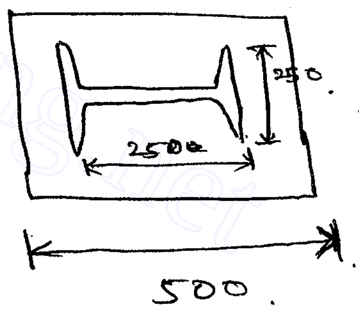
1. A slab base of size 500x500 mm is to be provided below a column section IS HB 250 $D_f = 9.7$ mm. $b_f = 250$ mm. supports a load of 2000 kN $f_y = 250$. $f_u = 410$ $\gamma_{m0} = 1.1$ $\gamma_{m1} = 1.25$. Find the t_s.

<u>solution</u>:

$$w = \frac{2000 \times 10^3}{500 \times 500} = 8\ N/mm^2.$$

$$a = \frac{500 - 250}{2} = 125\ mm$$

$$b = \frac{500 - 250}{2} = 125\ mm.$$



$$t_s = \sqrt{\frac{2.5 \times 8}{(250/1.1)} \times (125^2 - 0.3(125)^2)}$$

$$t_s = 31.7\ mm > 9.7\ mm$$

Provide 35 mm. thick base plate.

Gusseted Base:

* The design bending stress @ critical section.

$$\boxed{M_d = \frac{1.2 f_y \times Z}{\gamma_{m0}}}$$

* Thickness of gusseted base.

$$\boxed{t = c\sqrt{\frac{2.75w}{f_y}}}$$

C - cantilever. projection.

COLUMN SPLICES

* The design load on the splice section is dependent on end condition of column. to column connect

→ If ends are machine ended } = 50% of the column load will be transformed to lower portion of column & 50% will be carried by splice plate.

→ If ends are hand cut. (or) not machine ended. (no complete bearing) } = 100% of the column is shared by splice plate.

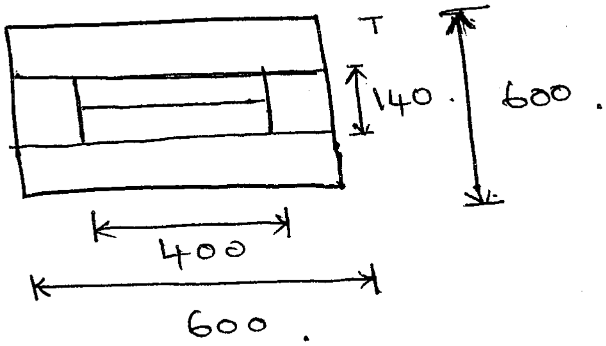
* If the moment is applied on the column. then it is converted into load that load is totally resisted by one splice plate only (i.e) Load on one splice plate

$$P_1 = \frac{P_{column}}{4} + \frac{M}{D} \quad \text{(Machine ended)}$$

$$P_2 = \frac{P_{column}}{2} + \frac{M}{D} \quad \text{(Not machine ended)}$$

* splice plate is designed as compression member.

1. what is the "C" value for the gusseted base plate shown in the figure.



cantilever projection $C = \frac{600-140}{2}$.

$= 230 \text{ mm}$.

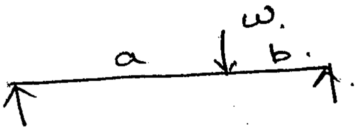
24/12/2015

STEEL

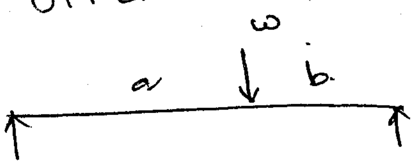
PLASTIC MOMENT CAPACITY

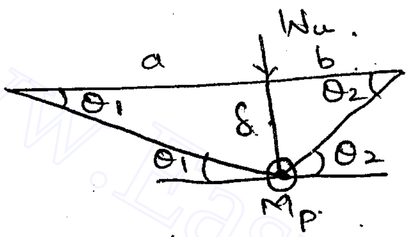
PLASTIC MOMENT THEORY.

1. Find out the collapse load and M_p for. S.S beam subjected to eccentric load.



UPPER Bound Theorem





$\delta = a\theta_1 = b\theta_2$

$$\boxed{\theta_1 = \frac{b\,\theta_2}{a}}$$

EWD = IES.

$W_u \times \delta = M_p(\theta_1 + \theta_2)$

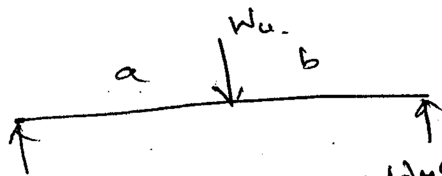
$W_u \times a\theta_1 = M_p\left(\theta_1 + \frac{\theta_1 a}{b}\right)$

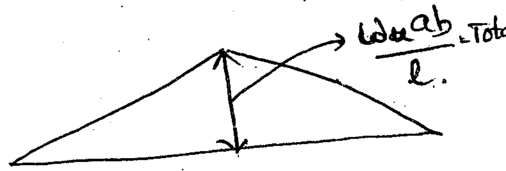
$W_u \times a\,\cancel{\theta_1} = M_p \cancel{\theta_1}\left(\frac{b+a}{b}\right)$

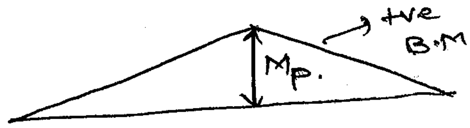
$$\boxed{W_u = \frac{M_p\, l}{ab}}$$

$$\boxed{M_p = \frac{W_u\, ab}{l}}$$

Lower Bound Theorem







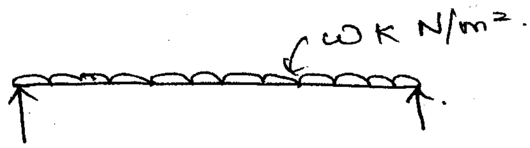
$-ve_{B.M}$.

Total B.M = (+ve B.M) + (-ve B.M)

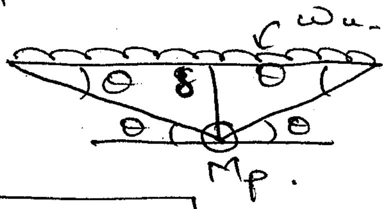
$$\boxed{\frac{W_u ab}{l} = M_p.}$$

$$\boxed{W_u = \frac{M_p\, l}{ab.}}$$

2.)



Upper bound Theorem



$$\boxed{\delta = \frac{\theta l}{2}}$$

E.W.D = I.E.S.

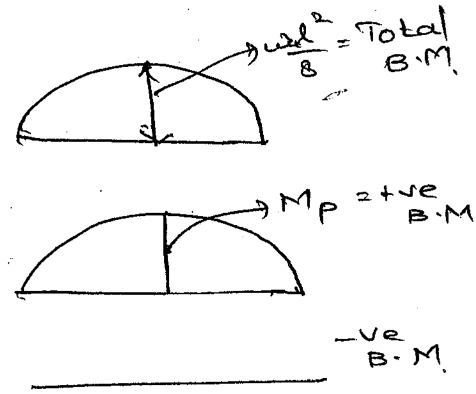
$$w_u \frac{\delta}{2} = M_p(\theta+\theta)$$

$$W_u \times l \times \frac{\cancel{\theta} l}{4} = 2M_p\cancel{\theta}$$

$$\boxed{M_p = \frac{W_u l^2}{8}}$$

$$\boxed{W_u = \frac{8M_p}{l^2}}$$

Lower bound Theorem

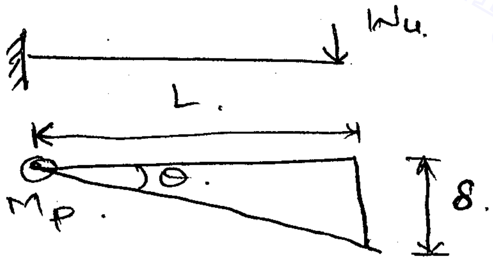


Total B.M = (+ve B.M) + (-ve B.M)

$$\boxed{\frac{W_u l^2}{8} = M_p}$$

$$\boxed{W_u = \frac{8M_p}{l^2}}$$

3.)



$$\boxed{\delta = \theta l}$$

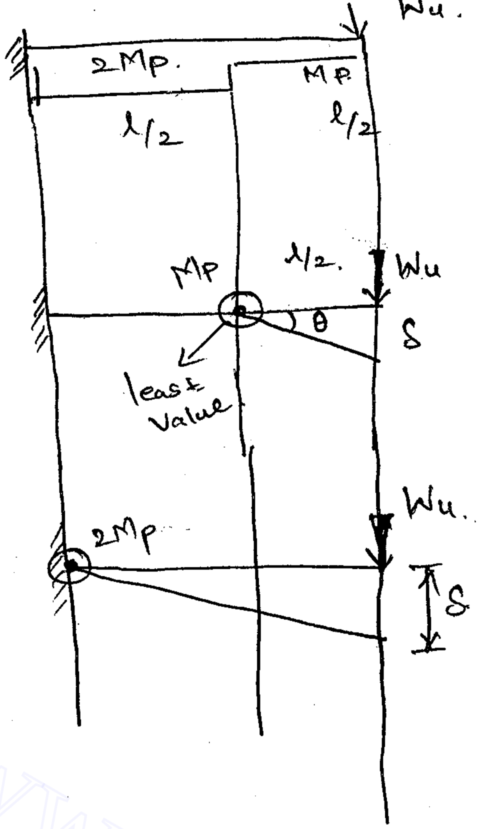
E.W.D = I.E.S.

$$W_u \times \delta = M_p \times \theta$$

$$W_u \times \cancel{\theta} l = M_p \cancel{\theta}$$

$$\boxed{M_p = W_u l}$$

4.)



case ①.

$\delta = \theta l/2$

EWD = IES.

$W_u \times \delta = M_p \times \theta$.

$W_u \times \frac{\theta l}{2} = M_p \theta$

$$\boxed{W_u = \frac{2M_p}{l}}$$

case ②.

$\delta = \theta l$.

EWD = IES.

$W_u \times \delta = \cancel{\frac{2M_p\theta}{l}} \cdot 2M_p\theta$

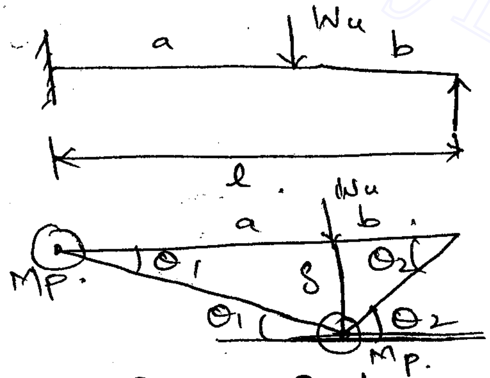
$W_u \times \theta l = 2M_p \theta$

$$\boxed{W_u = \frac{2M_p}{l}}$$

Least of case ① and case ②

$W_u = \frac{2M_p}{l}$.

⑤.



$\delta = \theta_1 a = \theta_2 b$.

$$\boxed{\theta_1 = \frac{b}{a}\theta_2}$$

EWD = IES.

$W_u \times \delta = M_p\theta_1 + M_p(\theta_1 + \theta_2)$.

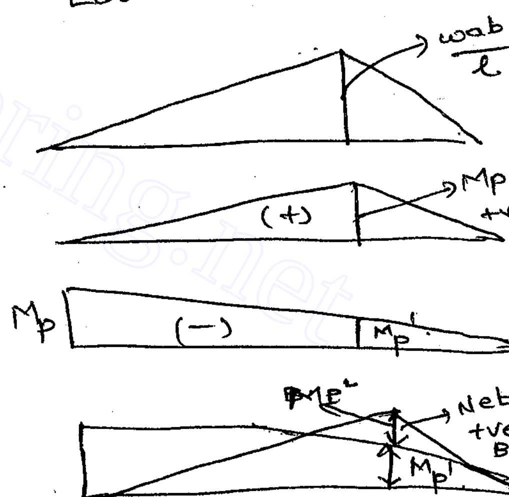
$W_u \times \theta_2 b = 2M_p \cdot \frac{b}{a}\theta_2 + M_p\theta_2$.

$W_u \times \cancel{\theta_2} b = M_p \cdot \cancel{\theta_2} \left[\frac{2b}{a} + 1\right]$.

