CBCS SCHEME

18CV72

Seventh Semester B.E. Degree Examination, Jan./Feb. 2023
Design of RCC and Steel Structures

Time: 3 hrs. Max. Marks: 100

Note: 1. Answer any TWO full questions, choosing ONE full question from each module.
2. Use of IS-456, IS-800, SP-16, SP(6) – steel tables are permitted.
3. Assume missing data suitably.

Module-1

1 Design slab and beam type combined footing for two columns of size 300mm × 300mm and 400 × 400mm subjected to 500kN and 700kN respectively. The centre to centre spacing between columns is 3.50m. The width of the footing is restricted to 1.5m. Take SBC of soil = 150kN/m^2. Use M_{25} and Fe415 grades. Also show reinforcement in L/S and C/S. **(50 Marks)**

OR

2 Design a cantilever retaining wall to retain an earth embankment 4m high above ground level. The density of earth is 18kN/m^3 and its angle of repose is 30°. The embankment is horizontal at top. The S.B.C. of soil is 200kN/m^2. The coefficient of friction between soil and concrete is 0.5. Adopt M-20 and Fe415 grades. Draw C/S elevation of retaining wall. **(50 Marks)**

Module-2

3 A line diagram of a roof truss with internal loads and forces in each members are shown in Fig.Q.3. Design the various members of the roof truss along with their end connection with bolt using property class 5.6 black bolts. Also design the bearing plate at the support for the reaction and anchor bolts for an uplift force of 15kN. Draw elevation of truss greater than half span. **(50 Marks)**

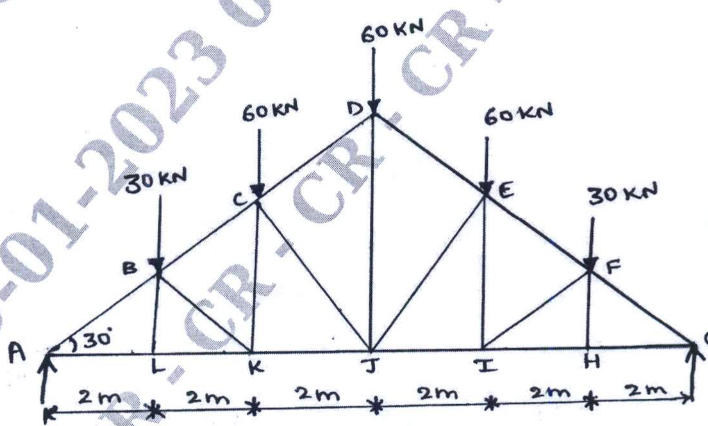

Fig.Q.3

Tabulation of member forces

Members	Length (m)	Force (kN)	Nature of Force
AB, GF	2.31	240.00	Compression
BC, FE	2.31	210.00	Compression
CD, ED	2.31	160.04	Compression
AL, GH	2.00	207.84	Tension
LK, HI	2.00	207.84	Tension
KJ, IJ	2.00	181.82	Tension
BL, FH	1.154	0.00	-
BK, FI	2.31	30.00	Compression
CK, EI	2.31	15	Tension
CJ, EJ	3.05	66.05	Compression
DJ	3.46	66.00	Compression

OR

4 Design a simply supported gantry girder to carry an electrically operated travelling crane with the following data:
Span of crane bridge = 25m
Column spacing = span of gantry girder = 8m
Wheel Base = 3.5m
Crane capacity = 200kN
Weight of crane bridge = 150kN
Weight of Trolley = 75kN
Min Hook Distance = 1.0m
Weight of Rail = 0.30kN/m
Height of Rail = 105mm
Also draw sectional elevation. **(50 Marks)**

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Note: 1. Answer any TWO full questions, choosing ONE full question from each module.
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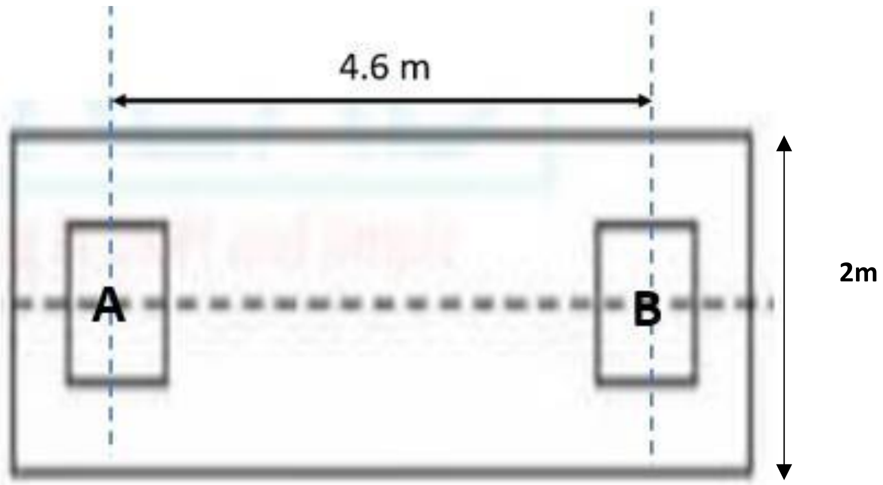
Design slab and beam type combined footing for two columns of size 300mm × 300mm and 400 × 400mm subjected to 500kN and 700kN respectively. The centre to centre spacing between columns is 3.50m. The width of the footing is restricted to 1.5m. Take SBC of soil = 150kN/m^2. Use M_{25} and Fe415 grades. Also show reinforcement in L/S and C/S.

Solutions



- **Footing base dimensions**

Assuming Δ P, the self-weight of the combined footing plus backfill to constitute 10 or 15 percent of the total column loads,

$\Delta P = (700 + 1200) \times 15/100 = 285$ kN

$P_1 + P_2 = 700 + 1200 = 1900$ kN

Allowable soil pressure or safe bearing capacity, $q_a = 130$ kN/m^2

$Area\ of\ the\ footing, A_{req} = \frac{P1 + P2 + \Delta P}{q_a} = 16.8$ m^2

Width of footing, B = 2m (Given in question)

Total Length of footing, L = $\frac{A_{req}}{B}$ = $\frac{16.8}{2}$ = 8.4 m

- **Locate the point of application of the column loads**

In order to obtain a uniform soil pressure distribution, the line of action or point of application of the resultant column load must pass through the centroid of the footing.

Assuming a load factor of 1.5, the factored column loads are:

- $P_{u1} = 700 \times 1.5 = 1050$ kN; $P_{u2} = 1200 \times 1.5 = 1800$ kN $\Rightarrow P_{u1} + P_{u2} = 2850$ kN

  Let $\bar{x}$ be the centroid of the column loads, where s = 4.6 m

  $$\Rightarrow \bar{x} = \frac{P_{u2}\ s}{P_{u1+} + P_{u2}} =$$

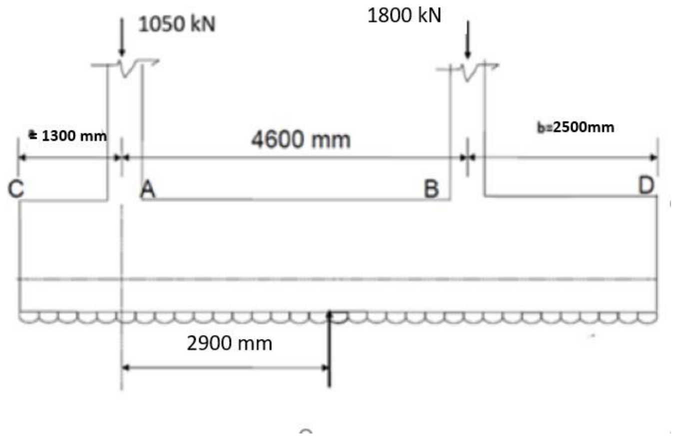
  $$= \frac{2400 \times 4.6}{1050+2400} = 2.9 \text{ m}$$

  If the cantilever projection of footing beyond column A is ‘a’ then,

  a + 2.9 = L /2 = a = 8.4/2 – 2.9 = 1.3 m

  Similarly, if the cantilever projection of footing beyond Column B is 'b' then,

  b = 8.4 - 1.3 - 4.6 = 2.5 m

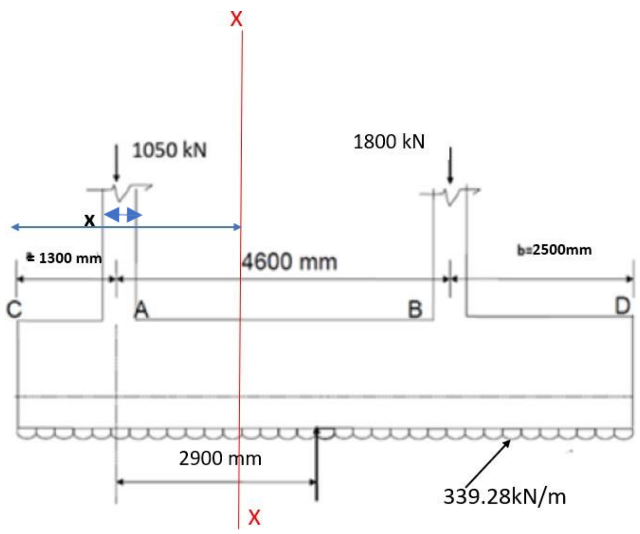


- **Uniformly distributed load acting in upward direction (soil pressure)**

Treating the footing as a wide beam (B = 2000 mm) in the longitudinal direction, the uniformly distributed load (acting upward) is given by q_{uB}

$$q_{uB} = \frac{P_{u1}+P_{u2}}{L} = \frac{1050+1800}{8.4} = 339.28 \text{ kN/m}$$