$= M_p \frac{b + b + a}{b \times a}$.

$$\boxed{W_u = M_p\left(\frac{l+b}{ab}\right)}$$

Lower Bound Theorem



$$\boxed{M_p' = \frac{M_p \times b}{l}}$$

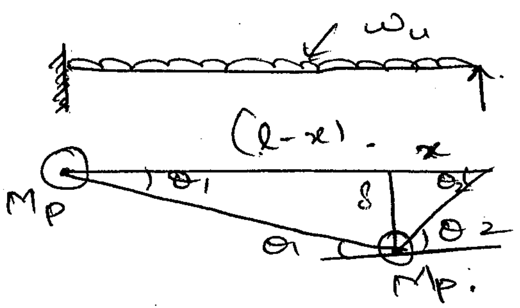
$\frac{w_u ab}{l} = M_p + M_p'$.

$\frac{w_u ab}{l} = M_p\left[1 + \frac{b}{l}\right]$

$w_u = M_p\left(\frac{l+b}{l}\right) \times \frac{l}{ab}$

$$\boxed{w_u = M_p \cdot \left(\frac{l+b}{ab}\right)}$$

6.)



$$\delta = \theta_1(l-x) = \theta_2 x.$$

$$\boxed{\theta_1 = \theta_2\left(\frac{x}{l-x}\right)}$$

EWD = IES.

$$W_u \times \frac{\delta}{2} = M_p\theta_1 + M_p(\theta_1+\theta_2)$$

$$W_u \times \frac{\theta_1}{2}(l-x) = 2M_p\theta_1 + M_p\left(\frac{l-x}{x}\right)\theta_1.$$

$$W_u \times \frac{\cancel{\theta_1}}{2}(l-x) = M_p\cancel{\theta_1}\left[2 + \frac{l-x}{x}\right].$$

$$W_u = \frac{2M_p}{l-x}\left(\frac{2x+l-x}{x}\right).$$

$$\boxed{W_u = \frac{2M_p(l+x)}{(lx-x^2)}}$$

Diff. w.r.t to x and equate to ze

$$\frac{\partial W_u}{\partial x} = 0.$$

$$2M_p \frac{d}{dx}\left(\frac{l+x}{lx-x^2}\right) = 0.$$

$$\frac{(lx-x^2)(1) - (l+x)(l-2x)}{(lx-x^2)^2} = 0.$$

$$lx - x^2 - [l^2 - 2lx + xl - 2x^2] = 0$$

$$lx - x^2 - l^2 + lx + 2x^2 = 0$$

$$x^2 + 2lx - l^2 = 0.$$

$$\cancel{x(x+2l)}$$

$$\cancel{x^2 + 3lx - lx - l^2 = 0.}$$

$$\cancel{x(x+l) - l(x+l) = 0.}$$

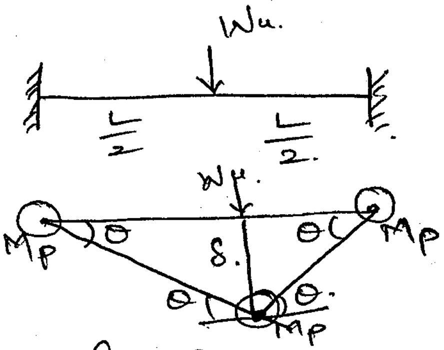
$$\cancel{(x+l)(x-l) = 0.}$$

$$\boxed{x = 0.414l}$$

$$W_u = \frac{2M_p(l+0.414l)}{(l\times 0.414l) - (0.414)l^2} = \frac{2M_p \times 1.414l}{0.2426l^2}$$

$$\boxed{W_u = 11.66\frac{M_p}{l}}$$

7.)



$$\delta = \frac{\theta L}{2}.$$

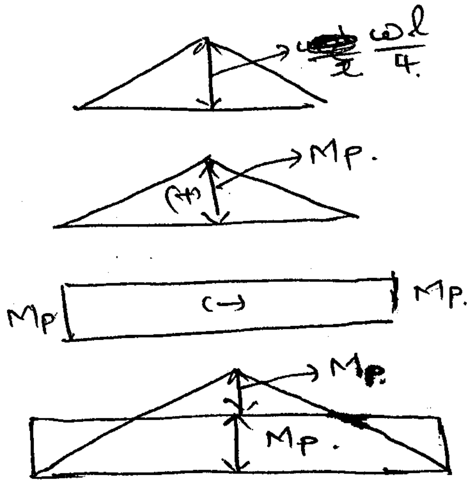
EWD = IES.

$$W_u \times \delta = M_p\theta + M_p(\theta+\theta) + M_p\theta$$

$$W_u \times \frac{\theta L}{2} = 4M_p\theta.$$

$$\boxed{W_u = \frac{8M_p}{L}}$$

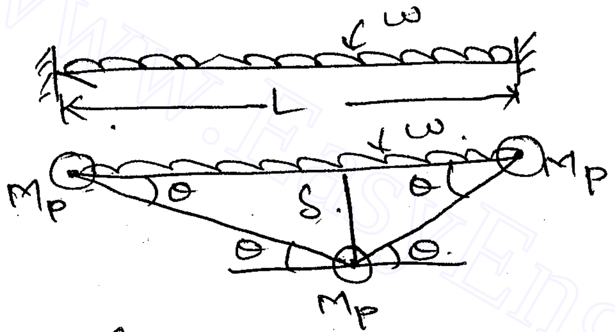
Lower Bound Theorm.



$$\frac{\omega l}{4} = 2MP.$$

$$\boxed{\omega_u = \frac{8M_p}{l}}$$

8.)



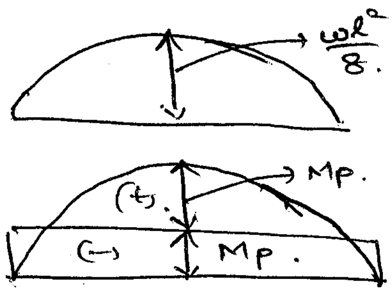
$$\delta = \frac{\theta \times L}{2}$$

EWD = IES.

$$\omega_u \times L \times \frac{\delta}{2} = M_p\theta + M_p(\theta+\theta) + M_p\theta.$$

$$\omega_u \times \frac{L}{2} \times \frac{\theta L}{2} = 4M_p\theta.$$

$$\boxed{\omega_u = \frac{16M_p}{L^2}}$$



$$\frac{\omega_u l^2}{8} = 2M_p.$$

$$\boxed{\omega_u = \frac{16M_p}{l^2}}$$

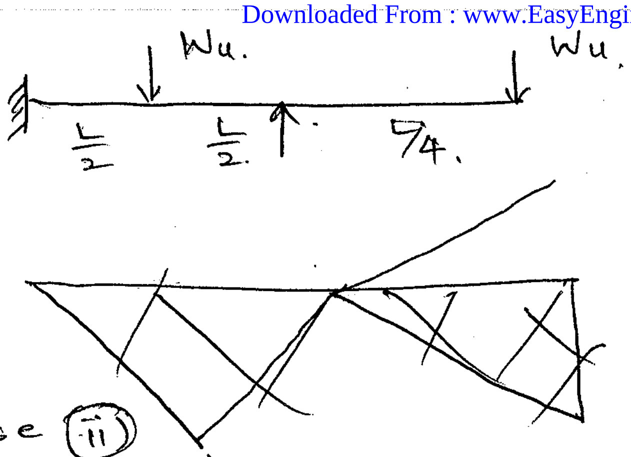
kN/m.

$$W = \omega_u \times l.$$

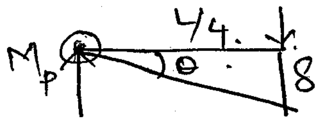
$$\boxed{W = \frac{16M_p}{l}}$$

kN.

9.)



Case (i).



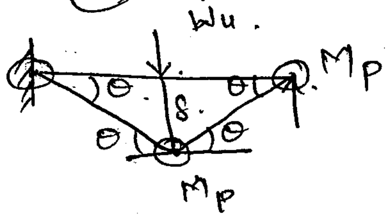
$$\boxed{\delta = \theta \frac{L}{4}}$$

$$W_u \times \delta = M_p \theta.$$

$$W_u \times \cancel{\theta} \frac{L}{4} = M_p \cancel{\theta}$$

$$\boxed{W_u = \frac{4 M_p}{l}}$$

case (ii)



$$\boxed{\delta = \theta \times \frac{L}{2}.}$$

$$W_u \times \cancel{\theta} \times \frac{L}{2} = 4 M_p \cancel{\theta}$$

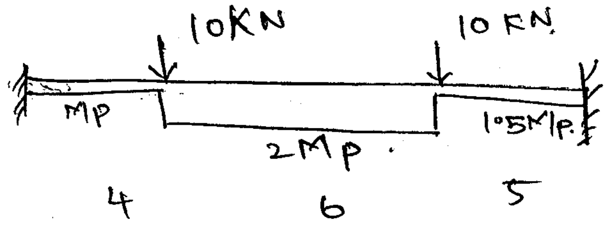
$$\boxed{W_u = \frac{8 M_p.}{L}}$$

Total Load.

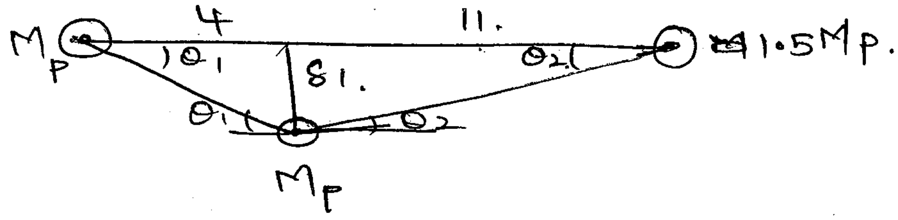
$$W_u = \frac{8 M_p}{L} + \frac{4 M_p}{L}$$

$$\boxed{W_u = \frac{12 M_p}{L}}$$

10.)



Case 1.



$\delta_1 = 4\theta_1 = 11\theta_2$

$$\boxed{\theta_1 = \frac{11}{4}\theta_2}$$

$\therefore$ EWD = IES.

$$10 \times \frac{11\theta_2}{4} = M_p\theta_1 + M_p\theta_1 + M_p\theta_2 + 1.5M_p\theta_2$$

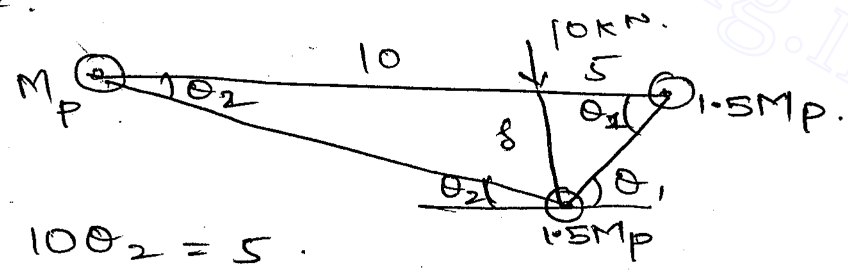
$$\frac{110\,\theta_2}{4} = 2M_p\theta_1 + 2.5M_p\theta_2$$

$$\frac{110}{4}\theta_2 = 2 \times \frac{11}{4}\theta_2 M_p + 2.5M_p\theta_2$$

$$\frac{110}{4}\cancel{\theta_2} = M_p\cancel{\theta_2}\left[\frac{22}{4} + 2.5\right]$$

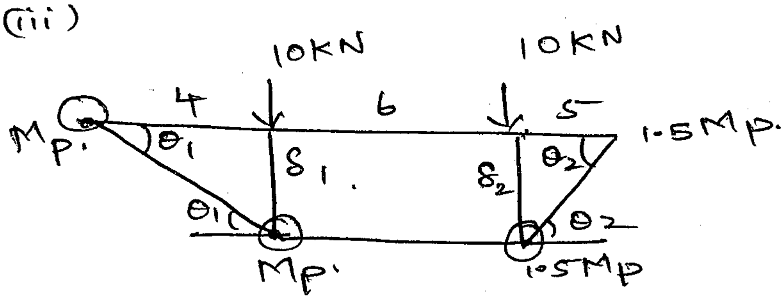
$$\boxed{M_p = 13.75 \text{ KN·m}}$$

Case 2:



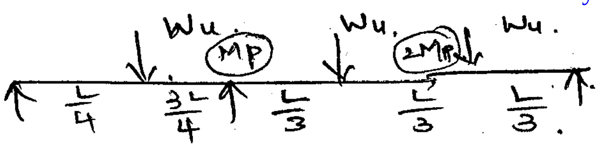
$\delta = 10\theta_2 = 5$.