- **Shear force calculations**



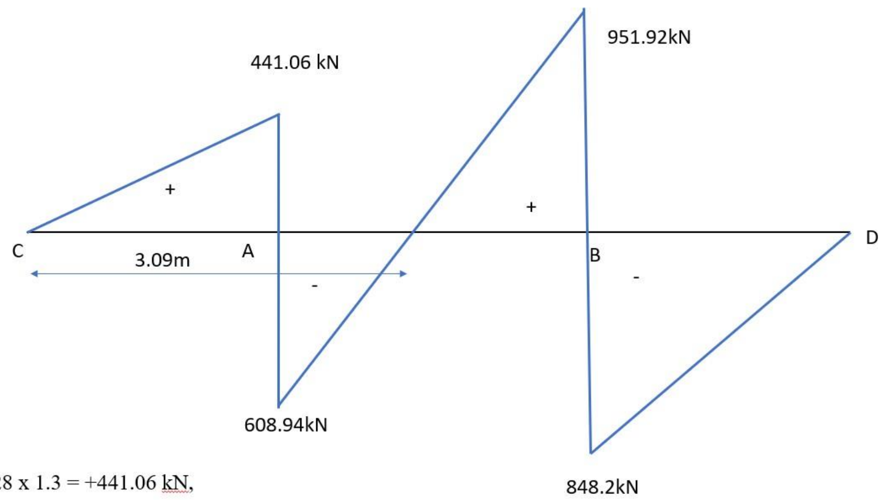
- Shear force at A, just before 1050 kN, left of section XX, V_{AC}= +339.28 x 1.3 = +441.06 kN
- Shear force at A, just after 1050 kN, left of section XX, V_{AB} = -1050+ 339.28 x 1.3 = -608.94 kN
- Shear force at B just after 1800kN, right of section XX , V_{BA} = +1800 – 339.28 x 2.5= + 951.92 kN
- Shear force at B just before 1800kN, right of section XX, V_{BD}=339.28 x 2.5= -848.2 kN

- **Location of zero shear , Left of section XX**

339.28 × X – 1050 = 0 , **location of zero shear,**

339.28 × X = 1050 , 1050/339.28 = X, X =3.09m

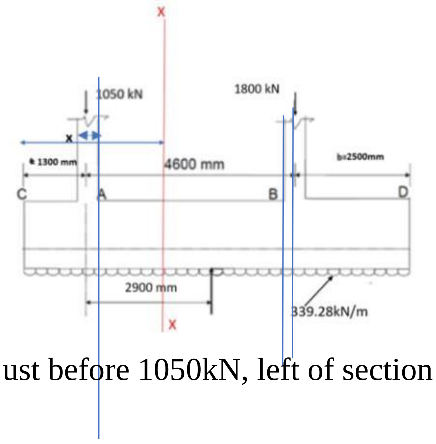
X = 3.09 m from C



- V_{AC}= 339.28 x 1.3 = +441.06 kN,
- V_{AB} = -1050+ 339.28 x 1.3= -608.94 kN
- V_{BA} = 1800 – 339.28 x 2.5= + 951.92 kN
- V_{BD}=339.28 x 2.5= -848.2 kN

Shear force diagram

- **Bending moment calculations**

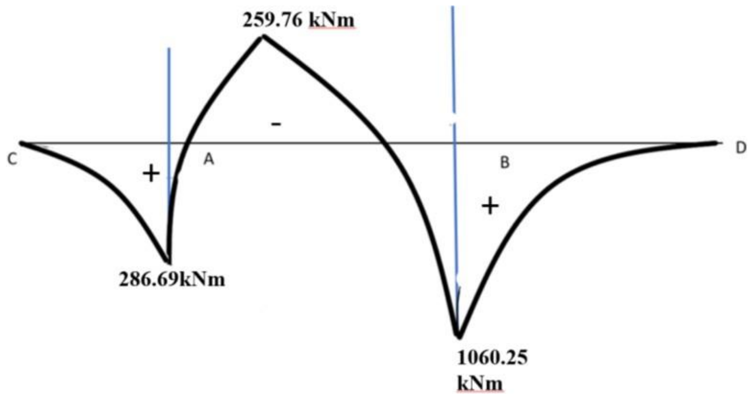


- BM at A, just before 1050kN, left of section XX, M_{AC}= 339.28 x 1.3 x1.3 /2 =+ 286.69 kNm
- BM at just at the inner face of Column A(1050kN), left of section XX,

  M_{AB} = -1050 x 0.35/2 + 339.28 x (1.3 + 0.35/2) x (1.3 + 0.35/2)/2

  = -1050 x 0.35/2 + 339.28 x (1.3 + 0.172) x (1.3 +0.172)/2 = + 185.32kNm
- Negative Bending moment at X = 3.09 m (Location of zero shear)

  Mu – = 339.28 × $(3.09)^2$/2 – 1050 × (3.09 -1.3) = - 259.76 kNm
- BM at B, just before 1800 kN , right of section XX = + 339.28 × 2.5^2/2 = +1060.25 kNm
- BM at B, just after the inner face of Column B (1800 kN) , right of section XX =
- 339.28 × **(2.5 +0.4/2)**^{2}/2 – 1800 × **0.4/2 = +** 876.67 kNm

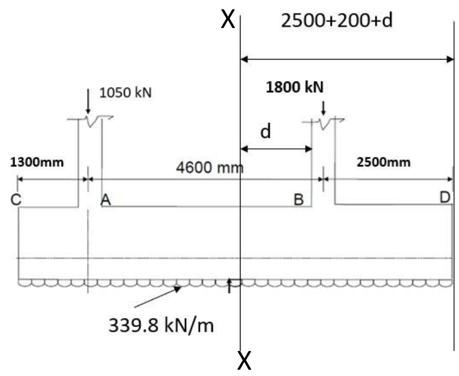


Bending moment diagram

- **Thickness of footing or effective depth of footing based on shear**

One-way shear (longitudinal): *Vu*1 calculate it at a distance "d" from the edge of the heavier column, where "d" is the effective depth of the footing.

The critical section (**always for column with heavier load**) for one-way shear is located at a distance *d* from the (**inner)**face of column B, and has a value



*Critical One-way shear force, Vu*1 at section XX (just right of XX section) =

Column load (B) - Uniformly distributed upward load intensity × (2500 + 200 + *d*)

= (1800 – 339.28 x (2.5 + 0. 200 + *d*)) = (882.54 – 339.28 x *d*) kN … (1)

Take τ_c = 0.48 N/mm^2 (for M 20 concrete, Assuming Percentage of steel as, pt = 0.50) IS 456 2000, page 73, table 19

Design shear strength of concrete, $V_{uc} = \tau_c \times B \times d = 0.48 \times B \times d$

Equate V_{uc} *and* V_{u1}

B is width of footing = 2000 mm

V_{uc} = 0.48 × 2000 × d = (960d) N…. (2)

Equating one-way shear force and design shear strength of concrete, (1) = (2)

$V_{u1} = V_{uc} \Rightarrow (882.54 \times 10^3 – 339.28 \times d) = 960d$, $882.54 \times 10^3 = 1299.8\ d$

⇒ Effective depth of footing, d = 679.25 **mm Rounded to 680 mm**

Use 20 mm φ bars with a clear cover of **75** mm, **Taking an overall depth or thickness of the footing**

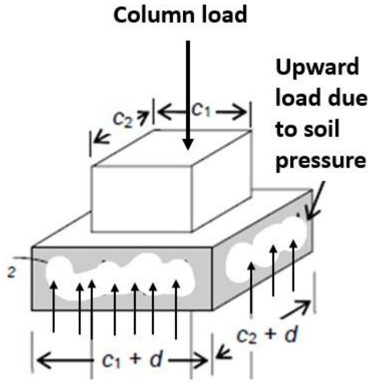
D **=** $d + 75 + 20/2$ = 680 + 75 + 20/2 = 765 mm

- **Two-way shear force for columns *A* and *B* (Punching shear)**

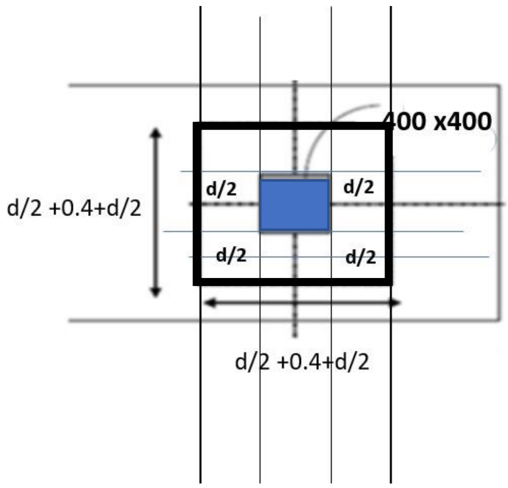
Two-way shear or punching shear ***(we need to consider the upward soil pressure not upward soil intensity) * Since it is acting on an area.***

Factored soil pressure or Upward soil pressure, q_u = (339.28) / (B ×1) = (339.8/2) = 169.64kN/m^2

The critical section is located d/2 from the periphery of columns *A* and *B.*



Shear stresses in footing slab due to punching shear

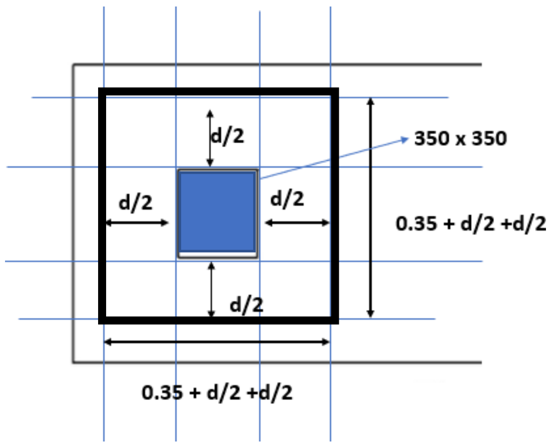


Punching shear or Two- way shear calculations for heavier Column B

V_{u2} = 1800 – 169.64 (0.4 + 0.680/2 + 0.680/2) X (0.4+0.680/2 + 0.680/2)

= 1602.13 kN @ B (Heavier column)

Punching shear or Two-way shear for Column A (350 mm x 350 mm)



Punching shear or Two-way shear @ A,

Two-way shear Vu_2 = (Column load at A) 1050 – 169.64 x (0.35 +0.680/2 + 0.680/2) x (0.35 +0.680/2 + 0.680/2)

= 870 kN @ A (Lighter column)

If no shear reinforcement is provided, Page 58, IS 456, Clause 31.6.3.1, calculated shear stress at critical section shall not exceed $k_s\,(0.25\sqrt{f_{ck}}\,)$

where

k_s = $(0.5 + \beta_c)$ but not greater than 1, β_c being the ratio of short side to long side of the column/ capital; and

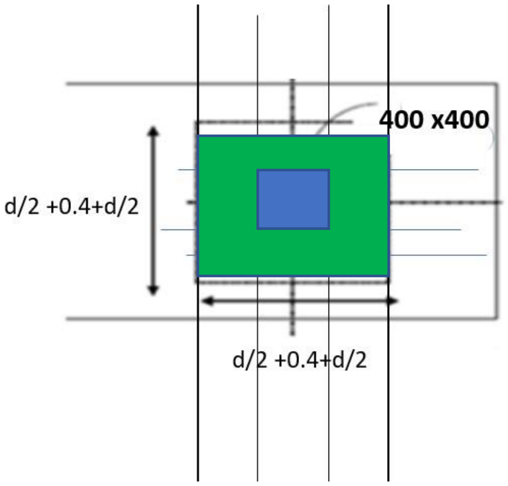
τ_c = $0.25\sqrt{f_{ck}}$ in limit state method of design, and $0.16\sqrt{f_{ck}}$ in working stress method of design.

For square columns, *ks* = (0.5 +βc) , βc = 350/ 350 = 400/400 = 1.0 , ks =(0.5 +1) but it should not be greater than 1, hence ks = 1

Permissible shear stress, $\tau_{c2} = k_s\,(0.25\sqrt{f_{ck}})$ = 1.0 × 0.25 ×√ 20 = **1.118** N/mm^2

Permissible two-way shear force for column *B (heavier column)*

Permissible two-way shear force, V_{uc} **= Permissible shear stress** × **(Area of the footing slab enclosed by the perimeter of the critical section)**



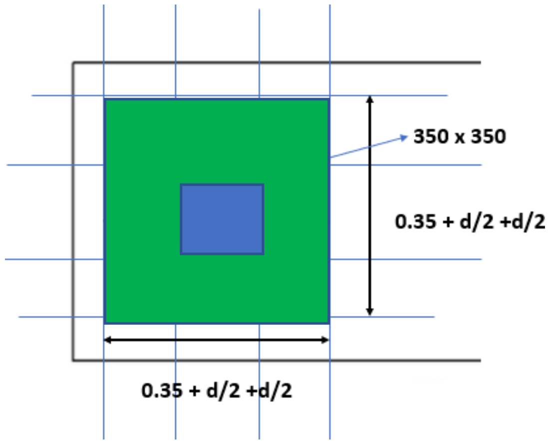
Perimeter of critical section (Green coloured area) = 4 X (400 + 680/2 + 680/2)

V_{uc} = 1.118 x [**4 X (400 + 680/2 + 680/2)**] × 680 = 3284.24 kN @ B

In the similar way lets calculate for Column A

Permissible two-way shear force for Column A

V_{uc} = 1.118 × [(350 + 680/2 + 680/2) × 4] × 680 = 3132.18kN @ A



Compare whether permissible two way shear force is greater than two shear way (Actual) force

V_{uc} = 3284.23 kN > V_{u2} = 1602.82 kN **@ B It is safe.**

V_{uc} = 3132.18**kN > V_{u2}** = 870.00kN **@ A . It is Safe.**

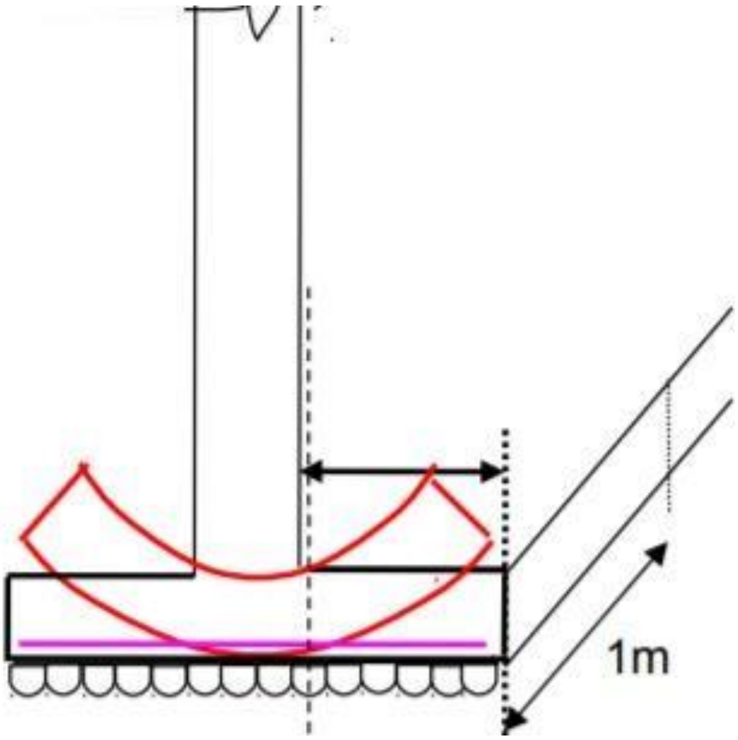
Hence safe against two way or punching shear, (if not provide shear reinforcement- stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

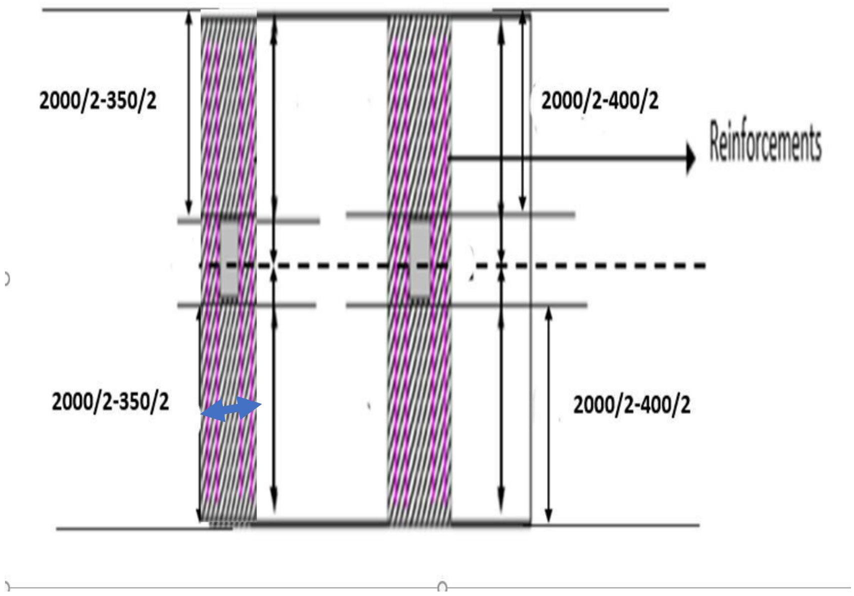
Maximum 'negative' moment: *Mu* = - 259.76 kNm	Maximum 'positive' moment: *Mu* = + 1060.25 kNm
$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$ *Mu* = 259.76 × 10^6 N mm B = b = 2000 mm, fck = 20 N/mm^2, fy = 415N/mm^2 d = 680 mm D = 765 mm Ast provided = 1075.67 mm^2 Check for (*Ast*)*min* = 0.0012 *BD* = 0.0012 × 2000 × 765 = 1836 mm^2	$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$ *Mu* = 1060.25 × 10^6 N mm B = b = 2000 mm d = 680 mm Ast provided = 4648.12 mm^2 Check for (*Ast*)*min* = 0.0012 *BD* = 0.0012 × 2000 × 765 = 1836 mm^2