Case (iii)

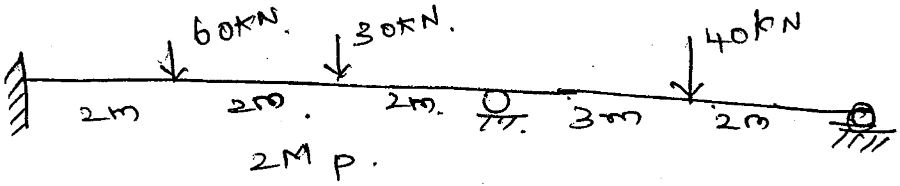


$\delta_1 = 4\theta_1$ $\qquad$ $\delta_2 = 5\theta_2$.

11.)



12.)

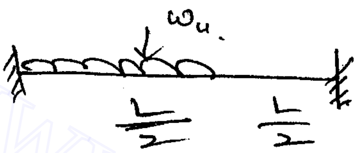


Determine Elastic Section modulus.

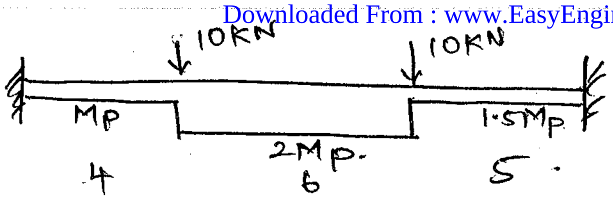
Take S.F = 1.2   F.O.S = 1.5, f_y = 250 MPa.

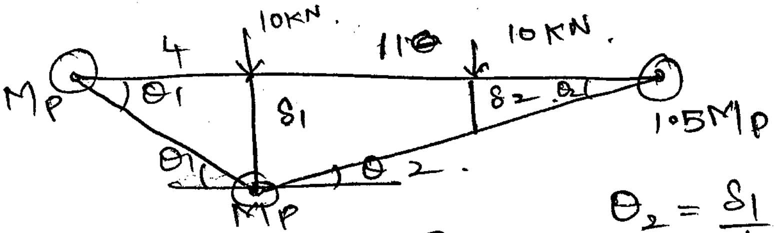
All given loads are working loads.

13.)



10.)





$\delta_1 = 4\theta_1 = 11\theta_2.$

$$\boxed{\theta_1 = \frac{11\theta_2}{4}}$$

$\theta_2 = \frac{\delta_1}{11} = \frac{\delta_2}{5}.$

$$\boxed{\delta_1 = \frac{11\,\delta_2}{5}}$$

EWD = IES.

$(W_{u1} \times \delta_1) + (W_{u2} \times \delta_2) = 2M_p\theta_1 + 2.5M_p\theta_2$

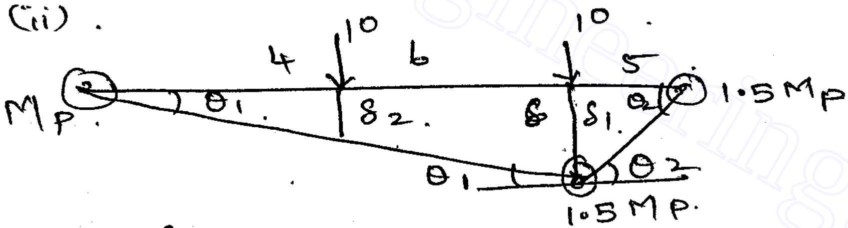
$\cancel{10 \times \frac{11\theta_2}{4}}\ (10 \times 11\,\theta_2) + 10 \times \frac{5\delta_1}{11} = 2M_p\frac{11}{4}\theta_2 + 2.5M_p\theta_2$

$110\,\theta_2 + \frac{\overset{25}{\cancel{50}}}{\cancel{4}}\frac{11\theta_2}{\cancel{4}\,2} = M_p\theta_2\left[\frac{11}{2} + 2.5\right].$

$122.5\cancel{\theta_2} = M_p\cancel{\theta_2}\,8.$

$$\boxed{M_p = 15.3125\ \text{kN.m}}$$

case (ii).



$\delta_1 = 5\theta_2 = 10\theta_1.$

$$\boxed{\theta_1 = \frac{\theta_2}{2}}$$

$\delta_2 = \frac{\delta_1}{10} \times 4.$

$$\boxed{\delta_2 = \frac{2\delta_1}{5}}$$

$(10 \times \delta_1) + (10 \times \delta_2) = 2.5M_p\theta_1 + 3M_p\theta_2.$

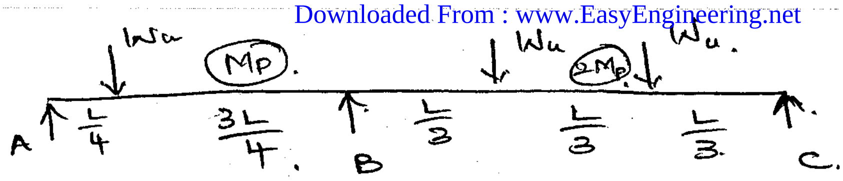
$(10 \times 5\theta_2) + \left(10 \times \frac{2}{\cancel{5}} \times \cancel{5}\theta_2\right) = 2.5\,M_p\frac{\theta_2}{2} + 3M_p\theta_2.$

$70\,\cancel{\theta_2} = 4.25\,M_p\,\cancel{\theta_2}.$

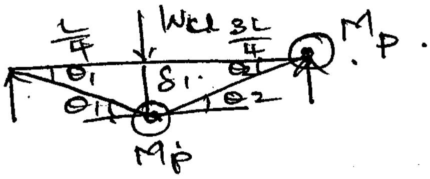
$$\boxed{M_p = 16.47\ \text{kN.m}}$$

$M_p = 16.47$ kN.m.
$2M_p = 32.94$ kN.m
$1.5M_p = 24.705$ kN.m

11.)



Beam AB.



$$\delta_1 = \frac{L}{4}\theta_1 = \frac{3L\theta_2}{4}$$

$$\boxed{\theta_1 = 3\theta_2}$$

$$W_u \times \delta_1 = M_p\theta_1 + 2M_p\theta_2$$

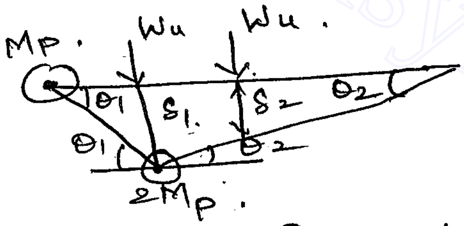
$$W_u \times \frac{3L\theta_2}{4} = M_p 3\theta_2 + 2M_p\theta_2$$

$$3 \cdot \frac{W_u L \cancel{\theta_2}}{4} = 5M_p\cancel{\theta_2}$$

$$\boxed{M_p = \frac{3W_uL}{20}}$$

$$\boxed{\begin{aligned} M_p &= \frac{3W_uL}{20} \\ 2M_p &= \frac{3W_uL}{10} \end{aligned}}$$

Beam BC.



$$\delta_1 = \frac{L}{3}\theta_1 = \frac{2L}{3}\theta_2$$

$$\boxed{\theta_1 = 2\theta_2}$$

$$\delta_2 = \frac{\delta_1 \, L/3}{2L/3}$$

$$\boxed{\delta_2 = \delta_1/2}$$

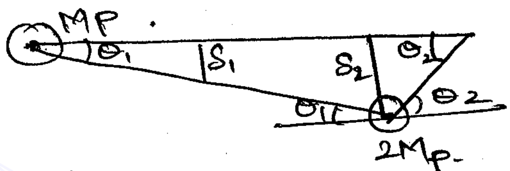
$$(W_u \times \delta_1) + (W_u \times \delta_2) = 3M_p\theta_1 + 2M_p\theta_2$$

$$W_u \frac{2L}{3}\theta_2 + W_u \frac{\delta_1}{2} = 6M_p\theta_2 + 2M_p\theta_2$$

$$\frac{2L}{3} W_u \theta_2 + \frac{2L}{3} \cdot \frac{\theta_2}{2} = 8M_p\theta_2$$

$$\frac{\cancel{2}L}{\cancel{3}} W_u \theta_2/2 \left[\frac{\cancel{3}}{\cancel{2}}\right] = 8M_p\cancel{\theta_2}$$

$$\boxed{M_p = \frac{W_uL}{8}}$$



$$\delta_2 = \frac{2L}{3}\theta_1 = \frac{L}{3}\theta_2$$

$$\boxed{2\theta_1 = \theta_2}$$

$$\delta_1 = \frac{\delta_2 \, L/3}{2L/3}$$

$$\boxed{\delta_1 = \frac{\delta_2}{2}}$$

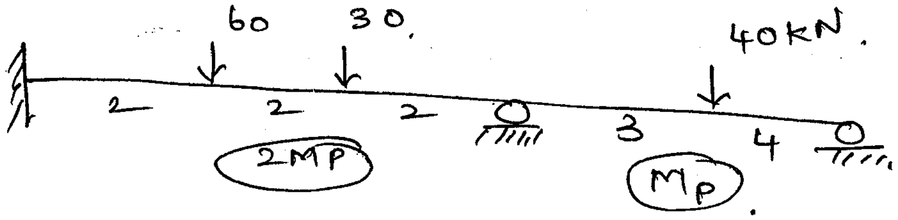
$$(W_u \times \delta_1) + (W_u \times \delta_2) = 3M_p\theta_1 + 2M_p\theta_2$$

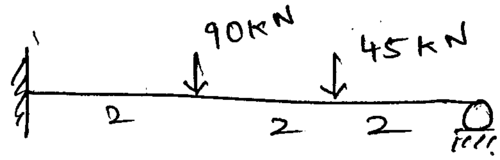
$$\frac{W_u}{2}\frac{2L}{3}\theta_1 + W_u \cdot \frac{2L}{3}\theta_1 = 3M_p\theta_1 + 4M_p\theta_1$$

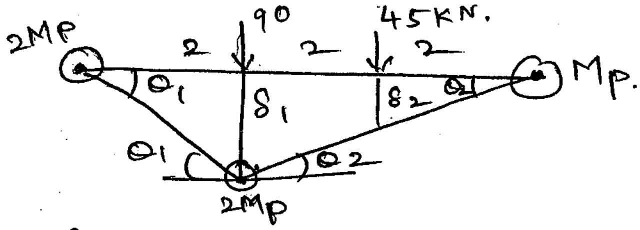
$$\frac{\cancel{2}L}{\cancel{3}} W_u \cancel{\theta_1}\left(\frac{3}{2}\right) = 7M_p\theta_1$$

$$\boxed{M_p = \frac{W_uL}{7}}$$

12)







$\delta_1 = 2\theta_1 = 4\theta_2.$

$\boxed{\theta_1 = 2\theta_2}$

$\delta_2 = \frac{2\delta_1}{4}$

$\boxed{\delta_2 = \frac{\delta_1}{2}}$

$(90\times\delta_1) + (45\times\delta_2)$

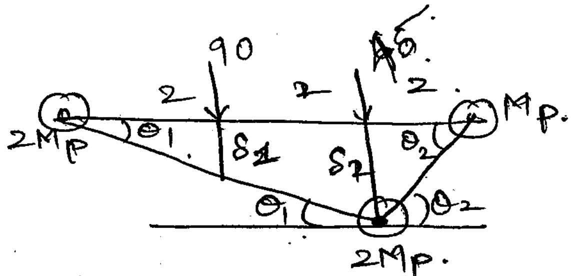
$= 4M_p\theta_1 + 3M_p\theta_2.$

$(90\times 4\theta_2) + \left(\frac{45}{2}\times 4\theta_2\right)$

$= 11M_p\theta_2.$

$\cancel{360} \; 450\,\theta_2 = 11 M_p\theta_2$

$\boxed{M_p = 40.91 \text{ kN·m}}$



$\delta_2 = 2\theta_2 = 4\theta_1.$

$\boxed{\theta_2 = 2\theta_1}$

$\boxed{\frac{\delta_2}{2} = \delta_1}$

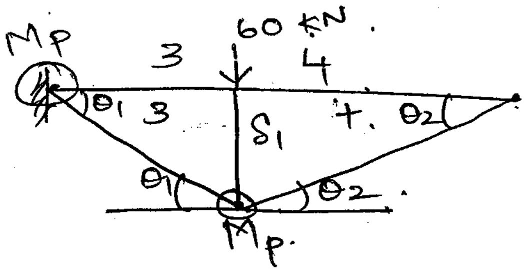
$(90\times\delta_1) + (45\times\delta_2)$