Ast provided < (*Ast)min, Hence provide (Ast)min* But we have assumed pt = 0.5 pt = 100 Ast, req/ (B × d) Choose 16 mm diameter bars, calculate no of bars = 1836 /(π/4 x 16^2) = 10 **Provide 10 # 16 mm diameter bars at top** • **Development length L_d = 47 x dia of bar** **= 47 x 16 = 752 mm**	Since pt was assumed as 0.5 %, we need to calculate Ast, req by using pt = 100 Ast, req/ (B × d) **Ast, req = 0.50 x 2000 x 680/100 = 6800 mm^2** Since Moment is more, we are choosing 6800 mm^2 and going for larger diameter bars ie 20 mm No of 20 mm dia bars = 6800/(π/4 x 20^2) = 22 **Provide 22 # 20mm diameter bars at bottom** • **Development length L_d = 47 x dia of bar** **= 47 x 20 = 940 mm**

Design of column strips as transverse beams



Transverse bending of footing

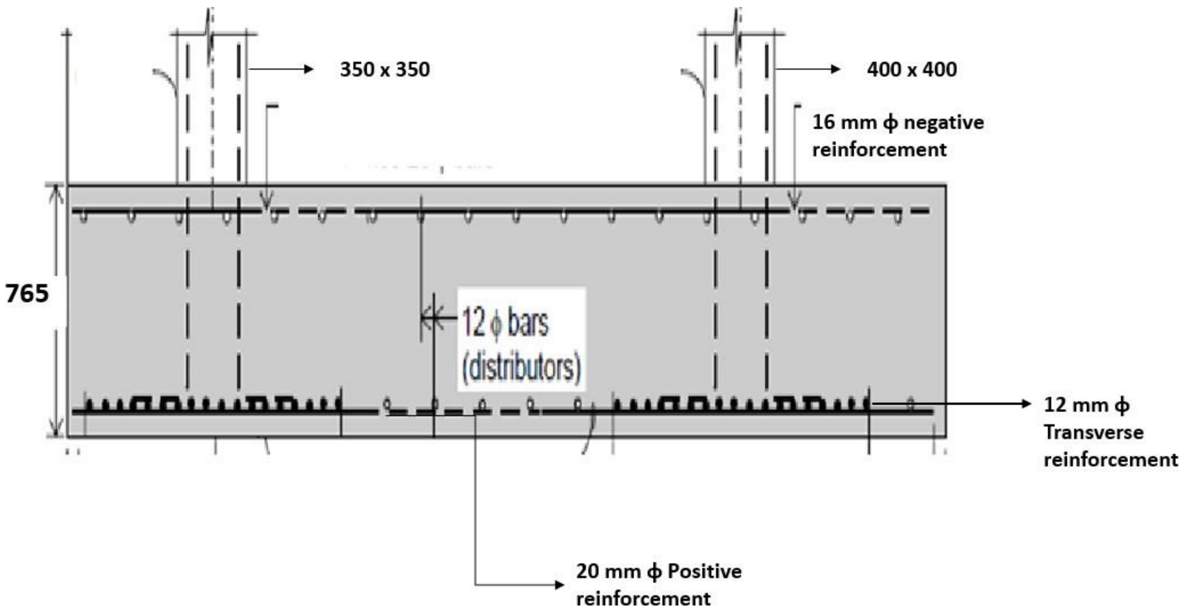


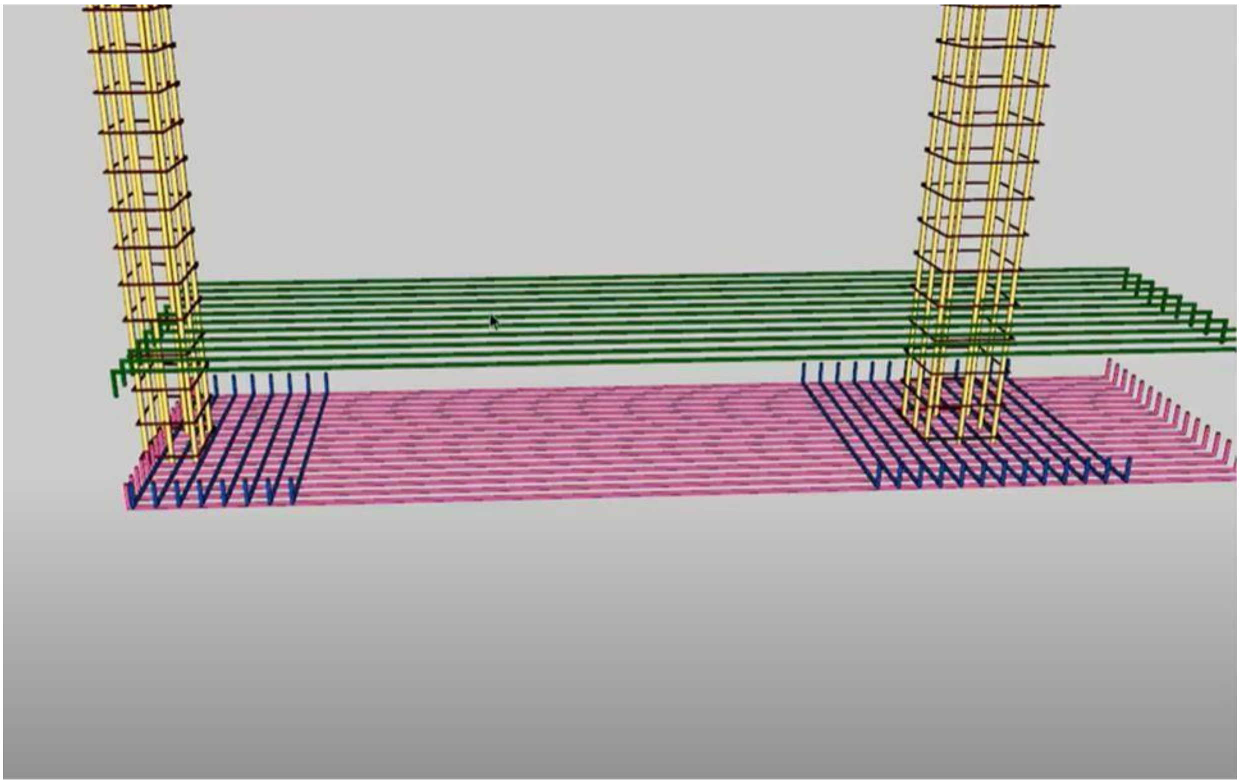
Transverse beam under column A	*Transverse beam under column* B
• Factored Column load A per width of footing = 1050/2.0 = 525 kN/m • Cantilever Projection of beam beyond column face = (2000 – 350)/2 = 825 mm =0.825 m • Maximum transverse moment at column face A : $Mu = 525 \times 0.825^2/2 =$ 178.66 kNm • Effective depth for transverse beam (Assume 16 mm φ bars as transverse reinforcement which is placed above the 20 mm φ longitudinal bars): *d* = 680 – 75 – 20 – 16/2 = 577 mm • Assume width of transverse beam, b = width of column + 2 x 0.75*d* b = 350 + 2 x 0.75 × 577 = 1215.5 mm $M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$	• Factored Column load B per width of footing = 1800/2.0 = 900 kN/m • Cantilever Projection beyond column face = (2000 – 400)/2 = 800 mm = 0.800m • Moment at column face B = $900 \times 0.80^2/2 = 288$ kNm • Width of transverse beam, b = width of column + 2 x 0.75d • 400 + 0.75× 577 + 0.75 × 577 = mm = 1265.5mm $M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$ Mu = 384×10^6 N mm b = 1265.5mm d = 577 mm Ast = $1956.2\ mm^2$