$= 4M_p\theta_1 + 3M_p\theta_2.$

$(45\times 4\theta_1) + (45\times 4\theta_1)$

$= 10M_p\theta_1$

$\boxed{M_p = 36 \text{ kN·m}}$



$\delta_1 = 3\theta_1 = 4\theta_2.$

$\boxed{\theta_1 = \frac{4}{3}\theta_2}$

$60\times\delta_1 = 2M_p\theta_1 + M_p\theta_2.$

$60\times 4\theta_2 = \frac{2\times 4}{3}M_p\theta_2 + M_p\theta_2.$

$\boxed{M_p = 65.45 \text{ kN·m}}$

$M_p = 65.45 \text{ kN·m}.$

$\cancel{Z_{pe}\ f_y}\ \frac{M}{I} = \frac{f}{y}.$

$\frac{M}{f} = \frac{I}{y} = Z.$

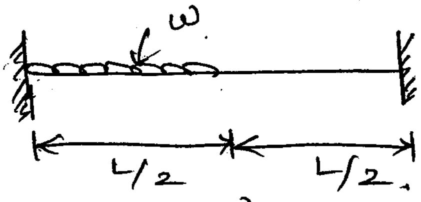
$Z_p = \frac{M_p}{f_y} = \frac{65.45\times 10^6}{250}.$

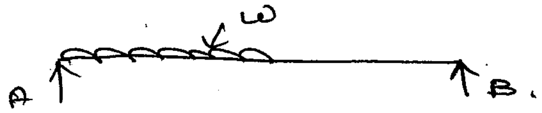
$\boxed{Z_p = 261.8\times 10^3 \text{ mm}^3}$

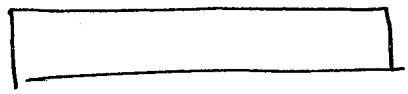
$Z_e = \frac{Z_p}{S.F} = \frac{261.8\times 10^3}{1.2}.$

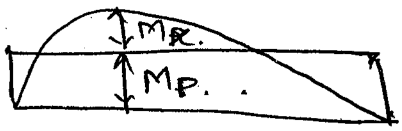
$\boxed{Z_e = 218.17\times 10^3 \text{ mm}^3}$

13.)









$\Sigma M_A = 0.$

$R_B \times l = \frac{w_c l^2}{8}.$

$R_B = \frac{w_c l}{8}.$

$R_A = \frac{w_c l}{2} - \frac{w_c l}{8}.$

$\boxed{R_A = \frac{3 w_c l}{8}}$

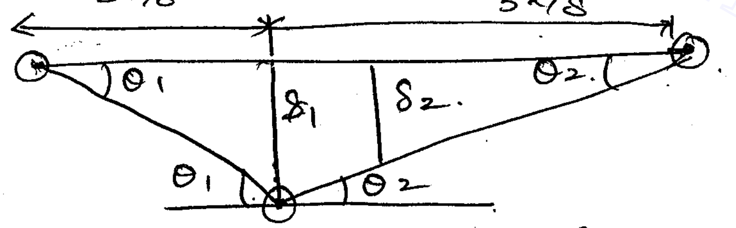
$V_{xx} = 0.$

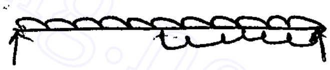
$\frac{3 w_c l}{8} - w_c x = 0.$

$w_c x = \frac{3 w_c l}{8}$

$\boxed{x = \frac{3l}{8}}$ where B.M is max.

$B.M_{max} = \frac{3 w_c l}{8} \times \frac{3l}{8} - w_c \frac{3l}{8} \times \frac{3l}{16}.$





$\delta_1 = \frac{3l}{8}\theta_1 = \frac{5l}{8}\theta_2$

$\boxed{\theta_1 = \frac{5\theta_2}{3}}$

EWD = I.E.S.

$\left(w_c \times L \times \frac{\delta_1}{2}\right) - \left(w_c \times \frac{L}{2} \times \frac{\delta_2}{2}\right) = 2M_p\theta_1 + 2M_p\theta_2.$

$\delta_2 = \frac{\delta_1}{\frac{5l}{8}} \times \frac{l}{2}$

$\boxed{\delta_2 = \frac{4\delta_1}{5}}$

$w_c \times \frac{L}{2} \times \frac{5l\theta_2}{8} - \frac{w_c L}{4}\left(\frac{4\delta_1}{5}\right) = 2M_p \frac{5\theta_2}{3} + 2M_p\theta_2.$

$\frac{5}{16} w_c l^2 \theta_2 - \frac{w_c L}{5} \cdot \frac{5l}{8}\theta_2 = \left(\frac{10}{3} + 2\right) M_p \theta_2$

$\left(\frac{5 w_c l^2}{16} - \frac{2 w_c l^2}{8}\right)\theta_2 = \frac{16}{3} M_p \theta_2$

$\frac{3 w_c l^2}{16} = \frac{16}{3} M_p.$

$M_p = \frac{3 w_c l^2 \times 3}{256}$

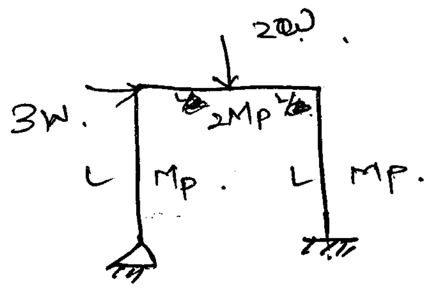
$\boxed{w = \frac{28.4 M_p}{l}}$

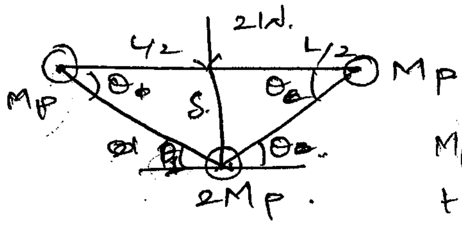
25/12/2015

STEEL.

PLASTIC MOMENT CAPACITY.

1.)





$M_p\theta$
$+ 2M_p(\theta+\theta)$
$+ M_p\theta$
$= 6M_p\theta.$

$\delta = \frac{L}{2}\theta.$

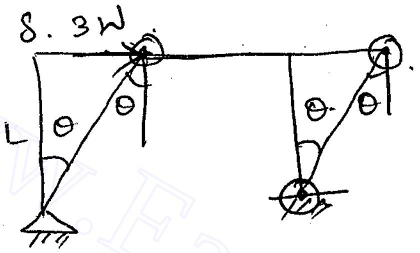
$2W \times \delta = 6M_p\theta$

$2W \times \frac{\theta L}{2} = 6M_p\theta$

$\boxed{M_p = \frac{W_u L}{6}}$

$\boxed{W_u = \frac{6M_p}{L}}$

Sway Mechanism



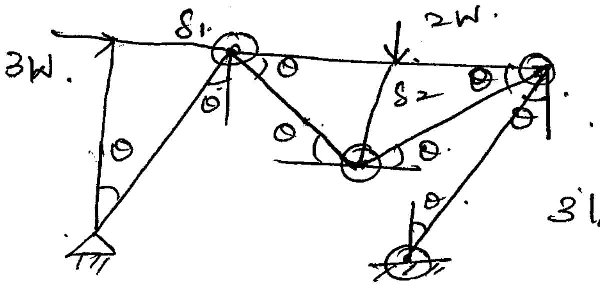
$\delta = \theta L$

$\cancel{3}W \times \delta = \cancel{3}M_p\theta$

$W\cancel{\theta}L = M_p\cancel{\theta}$

$\boxed{\frac{M_p}{L} = W}$

Combined Mechanism



$\delta_1 = LO.$

$\delta_2 = \frac{L\theta}{2}.$

$3W_u\delta_1 + 2W_u\delta_2$

$= M_p\theta + 2M_p(\theta+\theta)$
$+ M_p\theta + M_p\theta.$

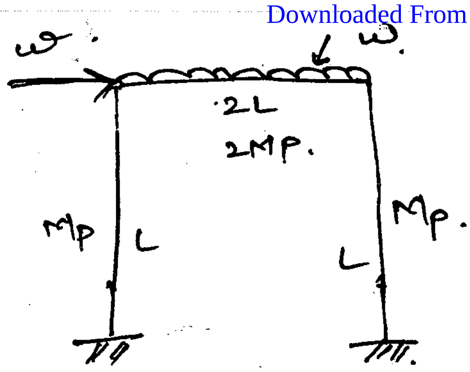
$(3W_u L\theta) + (2W_u L\theta) = 7M_p\theta.$

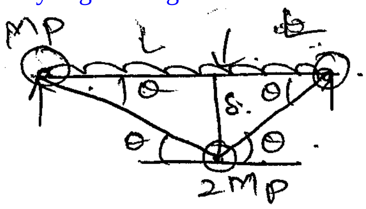
$5W_u L\cancel{\theta} = 7M_p\cancel{\theta}$

$\boxed{W_u = \frac{7M_p}{5L}}$

$\boxed{W_u = \frac{M_p}{L}}$

2.)



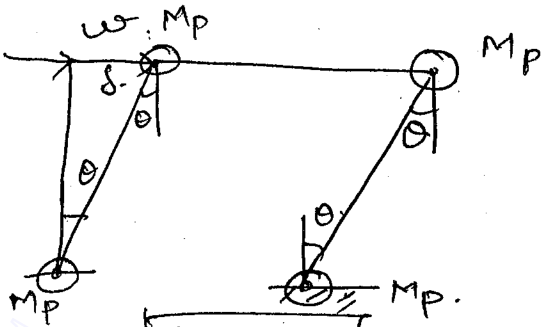


$\delta = l\theta$

$$\cancel{2}\, w \times l \times \frac{\delta}{\cancel{2}} = 6M_p\theta$$

$$w\, l \times l \times \cancel{\theta} = 6M_p\cancel{\theta}$$

$$\boxed{w = \frac{6M_p}{l^2}}$$

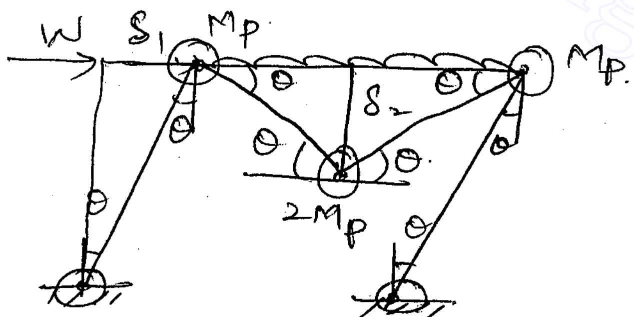


$$\boxed{\delta = L\theta}$$

$$W \times \delta = 4M_p\theta$$

$$W \times L\cancel{\theta} = 4M_p\cancel{\theta}$$

$$\boxed{W = \frac{4M_p}{L}}$$



$\delta_1 = L\theta \qquad \delta_2 = L\theta$

$$W \times \delta_1 + w \times L \times \frac{\delta_2}{2} = M_p\theta + M_p\theta + 4M_p\theta + M_p\theta + M_p\theta$$

$$(W \times L\cancel{\theta}) + w \times L \times \frac{L\cancel{\theta}}{2} = 8M_p\cancel{\theta}$$

$$\frac{2WL}{2} = 8M_p\theta$$

$$\cancel{M_p} = \boxed{W_u = \frac{8M_p}{L}}$$

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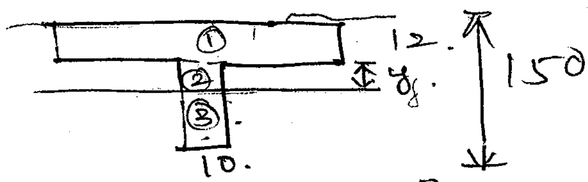
3.)



Equal Area Axis.

$$A_1 + A_2 = A_3.$$

$$(100 \times 12) + (10 \times y) = 10(138 - y)$$

$$1200 + 10y = 1380 - 10y.$$

$$20y = 1380 - 1200.$$

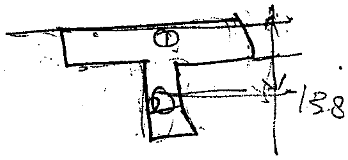
$$\boxed{y = 9\,m.}$$

EAA.

$$Z_p = A_1 y_1 + A_2 y_2 + A_3 y_3.$$

$$= [1200 \times (6+9)] + (10 \times 9 \times 4.5) + (10 \times 129 \times 64.5)$$

$$\boxed{Z_p = 101.61 \times 10^3}$$



Z_e:

$$\bar{y} = \frac{A_1 y_1 + A_2 y_2}{A_1 + A_2}$$

$$= \frac{(100 \times 12 \times 6) + (10 \times 138 \times 81)}{(1200 + 1380)}$$

$$\boxed{\bar{y} = 46.12\,mm.}$$ from top.

$$I_{xx} = I_{x1} + A(\bar{y} - y_1)^2 + I_{x2} + A(\bar{y} - y_2)^2.$$

$$= \frac{100 \times 12^3}{12} + 1200(46.12 - 6)^2 + \frac{10 \times 138^3}{12} + 1380(46.12 - 81)^2$$

$$= 4.858 \times 10^6.$$

$$= 5.8143 \times 10^6\,mm^4$$

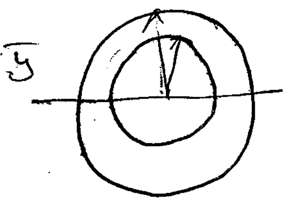
$$Z_e = \frac{5.8143 \times 10^6}{103.88} = 55.97 \times 10^3\,mm^3$$

$$S.F = \frac{Z_p}{Z_e} = \frac{101.61 \times 10^3}{55.97 \times 10^3} = 1.82$$

1.2
1.5
1.7
2
2.38

4) Find S.F. of ring having inner dia. 100mm thickness 10mm.

Z_e



$D = 100 + 2(10)$

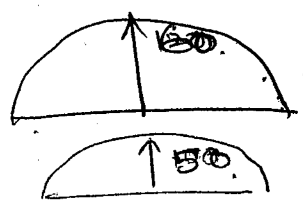
$\boxed{D = 120mm}$

$$Z_e = \frac{I}{y}$$

$$= \frac{\frac{\pi}{64}(120^4 - 100^4)}{120/2}$$

$$Z_e = 87.83 \times 10^3 \ mm^3$$

$$Z_p = 2[A_1 \times y_1 - A_2 \times y_2]$$



$$= 2\left[\left[\frac{\pi r_1^2}{2} \times \frac{4r_1}{3\pi}\right] - \left(\frac{\pi r_2^2}{2} \times \frac{4r_2}{3\pi}\right)\right]$$

$$= 2\left[\left(\frac{\pi \times 60^2}{2} \times \frac{4 \times 60}{3\pi}\right) - \left(\frac{\pi \times 50^2}{2} \times \frac{4 \times 50}{3\pi}\right)\right]$$

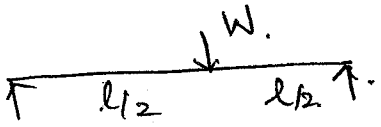
$$= 2\left[60.67 \times 10^3\right]$$

$$Z_p = 121.33 \times 10^3 \ mm^3$$

$$S.F = \frac{Z_p}{Z_e} = \frac{121.33}{87.83} = 1.38$$

STEEL STRUCTURE.

1.)



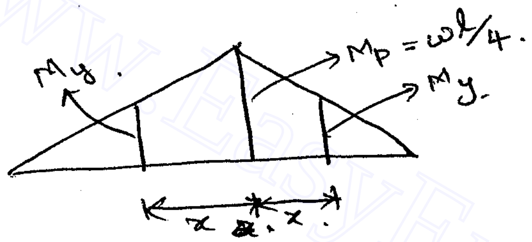
M_p will remain. @ mid span.

$$M_p = f_y Z_p.$$

$$\boxed{M_p = f_y \frac{bd^2}{4}.}$$

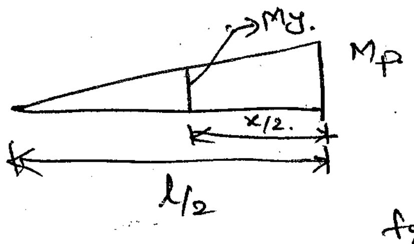
$$M_y = f_y \times Z_e.$$

$$\boxed{M_y = f_y \frac{bd^2}{6}.}$$



$M_p > M_y$.

length of plastic hinge.



$$\frac{M_p}{M_y} = \frac{l/2}{\frac{l}{2} - x}.$$

$$\frac{f_y\, bd^2/4}{f_y \frac{bd^2}{6}} = \frac{l/2}{\frac{l}{2} - x}$$

$$\boxed{x = \frac{l}{6}.}$$

plastic hinge length $= 2 \times x.$

$$= 2 \times \frac{l}{6}$$

$$= l/3.$$

Plastic hinge

when 3 (or) 4 members meet @ point plastic hinge is formed @ in all the member.

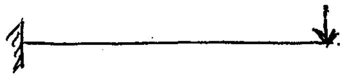
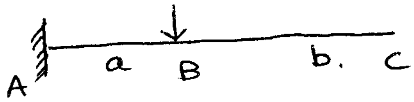
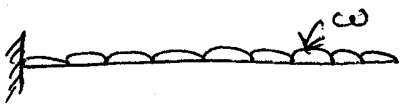
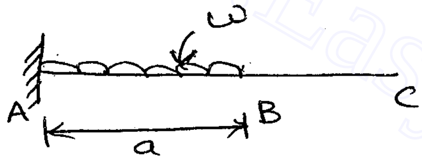
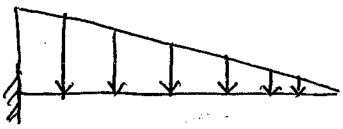
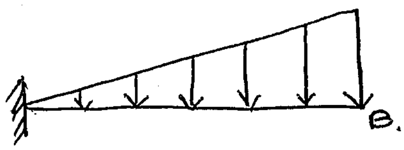
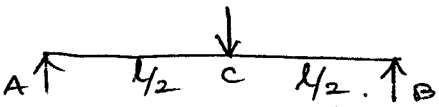
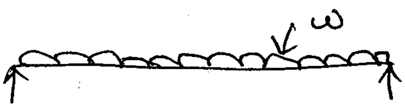
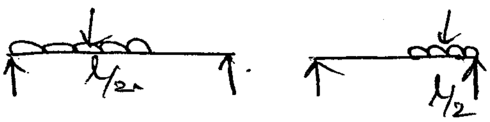
In elastic method of analysis. Equilibrium eqn is used

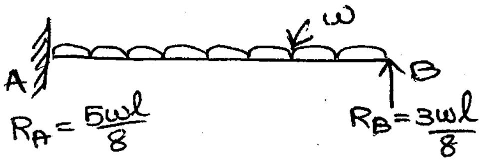
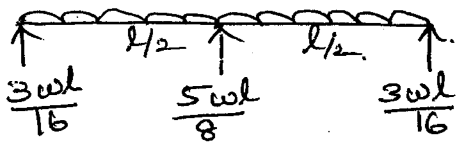
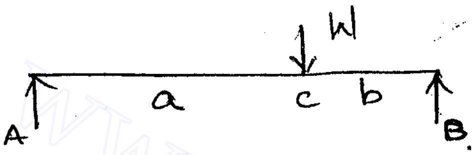
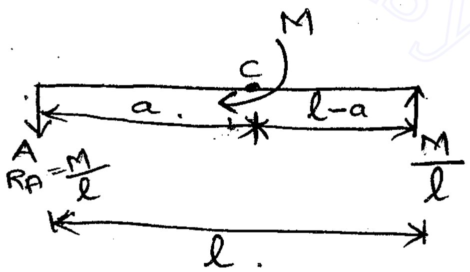
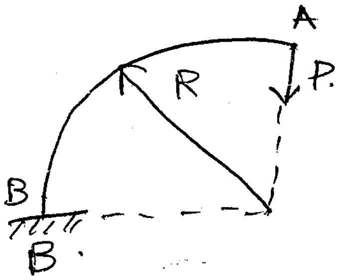
In Plastic method of analysis equilibrium yield and. mechanism is used.

29/12/2015

STRUCTURAL ANALYSIS

Finished

	Beam	θ	δ
1.		$\frac{wl^2}{2EI}$	$\frac{wl^3}{3EI}$
2.)		$\theta_B = \theta_C = \frac{wa^2}{2EI}$	$\delta_B = \frac{wa^3}{3EI}$ $\delta_C = \frac{wa^3}{3EI} + \frac{wa^2}{2EI}(l-a)$
3.)		θ $\frac{wl^3}{6EI}$	$\frac{wl^4}{8EI}$
4.)		$\frac{ML}{EI}$	$\frac{ML^2}{2EI}$
5.)		$\theta_B = \theta_C = \frac{wa^3}{6EI}$	$\delta_B = \frac{wa^4}{8EI}$ $\delta_C = \frac{wa^4}{8EI} + \frac{wa^3}{6EI}(l-a)$
6.)		$\theta_B = \frac{wl^3}{24EI}$	$\delta_B = \frac{wl^4}{30EI}$.
7.)		$\theta_B = \frac{wl^3}{8EI}$	$\delta_B = \frac{11}{120} \frac{wl^4}{EI}$
8.)		$\theta_A = \theta_B = \frac{wl^2}{16EI}$	$\delta_C = \frac{wl^3}{48EI}$.
9.)		$\theta_A = \theta_B = \frac{wl^3}{24EI}$	$\delta_C = \frac{5}{384} \frac{wl^4}{EI}$
10.)		—	$\delta_C = \frac{1}{2}\left(\frac{5}{384} \frac{wl^4}{EI}\right)$

Beam	θ	δ
		δ_{max} @ 0.421L from B $\delta_{max} = 0.005415 \frac{wl^4}{EI}$
		δ_{max} @ 0.21 l from end support. $\delta_{max} = 3.85 \times 10^{-4} \frac{wl^4}{EI}$
		$\delta_c = \frac{wa^2b^2}{3EI}$. δ_{max} @ $x = \sqrt{\frac{l^2-b^2}{3}}$ from A. $\delta_{max} = \frac{wb(a^2+2ab)^{3/2}}{9\sqrt{3}\,l\,EI}$.
	$\theta_A = \frac{M}{6EIl}(2l^2 - 6al + 3a^2)$ $\theta_B = \frac{M}{EIl}(3a^2 - l^2)$ when $a = b = l/2$. $\theta_B = \theta_A = \frac{Ml}{24EI}$.	$\delta_c = \frac{Ma(l-a)(l-2a)}{3EIl}$. $\delta_c = 0$.
		$\delta_V = \frac{P\cancel{R}\pi R^3}{4EI}$ $\delta_H = \frac{PR^3}{2EI}$.

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6/1/2016

STRUCTURAL ANALYSIS

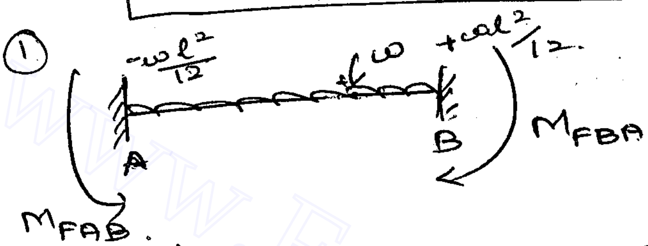
SLOPE AND DEFLECTION METHOD

Generalized equation.

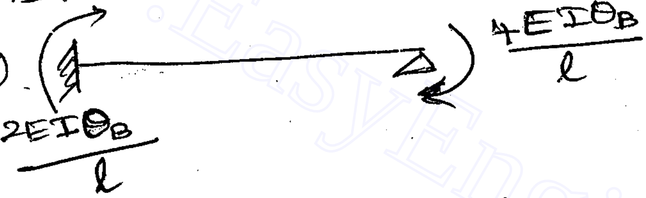
$$M_{AB} = M_{FAB} + \frac{2EI}{l}\left(2\theta_A + \theta_B - \frac{3\delta}{l}\right)$$

$$M_{BA} = M_{FBA} + \frac{2EI}{l}\left[\theta_A + 2\theta_B - \frac{3\delta}{l}\right]$$

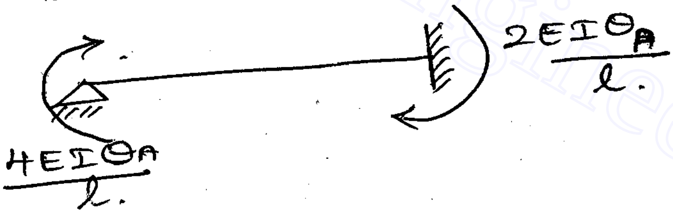
(1)



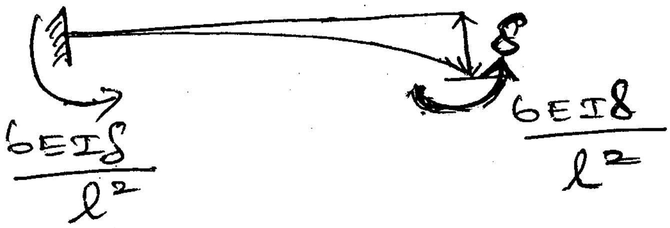
(2)



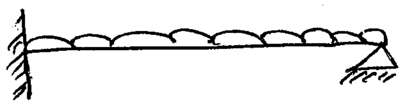
3.)