b = 1215.5 mm, d =577 mm Mu = 178.6×10^6 N mm Ast = 880.23 mm^2	
Page 48, CL No 5.2.1 • Minimum Ast = 0.0012 *bD* = $Ast\ min$ = .0012 ×1215.5 ×765 = 1115.83 mm^2 • Use 12mm dia bars (Your wish!!) • Number of 12 mm φ bars required = Ast/ area of one bar = 1115.83/ ($\pi/4 \times 12^2$) =10 **Provide 10 nos 12 mmφ bars** Check for development length = 47 × 12 = 564 mm	• Provide $(Ast)min$ = 0.0012 × 1265.5×765 = 1161.73 mm^2 Use 12 mm dia bars Number of 12 mm φ bars required =1956.2 /($\pi/4 \times 12^2$) = 17.29 = 18 **Provide 18 nos 12 mmφ bars** • Required development length = 47.0 × 12 = 564 mm is available beyond the column face.
Transfer of force at column base -*Column* A	**Transfer of force at column base** *Column* B
• Limiting bearing stress at IS 456 Page 65 , CL34.4 **34.4 Transfer of Load at the Base of Column** The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to $\sqrt{\frac{A_1}{A_2}}$ but not greater than 2; where A_1 = supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and A_2 = loaded area at the column base.	• Limiting bearing stress at Permissible bearing stress = $0.45 fck \sqrt{\frac{A1}{A2}}$ [$A1 = 2000^2$, $A2 = 400^2$ mm^2] $\sqrt{\frac{A1}{A2}} = 5 > 2$, $\sqrt{\frac{A1}{A2}} = 2$ = 0.45 × 20 × 2 = 18 MPa *Permissible bearing resistance* = 18 × 400^2 = 2880 kN *2880kN > 1800kN , Hence safe*

For working stress method of design the permissible bearing stress on full area of concrete shall be taken as $0.25 f_{ck}$; for limit state method of design the permissible bearing stress shall be 0.45 f_{ck}. Permissible bearing stress = $0.45 fck \sqrt{\frac{A1}{A2}}$ $A1 = 2000^2$ (2000mm is footing width) A2 = 350 x 350 = $=\sqrt{\frac{A1}{A2}} < 2$ = 5.71> 2 , Take $\sqrt{\frac{A1}{A2}} = 2$ Permissible bearing stress = 0.45 X 20 X 2 =18 N/mm^2 Permissible bearing resistance or force F_{br} = Permissible bearing stress × column area $= 18 \times 350^2 = 2205 \times 10^3$ N = 2205 kN *2205> 1050 kN, Hence safe.*	

Reinforcement detailing





OR

Design a cantilever retaining wall to retain an earth embankment 4m high above ground level. The density of earth is $18 kN/m^3$ and its angle of repose is 30°. The embankment is horizontal at top. The S.B.C. of soil is $200 kN/m^2$. The coefficient of friction between soil and concrete is 0.5. Adopt M-20 and Fe415 grades. Draw C/S elevation of retaining wall.

(50 Marks)

2. Design a cantilever retaining wall to retain soil of earth of height 4.0 m above ground level. The soil is having safe bearing capacity and density are respectively $200 kN/m^2$ and $18 kN/m^3$. Design all components of retaining walls with all stability checks. Also draw detailed drawing. Use M 20 and Fe 415. Angle of Repose, $\Phi = 30^0$, Coefficient of friction between concrete and soil $\mu = 0.5$.

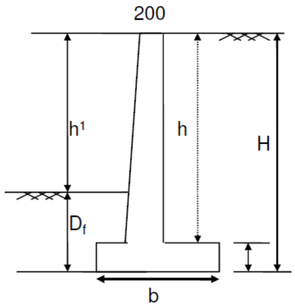
Solution

Height of earth fill, h'= 4.0 m, Safe bearing Capacity, SBC = 200 kN/m^2, Density of soil, γ = 18 kN/m^3, co-efficient of friction between concrete and soil, μ = 0.5, angle of repose φ =30°

We need to fix the total height of retaining wall, $H = h' + D_f$

- Depth of foundation, D_f – (Height of wall below Ground level)

Using Rankine's formula: find depth of foundation



$$D_f = \frac{SBC}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2 = \frac{SBC}{\gamma}k_a^{\ 2}$$

Active earth pressure coefficient $k_a = \frac{1-\sin 30^0}{1+\sin 30^0} = \frac{1}{3}$

Passive earth pressure coefficient $k_p = \frac{1}{k_a} = 3$

$= \frac{200}{18}\left[\frac{1-\sin 30^0}{1+\sin 30^0}\right]^2$ **1.23 m ≈ 1.2 m**

Therefore total height of retaining wall H = 4.0 + 1.2 = 5.2 m

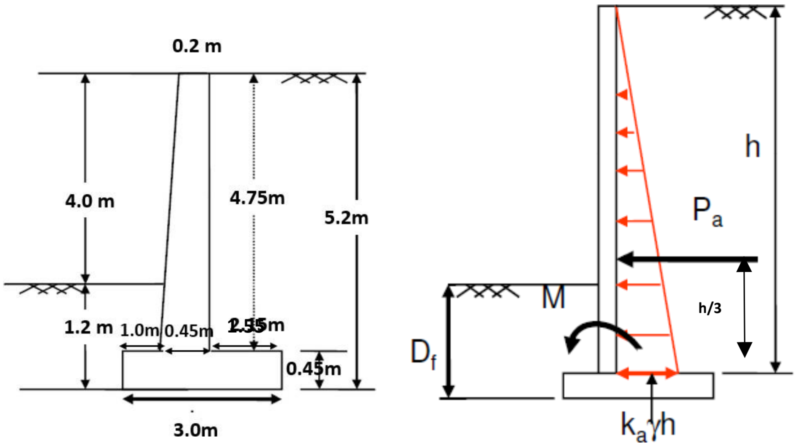
- ## Proportioning of wall

a. **Thickness of base slab** = (1/10 to 1/14) H = 1 /10 × 5.2 to 1 /14 × 5.2 = 0.52m to 0.37m, say 0.45 m - **450** mm

b. **Width of base slab** = b = (0.5 to 0.6) H = 0.5 × 5.2 or 0.6 × 5.2 = 2.6 m to 3.12 m say 3.0 m

c. **Toe projection** = (1/3 to ¼) b = 1 /3 × 3.0 or 1/4 × 3.0 = 1.0 m to 0.75 m say **1.0** m(your wish !!)

d. Provide **450 mm** **thickness for the stem at the base (overall depth D)** and 200 mm at the **top**



- ## Design of stem

To find Maximum bending moment at the junction

Height of stem, h = 5.2 – 0.45 = 4.75 m

Active earth pressure acting on stem slab, $P_a = \frac{1}{2}(k_a \times \gamma \times h)\,h =$

$P_a = 1/2 \times 1/3 \times 18 \times 4.75 \times 4.75 = 67.68$kN

Total Bending moment at any height, $M = P_a \times \frac{h}{3}$

$$M = 14.25 \times \frac{4.75}{3} = 107.16 \text{ kN-m}$$

$M_u = 1.5 \times M = 160.74$ kN-m

Taking 1m length of wall,

We have overall depth at base or thickness of stem slab as, D = 450 mm

Check for effective depth “d”

$$M_{u,lim} = 0.36\,\frac{x_{u,max}}{d}\left(1 - 0.42\,\frac{x_{u,max}}{d}\right) b d^2 f_{ck}$$

Put Mu,lim = 160.74×10^6, b = 1000mm , fck = 20N/mm^2

xu,max /d = 0.48 , Fe 415 , IS 456 2000

$160.74 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 20$

d = 241.3 mm ≈ 242 mm

effective cover = clear cover + bar diameter/2 (assuming 12 mm φ bars)

= 40 + 12/2 = 46 ≈ 50 mm

d = Overall depth – effective cover = 450 – 50 = 400 mm >> 242 mm, hence safe

- Area of steel for stem slab

$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$

d = 400 mm, b =1000 mm, Mu =160.74 x 10^6 Nmm, fy = 415 N/mm^2 , fck = 20 N/mm^2
Ast = 1184.86 mm^2
Ast, min = $0.0012 \times b \times D = 0.0012 \times 1000 \times 450 = 540\ mm^2$
Ast > Ast,min, hence Ok.

- Main steel

Provide 16 mm φ bars as main steel

Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 16^2}{1184.86}$ = 169.69 mm ≈ 170 mm or 160 mm (Your wish!!)

Main steel #16 mm φ @ 170 mm c/c < 300 mm or 3 times effective depth "d" (Check!!!) IS 456 2000

- Distribution steel or Ast,min

= 0.12% Gross Area = 0.0012 × 450 × 1000 = 540 mm^2
Use 10 mm φ bars, spacing required
Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 10^2}{540}$ = 145.4 mm ≈ 140 mm or 150 mm (Your wish !!)

Distribution bars #10 mm φ @ 150 mm c/c < 450 mm and 5 times effective depth "d" ok (check!!) IS 456 2000

- Development length L_d

L_d = 47 Φ $_{bar}$ = 47 × 16 = 752 mm =0.752 m

- Curtailment of bars

Curtail 50% steel from top, Ast = $\frac{50}{100} \times 1184.86 = 590\ mm^2$

$\left(\frac{h_1}{h}\right)^2 = \frac{1}{2}$, $\left(\frac{h_1}{5.2}\right)^2 = \frac{1}{2}$, $\frac{h_1^2}{5.2^2} = \frac{1}{2}$, $h_1 = 3.67\ m$, is the curtailment length or cutting length

Actual point of cut off or cutting position = 3.67 –L_d =3.67 - 0.752 = 2.91 m is the cutting length from top.

Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 16^2}{590}$ = 340.7 mm ≈ 340 mm As IS 456 spacing < 300 mm or 3 times effective depth "d" (Check!!!)

$s = 340$ mm > 300 mm

Instead of this you have to provide every alternate bars at 300 mm c/c.

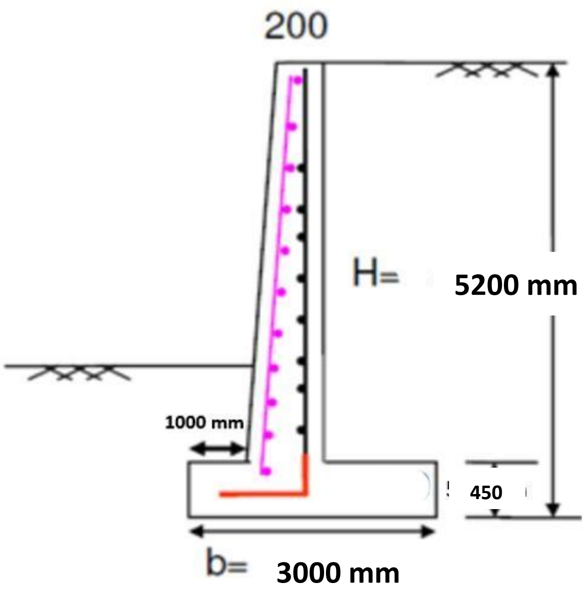
Spacing of bars 16 mm φ @ 300 mm c/c. Hence it is ok.

Secondary steel for stem at front (Temperature steel)

0.12% Gross Area = 0.12 × 450 × 1000/100 = 540

mm^2

#10 @ 150 mm c/c< 450 mm and 5d Hence it is ok.



- Check for shear for stem slab

Max. Shear Force at Stem Junction, P_a = 67.68kN (Lateral earth pressure)
Ultimate Shear Force = Vu =1.5 x [redacted] kN
Nominal shear stress = τ_v = Vu/bd = 101.52 x 10^3 / (1000 x 400) =0.25 N/mm^2
To find τc, calculate $p_t = \frac{100\,A_{st}}{b \times d} = \frac{100 \times 1184.}{1000 \times 400} =$ 0.295 %

Use IS:456-2000, Page 73, Table 19, p_t = 0.295 %, M 20

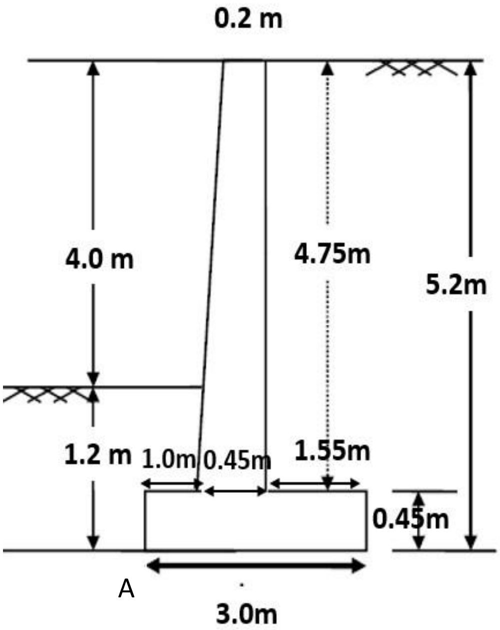
τc = 0.379N/mm^2≈ 0.38 N/mm^2

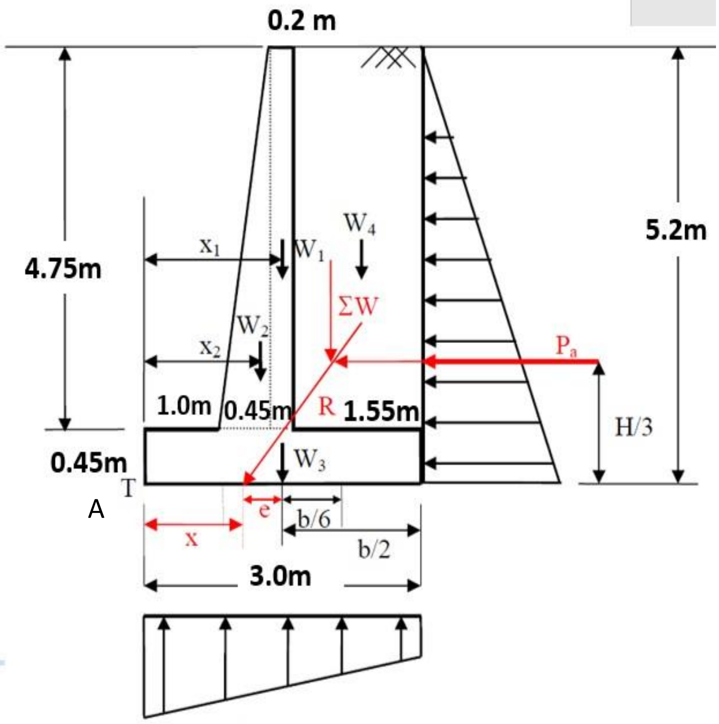
Compare τ_v and τ_c , [redacted]

$\tau_v < \tau_c$ Hence safe in shear. No need of shear reinforcement.

Stability analysis – 1. To find factor of safety against overturning

Calculations of **Resisting Moment** **ΣM_R** – **Self weight of wall and weight of earth fill retained by heel slab**





Pressure below the Retaining Wall

Load	Magnitude, kN	Distance from A, m	Bending moment about A kN-m
Stem W1	0.2 x4.75 x1x25 =23.75	(1.0+0.25+0.2/2) =1.35	32.06
Stem W2	½ x 0.25 x 4.75 x1x25 =14.84	1.0 + 2/3 x 0.25 = 1.16	17.21
Base slab W3	3.0 x0.45 x1 x25 = 33.75	3.0/2 = 1.5	50.63
Back fill, W4	1.55x4.75x1x18 = 132.525	1.0+0.45+1.55/2 =2.225	294.857
Total	**ΣW=204.86 kN**		**ΣM_R = 394.76 kN-m**

Calculations of Overturning Moment **M_O – Lateral earth pressure about the base slab**

Load	Magnitude, kN	Distance from A, m	Bending moment about A kN-m
Hori. earth pressure =P_H	P_H = ½ x 1/3x18x5.2^2 =81.12 kN	H/3 =5.2/3	**M_O = 140.61**

Stability checks:

1. **Check for overturning**:

As per IS: 456:2000, *(Factor of Safety) overturning* should satisfy condition that $\Sigma M_R / M_O >$ 1.55

ΣM_R = 394.757 kNm, M_O = 140.61 kNm

(F.S) overturning = $\Sigma M_R / M_O$ = 2.80 >1.55 Hence it is safe

2. Check for Sliding: $\Sigma W = 204.86kN$

P_H =81.12 kN (Horizontal earth pressure)

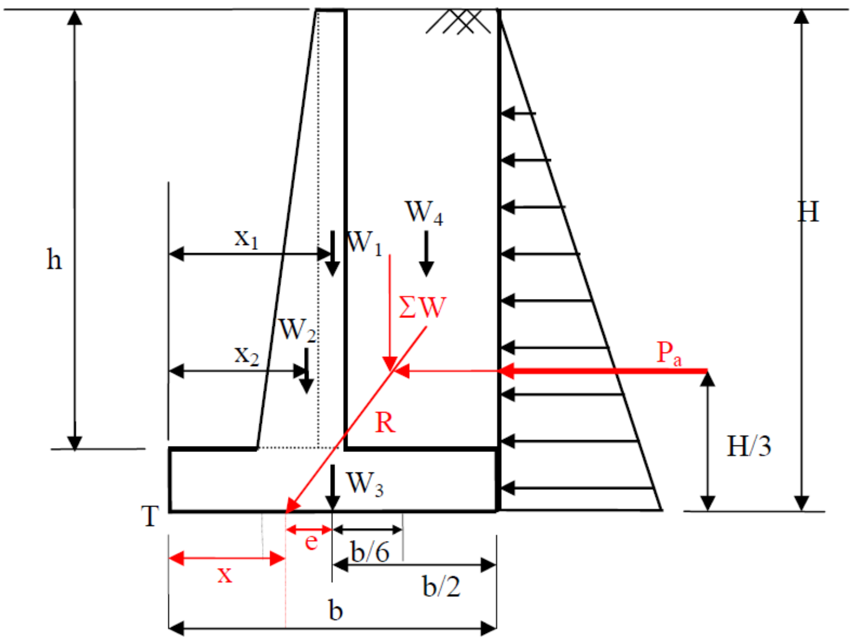
As per IS: 456:2000, *(F.S) sliding* should satisfy condition that $\mu\sum W/P_H \geq 1.55$

$$\frac{\mu\sum W}{P_H} = \frac{0.5 \times 204.86}{81.04} = 1.26$$

(F.S) sliding = 1.26 ≤1.55 Hence it is not safe against sliding. Shear key is needed.