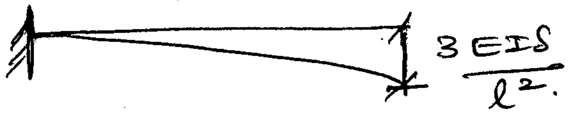


4.)



one End Fixed and other end hinged.





$$M_{AB} = M_{FAB} - \frac{3EI\delta}{l^2} + \frac{3EI\theta_A}{l}$$

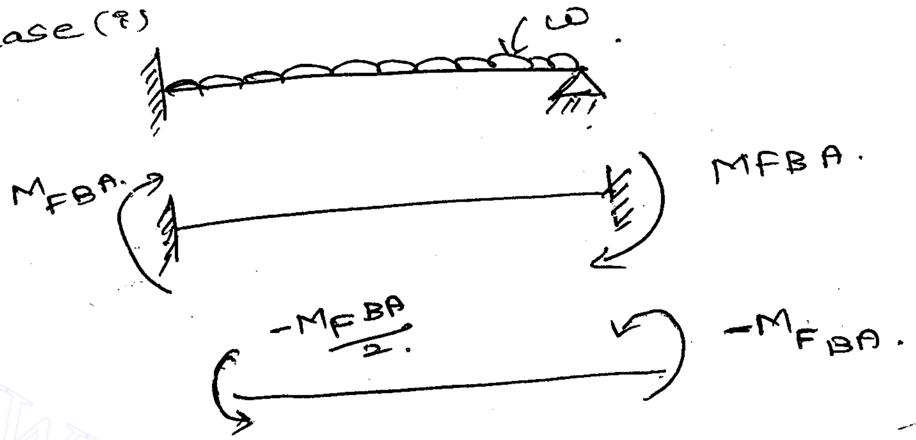
$$\boxed{M_{AB} = M_{FAB} + \frac{3EI}{l}\left[\theta_A - \frac{\delta}{l}\right]}$$

$$\boxed{M_{BA} = 0.}$$

Explanation:

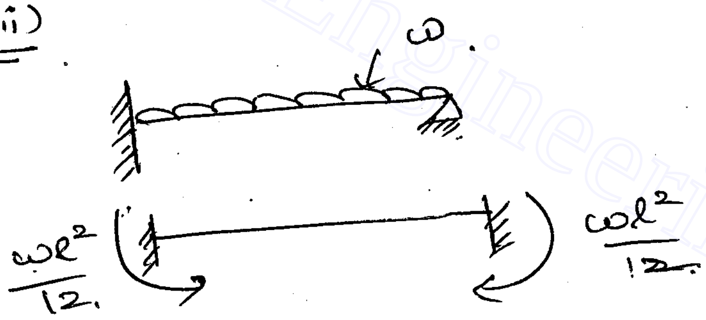
Determine the fix end moment @ A in case of end being hinged from the fixed end moments. Corresponding to the case of both end fixed.

case (i)



$$\left.\begin{array}{l}\text{Fixed End Moment} \\ \text{@ A}\end{array}\right\} = M_{FAB} - \frac{M_{FBA}}{2}$$

case (ii)



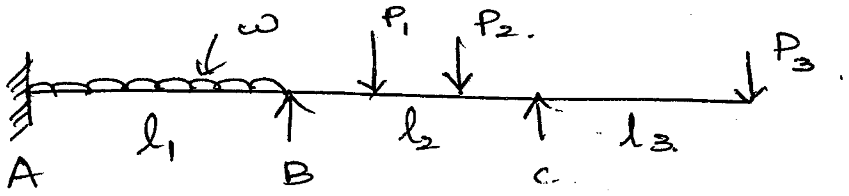
$\cancel{M_{FAB}}$

$$= M_{FAB} - \frac{M_{FBA}}{2}$$

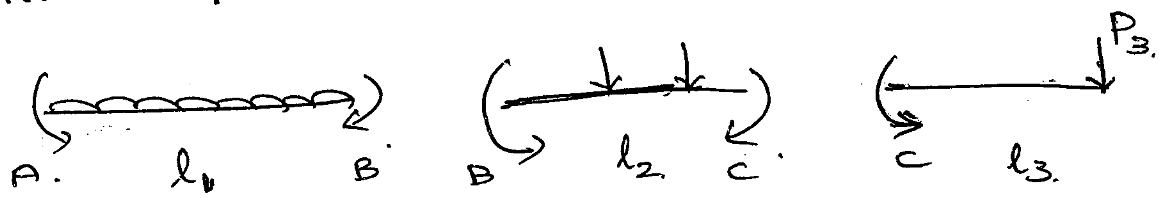
$$= -\frac{\omega l^2}{12} - \frac{\omega l^2}{24}$$

$$\left.\begin{array}{l}\text{Fixed end} \\ \text{moment @ A}\end{array}\right\} = \frac{\omega l^2}{8}$$

Method to find out. Find End Moment.



Joint Equilibrium Equation.



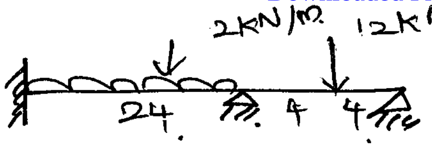
$$\boxed{\begin{array}{l} M_{BA} + M_{BC} = 0. \\ M_{CB} - P_3 l_3 = 0 \end{array}}$$

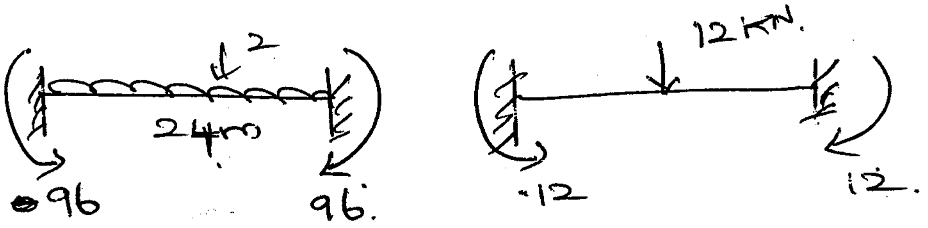
Shear Equation:

$$\Sigma V = 0.$$

$$R_A + R_B + R_C - [\omega l + P_1 + P_2 + P_3] = 0.$$

$$\boxed{R_A + R_B + R_C = [\omega l + P_1 + P_2 + P_3]}$$

1.) 



$M_{FAB} = -96 \text{ kN·m}$ $\quad M_{fBC} = -12 \text{ kN·m}$

$M_{fBA} = 96 \text{ kN·m}$ $\quad M_{fCB} = 12 \text{ kN·m}$

$$M_{AB} = M_{fAB} + \frac{2EI}{l}\left[2\cancel{\theta_A}^{0} + \theta_B - \frac{3\cancel{\delta}^{0}}{\cancel{l}}\right]$$

$$M_{AB} = -96 + \frac{2EI}{24}\theta_B.$$

$$M_{BA} = M_{fBA} + \frac{2EI}{l}\left[\cancel{\theta_A}^{0} + 2\theta_B - \frac{3\cancel{\delta}^{0}}{l}\right]$$

$$M_{BA} = 96 + \frac{4EI\theta_B}{l}.$$

$$M_{BC} = M_{fCB} + \frac{2EI}{l}\left[2\theta_B + \theta_C - \frac{\cancel{3\delta}^{0}}{l}\right]$$

$$M_{BC} = -12 + \frac{4EI\theta_B}{l} + \frac{2EI\theta_C}{l}.$$

$$\boxed{M_{CB}} = 12 + \frac{2EI}{l}\left[\theta_B + 2\theta_C - \frac{3\cancel{\delta}^{0}}{l}\right].$$

$$= \frac{2EI\theta_B}{l.} + \frac{4EI\theta_C}{l.} + 12.$$

Joint Equilibrium Equation:

$$M_{BA} + M_{BC} = 0.$$

$$84. + \frac{4EI\theta_B}{24} + \frac{4EI\theta_B}{8} + \frac{2EI\theta_C}{8} = 0.$$

$$84 + \frac{16EI\theta_B + 6EI\theta_C}{24} = 0.$$

$$16 EI\theta_B + 6EI\theta_C = -84 \times 24.$$

$$8EI\theta_B + 3EI\theta_C = \frac{-84 \times 24}{2}.$$

$$8\theta_B + 3\theta_C = -\frac{1008}{EI} \quad \text{--- (1)}.$$

$M_{CB} = 0.$

$$\frac{2EI\theta_B}{8} + \frac{4EI\theta_C}{8} = -12$$

$$2\theta_B + 4\theta_C = \frac{-12 \times 8}{EI}.$$

$$\theta_B + 2\theta_C = \frac{-12 \times 8}{2 \times EI}.$$

$$\theta_B + 2\theta_C = \frac{-48}{EI} \quad \text{--- (2)}.$$

$$\boxed{\theta_B = -\frac{144}{EI} \qquad \theta_C = 48/EI}$$

$$M_{AB} = -96 + \frac{2EI}{24} \times \frac{-144}{EI}$$

$$= -108 \text{ kN.m}$$

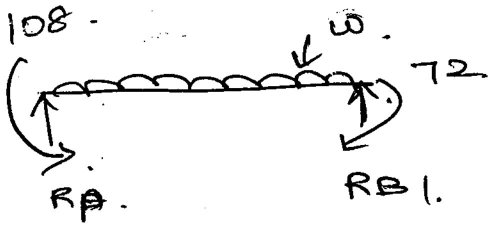
$$M_{BA} = 96 + \frac{4EI}{24} \times \left(-\frac{144}{EI}\right).$$

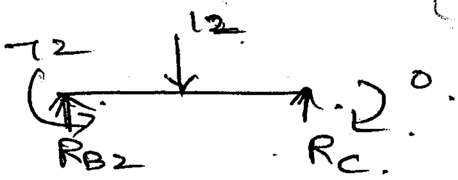
$$= 72 \text{ kN.m}.$$

$$M_{BC} = -12 + \frac{4 \times EI}{8}\left(\frac{-144}{EI}\right) + \frac{2 \times EI}{8}\left(\frac{48}{EI}\right)$$

$$= -72 \text{ kN.m}$$

$$\boxed{M_{CB} = 0.}$$





$\Sigma M_A = 0$.

$R_B \times 24 - \left(\frac{2\times24^2}{2}\right) - 72 + 108 = 0$.

$\boxed{R_{B_1} = 22.5}$

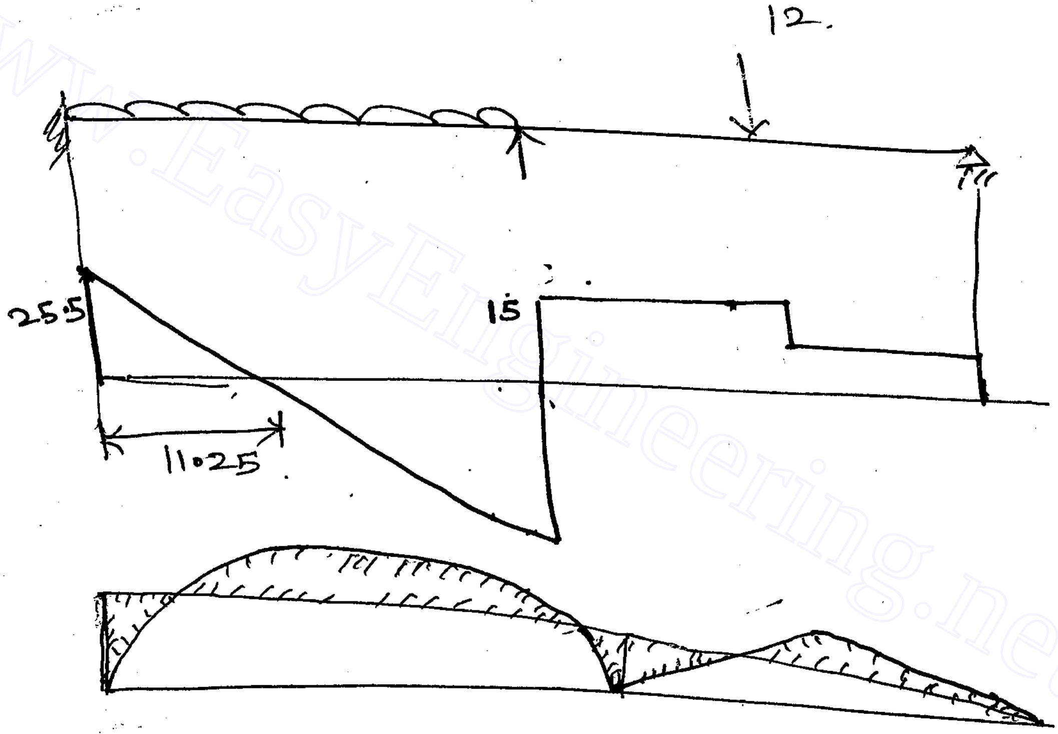
$R_A = 25.5 \text{ kN}$.

$\Sigma M_C = 0$.

$R_{B_2} \times 8 - 72 - (12\times4) = 0$.

$\boxed{R_{B2} = 15}$ kN

$R_C = -3 \text{ kN}$



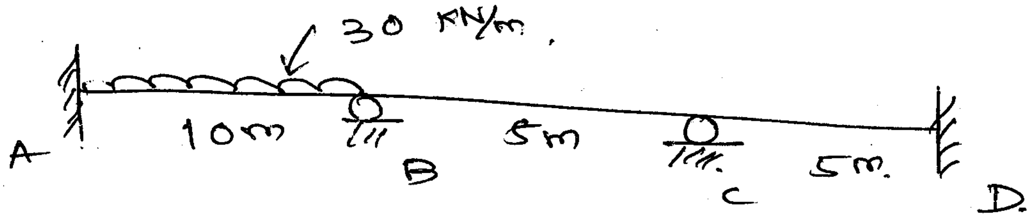
V.F $= 0$.

$R_A = 2\times x$.

$\frac{22.5}{2} = x$

$x = 11.25 \text{ m}$.

2)



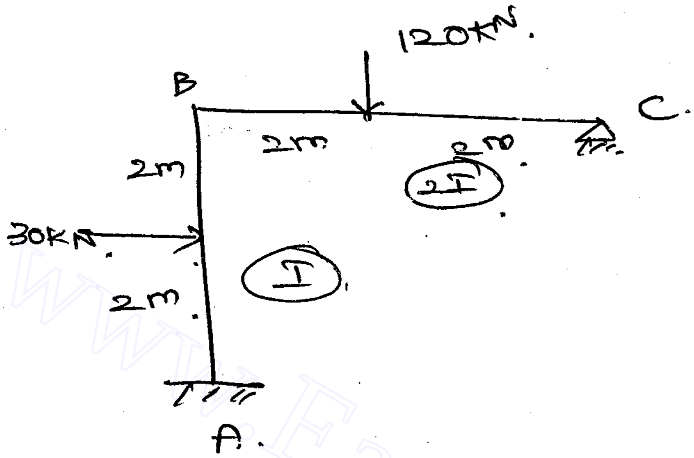
$E = 2 \times 10^5 \ N/mm^2$ $\qquad I = 60 \times 10^6 \ mm^4$

$\delta_B = 30cm$

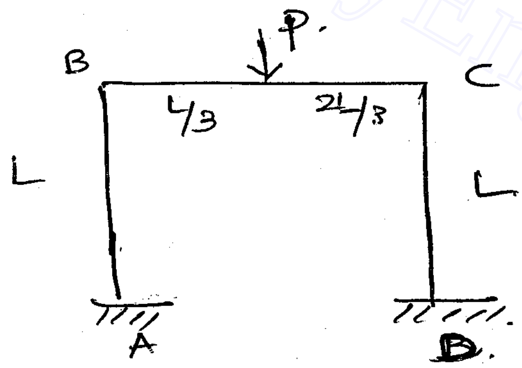
$\delta l_B = +30$

$\delta l_{CD} = -30$

3)



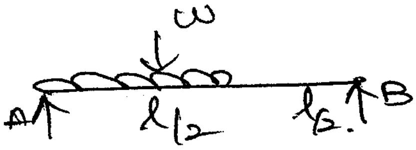
4)



9/1/2016

Structural Analysis.

1.)

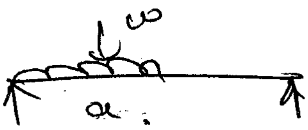


$~~\theta_A = \frac{3}{128} wl^3~~$

$$\theta_A = \frac{3}{128}\frac{wl^3}{EI}.$$

$$\theta_B = \frac{1}{384}\frac{wl^3}{EI}.$$

2.)



$$\theta_A = \frac{wa^2}{24EI}(2l-a)^2.$$

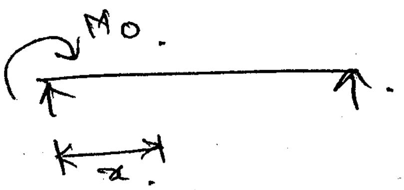
$$\theta_B = \frac{wa^2}{24EI}(2l^2-a^2).$$

3.)

$$\theta_A = \theta_B = \frac{wa(l-a)}{2EI}.$$

$$\delta_c = \delta_{max} = \frac{wa}{24EI}(3l^2-4a^2).$$

4.)

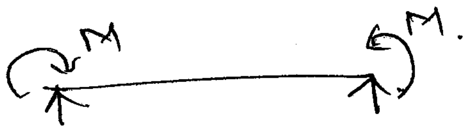


$$\theta_A = \frac{ML}{3EI} \qquad \theta_B = \frac{ML}{6EI}.$$

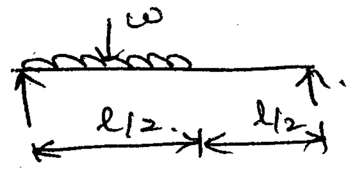
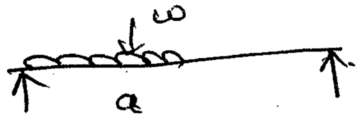
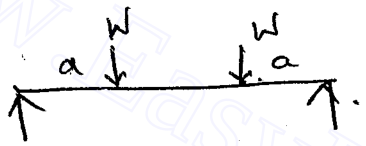
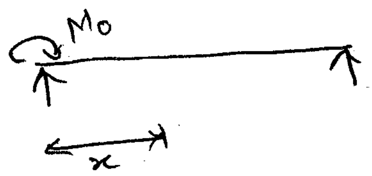
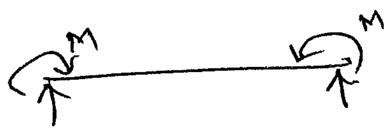
$$@\ x = 0.42 \qquad \delta_{max} = \frac{ML^2}{9\sqrt{3}EI}.$$

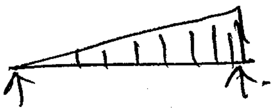
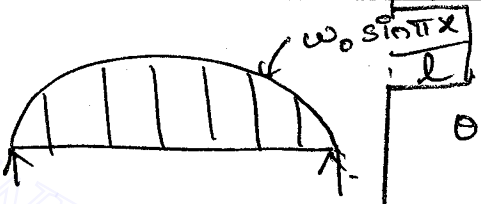
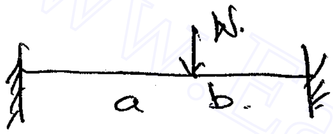
5.)

5.)



$$\theta_A = \theta_B = \frac{ML}{2EI} \qquad \delta_c = \frac{ML^2}{8EI}$$

S.NO	Beam	θ.	δ.
1.		$\theta_A = \frac{3}{128} \cdot \frac{wl^3}{EI}$ $\theta_B = \frac{7}{384} \frac{wl^3}{EI}$	
2.)		$\theta_A = \frac{wa^2}{24EI}(2l-a)^2$ $\theta_B = \frac{wa^2}{24EI}(2l^2-a^2)$	
3.)		$\theta_A = \theta_B = \frac{wa(l-a)}{2EI}$	$\delta_C = \delta_{max} =$ $= \frac{wa}{24EI}(3l^2-4a^2)$
4.)		$\theta_A = \frac{ML}{3EI}$ $\theta_B = \frac{ML}{6EI}$	@ $x = 0.42$ $\delta_{max} = \frac{ML^2}{9\sqrt{3}EI}$
5.)		$\theta_A = \theta_B = \frac{ML}{2EI}$	$\delta_C = \frac{ML^2}{8EI}$

6)		$\theta_A = \frac{7}{360} \cdot \frac{wl^3}{EI}$. $\theta_B = \frac{wl^3}{45EI}$	$\delta_C = \frac{5}{768} \frac{wl^4}{EI}$. δ_{max} @ $0.519L$ $\delta_{max} = 0.00652 \frac{wl^4}{EI}$
7.)		$\theta_A = \theta_B = \frac{5}{192} \cdot \frac{wl^3}{EI}$	$\delta_C = \delta_{max.} = \frac{wl^4}{120EI}$.
8.)		$\theta_A = \theta_B = \frac{w_o l^3}{\pi^3 EI}$	$\delta_c = \delta_{max} = \frac{w_o l^4}{\pi^4 EI}$.
9.			$\delta = \frac{wa^3b^3}{3EIl^3}$.

A rectangular beam of section is simply supported on a span "l" and carries a concentrated ~~load~~. load "W" at its centre

Strain energy due to shear $U = \frac{3}{20} \frac{w^2 l}{Gbd}$.

The deflection due to shear @ midspan is given by.

$U = \frac{3}{20}$.

$$\boxed{U = \frac{1}{2} \times W \times \delta c.}$$

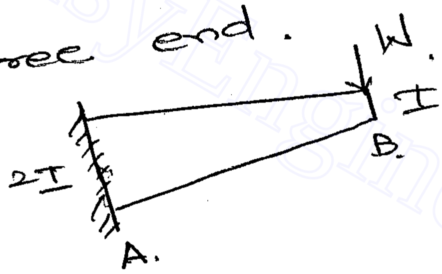
$$\boxed{\delta_c = \frac{3}{10} \cdot \frac{WL}{Gbd}}$$

A Horizontal beam rests on two supports on the same level and carries a udl "ω". The supports are symmetrically placed. In order to greatest downward deflection @ its centre may have a least value, the deflection @ its ends and @ the ends must be equal.

The position of support should be such that the distance b/w support = 0.554 x total beam length

In order to have both the supports @ same level @ horizontal position, the slope on each end must be zero.
The distance b/w the supports for above condition must be such that
0.5774 x total beam length

2) A cantilever beam fixed @ A and free @ B. changing moment of Inertia from I (B) to 2I (A). carrying a concentrated ~~f~~ load W @ free end.



$$\delta_B = \frac{WL^3}{8EI}\left(\log_e 2 - 0.5\right)$$

26/12/2015 Finished

SURVEYING

LATITUDE AND DEPARTURE

1\. * In a traverse, latitude and departure of the sides are calculated. It was calculated that $\Sigma L = 1.39$, $\Sigma D = -2.17$, calculate the length of bearing and closing error.

$$e = \sqrt{\Sigma L^2 + \Sigma D^2}$$

Solution:

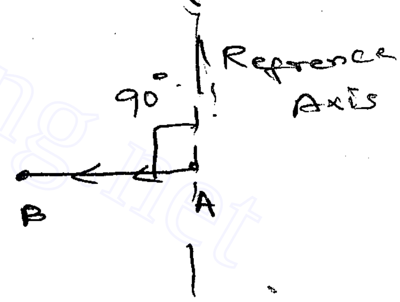
$$\text{Closing error} = \sqrt{(1.39)^2 + (2.17)^2}$$

$$= 2.57\,m.$$

$$\boxed{\tan\theta = \frac{\Sigma D}{\Sigma L}}$$

$$\boxed{\theta = 57^\circ 21' 29.83''}$$

2\. A man travels from A due west and reaches a point B. Distance b/w A and B ~~189.6m~~ 139.6m calculated latitude and departure of line AB.



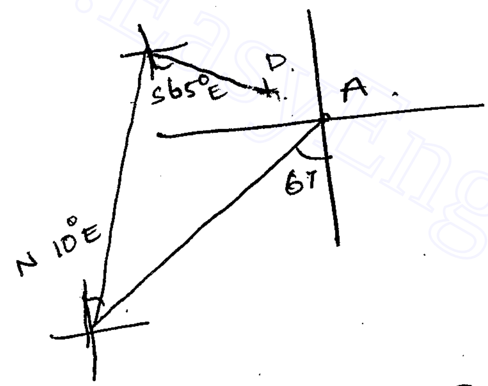
$$\theta = 90^\circ$$

$$\text{Latitude} = l \cos 90^\circ = 0.$$

$$\text{Departure} = l \sin 90^\circ = l.$$

$$= 139.6\,m.$$

3 through the woods a . Survey with hand compass strated from point A and walked 1000 steeps S 67°W. and reached point "B" and then he . changed the direction and walked 512 steps in the direction N 10°E and then reached point "c" and then again changed the direction and walked 1504 step. S°65°E . The surreyor wants to . reach. A . In which direction we must go. to reach point and how many steps . A .



l Bearing

Station l Bearing.

Station	l	Bearing	Direct	$l\cos\theta$	$l\sin\theta$
AB	1000	67°	SW	−390.73	−920.50
BC	512	10	NE	+504.22	+88.91
CD	1504	65	SE	−635.62	+1363.08
DA	x			$l\cos\theta$	$l\sin\theta$

$$\Sigma l\cos\theta = -390.73 + 504.22 - 635.62 + l\cos\theta$$

$$\boxed{l\cos\theta = +522.13}$$

$$\Sigma l\sin\theta = 531.49$$

$$l\sin\theta = -531.49$$

$$(l\cos\theta)^2 + (l\sin\theta)^2 =$$

$$\frac{l\cos\theta}{l\sin\theta} = \frac{522.13}{531.49}$$

$$\boxed{\theta = 44.491°}$$

4. Determine the line EA.

Line	length	Bearing
AB	144.1	85°30'.
BC	201.2	15°
CD	168.4	285°30'.
DE	~~[illegible]~~ 172.6.	195°30'
EA	?	?

5. Closed traverse having following length and Bearing.

Line	Length	Bearing. γ (Rotating east)
AB	200 m.	178°.
BC	98	270°.
CD	Not obtained	1°.
DA.	86.4	

$l\cos\theta$	$l\sin\theta$.
$200\cos\theta$.	$200\sin\theta$.
-99.94.	3.420.
$x\cos 270° = 0$	$-x$.
86.38	1.5078.
0	

$\Sigma l\cos\theta = 0$.

$200\cos\theta - 99.94 + x\cos 270° + 86.38 = 0$.

$$\boxed{\theta = 86° 6' 44.51''}$$

$\Sigma l \sin\theta$.

$200\sin(86°.112) + 3.420 - x + 1.5078 = 0$.

$$\boxed{x = 204.467\text{ m}}$$

6.) Co-ordinate A & B are given below. A third point "C" has been choosen in such a way that AC and CB are 29°30' and 45°45'. Calculate the length of AC and CB.

Point.	Northing.	Easting.
A	150.	200.
B	1500.	1300.

Total latitude = 1500 − 150.

= 1350.

Total Easting = 1300 − 200.

= 1100.

7. A traverse is run to set out line "MQ" & 1900 m. @ right angle to a given line ~~MQ~~. 'MN' The length & bearings are ~~obser~~ observed calculate the length and bearing PQ.

Line	length	Bearing
MN.	—	360°.
MO.	880	120°.
OP.	1000	86°30'.
PQ.	?	?

19/12/2015

SURVEY.

LOCAL ATTRACTION.

* It denotes the influence which prevents the needle from pointing the magnetic north in a given locality. It may be due to electrical cories, steel structures, rail roads, underground rail pipes etc.,
* It can be detected by absorbing the line and finding the F.B and B.B of the difference.

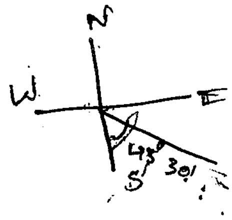
If $F.B \sim B.B = 180^\circ$ then both the stations are free from local attraction.

* The amount of errors due to local attraction @ a station is same in all the bearings @ that place and in the same direction.

Elimination of Local Attraction:

1. By calculating the local attraction @ each station.
2. By included angles.

1. The bearing absorbed in traversing with a compass @ a place where local attraction are suspected are given below. At what station, the suspected local attraction. Find the corrected bearing of the line.



AB.	S 45° 30' E.	N 45° 30 W.
BC	S 60° E.	N 60° 40' W.
CD	N 3° 20' E	S 5° 30' W.
DA.	S 85° W.	N 83° 30 E.

AB.	134° 30'	314° 30'.
BC	120°	299° 20'. 300°.
CD	3° 20'. 4°.	185° 30'. 184°.
DA.	265° 263° 30.	83° 30'.

To ~~134~~ 314° 30' − 134° 30' = 180°.

∴ There is no local attraction in AB.

∴ B.B in BC = 120° + 180°.
= 300°.

Correction in C = 300° − 299° 20'.
= 40°.

Corrected F.B in C = 3° 20' + 40'.
= 4°.

Corrected B.B in D = 4° + 180°.
= 184°.

Correction in D = 185° 30' − 184°.
= 1° 30'.

Corrected F.B value = 265° − 1° 30'
= 263° 30'

		F.B	B.B.
	AB	S 45° 30' E	N 45° 30' W.
	BC	S 60° E	N 60° W.
	CD	N 4° E	S 4° W.
	DA	S 83° 30' W	N 83° 30' E.

1. The following bearing was taken in a closed traverse.

Line	AB	BC	CD	DE	EA
FB	48°25'	177°45'	104°15'	165°15'	259°30'
BB	230°	356°	284°55'	345°15'	79°

State the station which are affected by local attraction by how much. Determine the correct bearing. Calculate the true bearing if the declination 1°30' W.

solution:

Station affected by local attraction

A, B, C

B.B of A $= 259°30' - 180°$

$= 79°30'$

Correction @ A. $= 79°30' - 79°$

$= 30'$

Correction @ F.B A $= 30' + 48°25'$

$= 48°55'$

corrected angle @ B $= 48°55' + 180°$

$= 228°55'$

Correction @ B $= 228°55' - 230°$

$= -1°5'$

corrected F.B @ B $= 177°45' - 1°5'$

$= 176°40'$

Corrected angle @ C $= 176°40' + 180°$

$= 356°40'$

Correction @ C $= 356°40' - 356°$

$= 40'$

Corrected F.B. @ C $= 104°15' + 40'$

$= 104°55'$

$104°55' + 180° = \cancel{320°55'}$ 284°55'.

~~Correction~~ = ~~284°55' - 284°~~.

= ~~55'~~.

~~Corrected \\ Angle \\~~

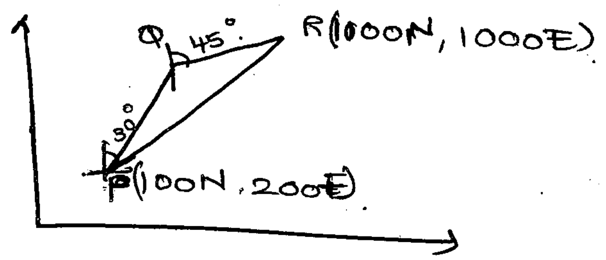
Corrected True Bearing Angle:

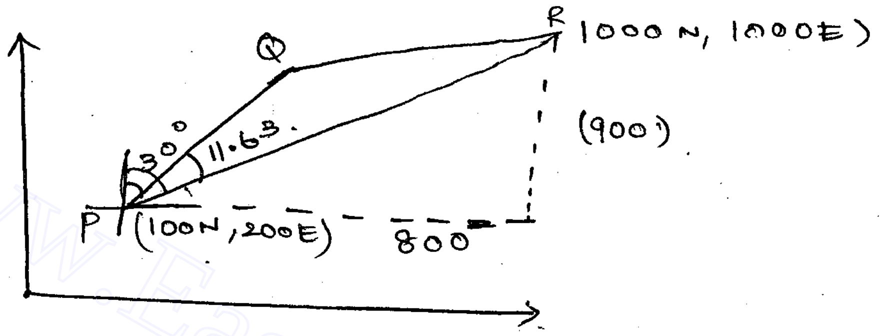
	F.B	B.B.
AB	49°25'	229°25'
BC	176°15'	356°15'
CD	103°25'.	283°25'.
DE	163°45'.	343°45'.
EA	258°27'.	78°27'.

2/9/16.

Survey.

1.) In the figure given below . the length PQ (WBC 30°) and QR (WCB 45°) respectively. upto 3 places of decimal are.





$$PR = \sqrt{800^2 + 900^2} = 1204.159$$

$$\theta = \tan^{-1}\left(\frac{900}{800}\right) = 48.37$$

$\Sigma L = 0.$

$$PQ \cos 30° + QR \cos 45° + 1204.159 \cos 48.37°$$

$\Sigma D = 0$.

$$PQ \sin 30° + QR \sin 45° + 1204.159 \sin 48.37°.$$

$PQ = 273$

$QR = 938$

Method – II. (Projection Method.)

$$x \cos 30 + y \cos 45^\circ = 900.$$

$$x \sin 30^\circ + y \sin 45^\circ = 800.$$

$$\boxed{\begin{array}{l} x = 293 \\ y = 938. \end{array}}$$

Method – III (sine rule)

$$\boxed{\begin{array}{l} 1^\circ = 4' \\ 1' = 4''. \end{array}}$$

2. The latitude and departure of a line AB are +78m and −45.1 m respectively. The WCB bearing of the line AB

$$l\cos\theta = 78.$$

$$l\sin\theta = -45.1.$$

$$\theta = \tan^{-1}\left(\frac{-45.1}{78}\right).$$

$$\boxed{\theta = 330^\circ.}$$

3.)

Line	F.B	B.B.
AB	126°45'	308°00'
BC	45°15'	227°30'
CD	340°30'	161°45'
DE	258°30'	78°30'
ED.	216°30'.	31°45'.

$31°45' - 36°30' = -4°45'.$

$302° - 308° = .$

6/1/2016.

SURVEYING.

1. A 30m chain was tested before a survey and found to have a length. 29.93 If the length of the line measured with this chain 273.3m. Find the true length.

$$\text{True length} = \text{Measured length} \times \left(\frac{L'}{L}\right)$$

$$= 273.3 \times \left(\frac{29.93}{30}\right)$$

$$= \underline{\underline{272.66 \text{ m}}}.$$

2. A true length of a line measured from a plan as per scale was 1276.54m. when the line was measured by 30m long chain, the length was measured as 1274.84m find the chain and also change in length of the true length.

$$\text{True length} = \text{Measured length} \left(\frac{L'}{L}\right)$$

$$1276.54 = 1274.84 \left(\frac{L'}{30}\right)$$

$$L' = 30.04 \text{ m}$$

$$\delta L = L' - L$$

$$= 0.04 \text{ m. (too long)}$$

3. A 30m chain used to measure the length of a line was test before the line was measured and found to be 29.95m. The line was measured and recorded as 590.48m. The chain ~~a~~ was ~~tested~~ again and found to be 30.08 m long. Find T.L

There ~~se~~ is change of length ~~is~~ of chain gradually and hence average length of chain must be used to find true length.

$$L'_{avg} = \frac{30.08 + 29.95}{2}$$

$$= 30.015.$$

$$T.L = 590.48 \times \frac{30.015}{30}$$

$$= 590.7754\ m.$$

4) A rectangular polt was measured with 20m chain by 12 cm too long the ~~s~~ length of side of the rectangle 280×480. Find the true area of the plot.

$$\text{True area} = \text{Measure area} \times \left(\frac{L'}{L}\right)^2.$$

$$= 280 \times 480 \left(\frac{20.12}{20}\right)^2$$

$$\cancel{= 136020.624}$$

$$= 136017.63\ m^2.$$

4.) A 30m chain was tested before starting the days work and found to 20 cm too short. After measuring a length of 1200m. The chain was tested again and was found to be 10cm too long @ the end of the days works the was tested again and was found to be 30cm too long. Find the T.L of line If the total length measured 2648 m.

$$L_1^{\prime}{}_{avg} = \frac{29.8 +}{}$$