3. **Check for subsidence**: (Max. pressure at the toe should not exceed the safe bearing capacity of the soil under working condition)

Let the resultant cut the base at distance 'x' from toe T,
x = ∑M / ΣW, where ∑M = Net moments about toe = ΣM_R - M_O = 394.757 – 140.61 = 254.15 kNm



$$x = \frac{254.15}{204.86} = 1.24 \text{ m} \quad , b = 3\text{m}$$

- **Eccentricity e = b/2 – x** = 3/2 – 1.24 = 1.5 – 1.24 **= 0.26m < b/6** , 0.26 <0.5
- e = 0.26m
- (Eccentricity of force should not exceed one sixth of base)

Here e < b/6. Hence it is safe.

Pressure below the base slab

$$\text{Max. pressure} = P_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

103.79kN/m^2 < SBC, safe bearing capacity

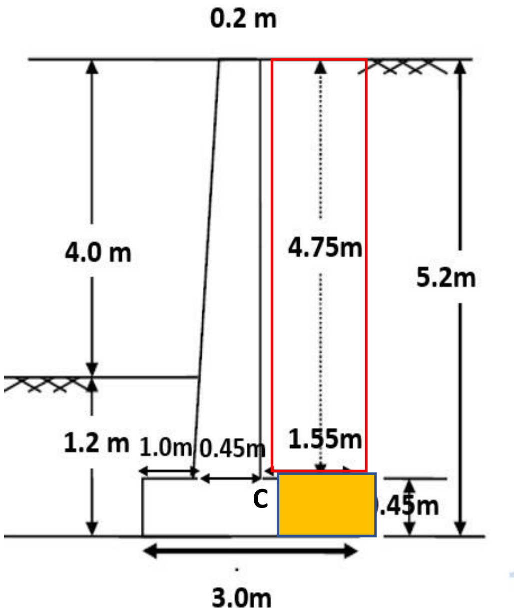
$$\text{Min. pressure} = P_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$$

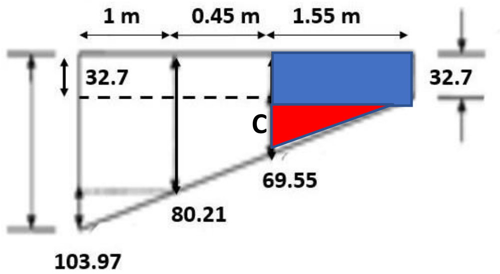
32.7kN/m^2 > zero, So there is no tension or separation developed at base slab, Hence it is safe

Both values of pressure are lesser than SBC (200 kN/m^2) . Hence it is safe.

Design of Heel Slab

Calculations of Moment about heel slab C





Load	Magnitude, kN	Distance from C, m	BM, M_C, kN-m
Backfill or earthfill	1.55 x 4.75 x 1 x18 = 132.52	1.55/2 = 0.775	102.703
Heel slab	0.45 x1.55 x 25x1 = 17.43	1.55/2 = 0.775	13.51
Upward Pressure distribution, (below heel slab) rectangle	-32.7 x 1.55 = -50.68	1.55/2 = 0.775	-39.28

Upward Pressure distribution, Triangle 	- ½ x (69.55 - 32.7) x 1.55 = -28.55	$\frac{1}{3}$ x1.55 =0.516	-14.73
Total Load at junction C	70.72	**Total BM at Junction C**	**ΣM_C = 62.18**

ΣM_C = 62.18 kNm

Mu = 1.5 x 62.18 = 93.27 kNm

$$M_u = 0.87\ f_y\ A_{st}\ d\left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$$

Mu = 62.18 x 10^6 Nmm, b = 1000 mm, d = 400 mm, f_{ck} =20 N/mm^2 f_y = 415 N/mm^2

Ast = 669.04 mm^2

Use 12 mm φ bars (it is base slab) (You can choose 12 mm also)

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{669.04}$ = 170

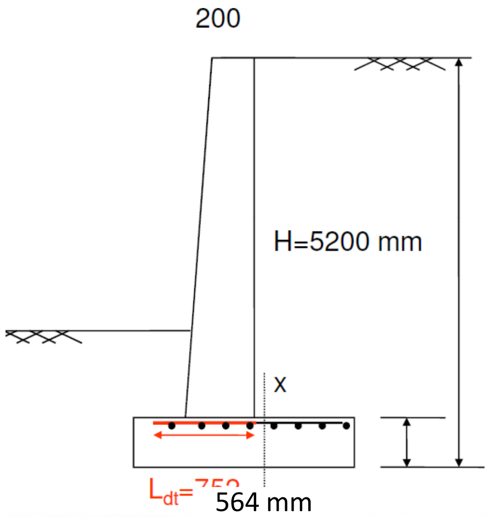
Main steel #12 mm Φ @170 mm c/c < 300 mm and 3d ok. Hence it is safe.

Development length

L_d = 47 φ_{bar} = 47 x 12 = 564 mm

Distribution steel

#10 mm Φ @ 140 mm c/c < 450 mm and 5d ok



Check for shear at junction (Tension)

Critical section for shear is at the face as it is subjected to tension.

Maximum shear =V= 70.72 kN, V_U, max = 70.72 x 1.5 =106.08 kN

$$\tau_v = \frac{V_U}{b \times d} = \frac{106.08 \times 10^3}{1000 \times 400} = 0.26$$

$$pt = \frac{100 \times 669.04}{1000 \times 400} = 0.167\%$$

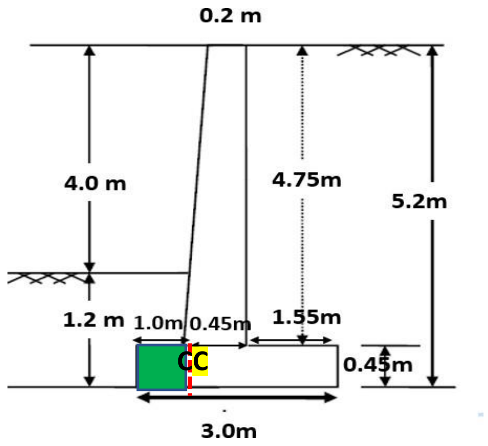
Use IS:456-2000, Page 73, Table 19, pt = %

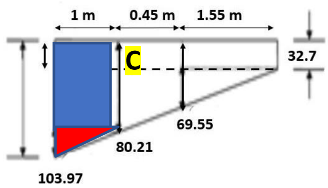
τ_c = 0.297 N/mm^2

Compare . τ_v and τ_{uc} , $\tau_v < \tau_c$ Hence it is safe in shear.

Design of toe

To find the maximum bending moment





Load	**Magnitude, kN**	**Distance from C, m**	**BM, M_C, kN-m**
Self wt Toe slab	1.0 x 0.45 x 25 = 11.25	1.0/2	5.625
Upward Pressure distribution, rectangle	-80.21 x 1.0 = -80.21	1.0/2	-40.10
Upward Pressure distribution, Triangle	-½ x (103.97-80.21) x 1.0 = -11.88	2/3 x 1 = 0.66	-7.92
Total Load at junction		**Total BM at junction**	**ΣM_C = -42.4**

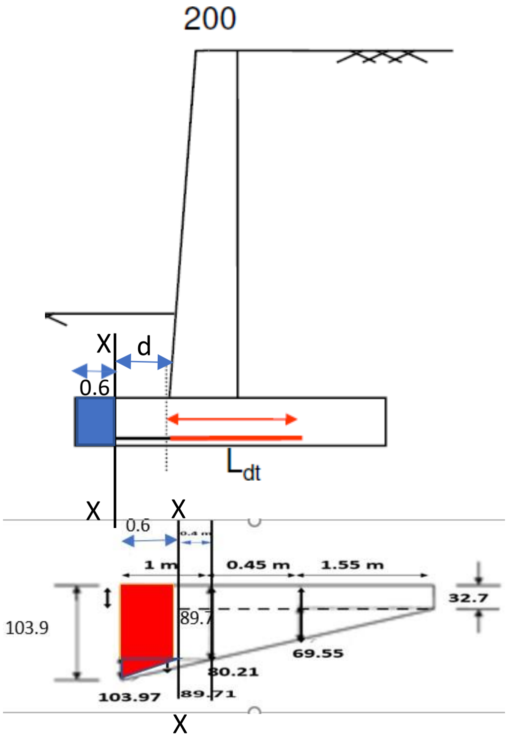
Mu = 1.5 x 42.4= 63.75kN-m,

Ast = 452.02 mm^2 , since Area of steel is less, we have used 10 mm Φ bars

Provide Main steel #10 @ 150 mm c/c < 300 mm and 3d ok

Development length:

L_d = 47 φbar = 47 x 10 = 470 mm



Check for shear: at d from junction, d =400 mm

Net shear force at the section XX

V= **-(103.9 +89.7) /2 x 0.6** + **0.45 x 0.6 x 25** = -51.33 kN

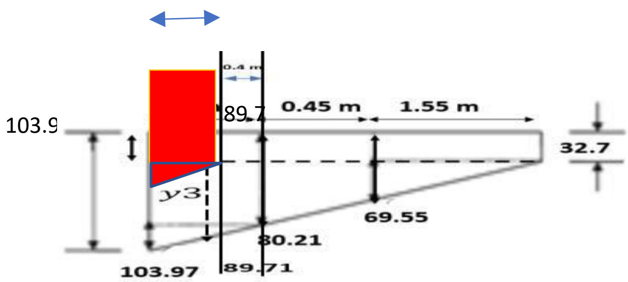
$V_{U,max}$ = 51.33 x 1.5 = 76.9 kN

ζv = 76.9x1000/ (1000x 400) =0.19 MPa,

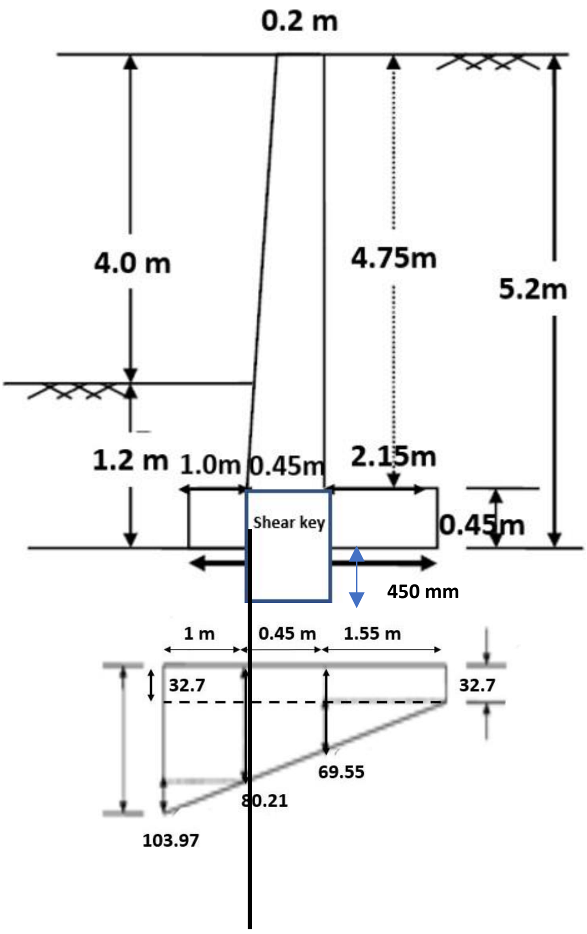
find pt = 100 x Ast /(1000 x 400) =0.113

From IS:456-2000, ζc = 0.28 MPa

ζv < ζc, Hence safe in shear.



- Design of Shear key



Assume width and height of shear key as 450 mm. Let Pp be Total passive force developed in the front of shear key.

Pp = Pressure at junction of toe × width of shear key ×coefficient of passive pressure × 1m

= **80.21** × 0.45 × 3×1 = 108.28kN

Factor of safety against sliding

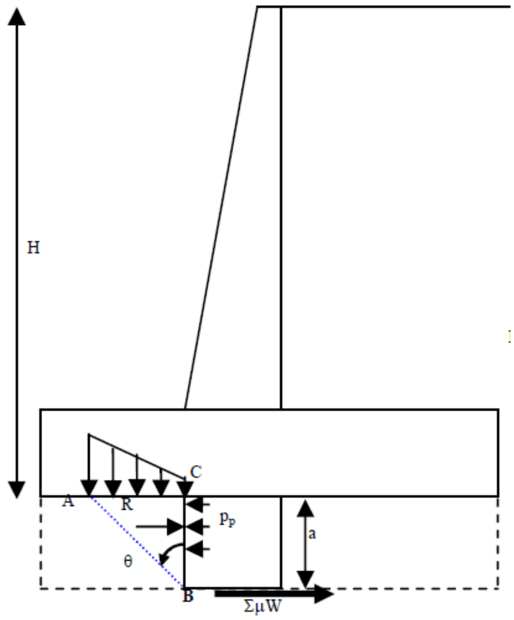
FOS= $\frac{\text{Pp} + \mu\sum W}{P_A} = \frac{108.28+0.6\times 204.86}{\mathbf{81.12}}$ **= 2.85≥1.55**

Hence safe

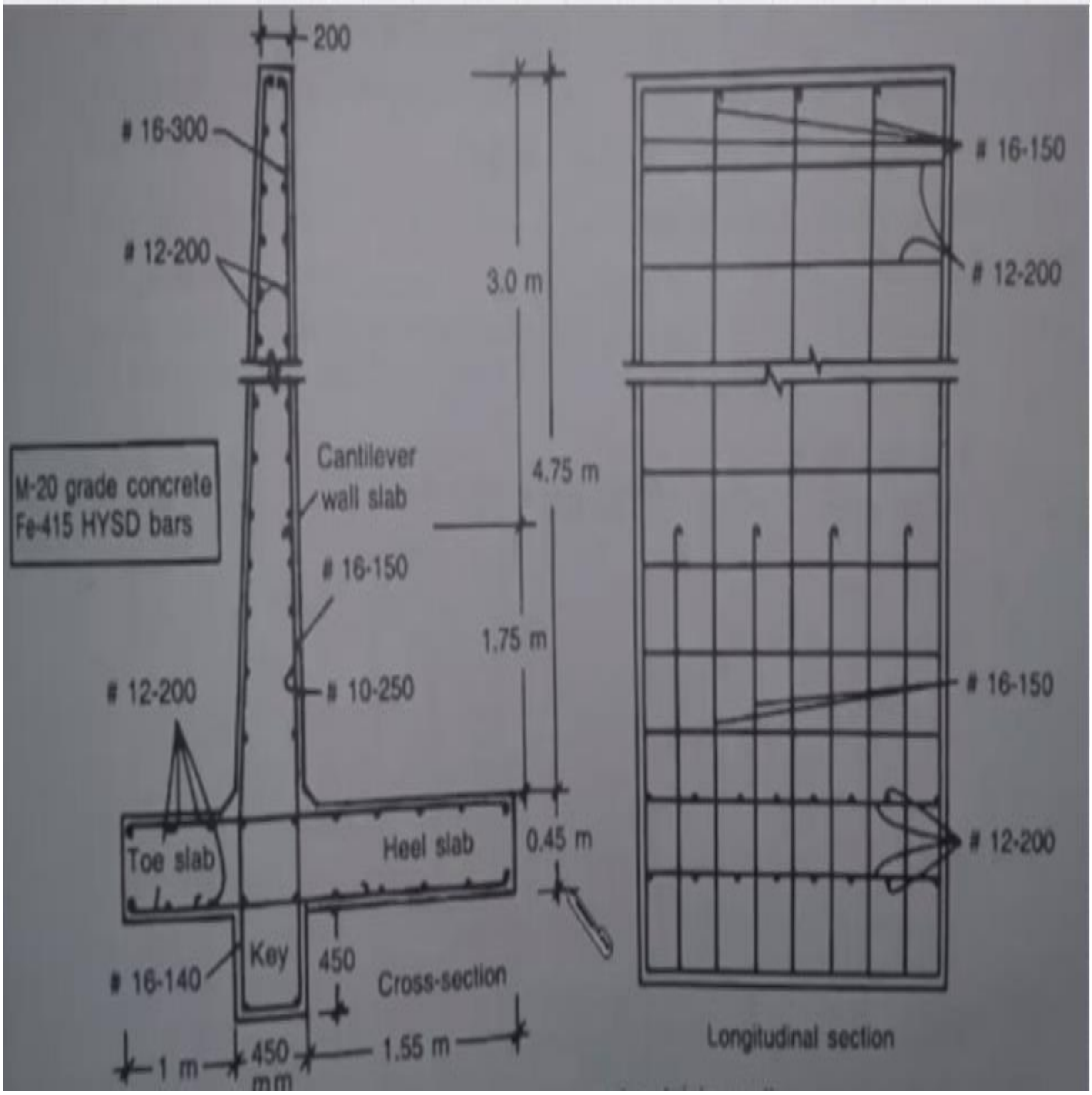
Provide 0.3 % of cross-sectional area for shear key as reinforcement

$= \frac{0.3}{100} \times 450 \times 1000 = 1350mm^2$

Provide 16mm dia @140mmc/c



$\theta = 45 + \phi/2$



Reinforcement detailing of Retaining wall with shear key

Module-2

3 A line diagram of a roof truss with internal loads and forces in each members are shown in Fig.Q.3. Design the various members of the roof truss along with their end connection with bolt using property class 5.6 black bolts. Also design the bearing plate at the support for the reaction and anchor bolts for an uplift force of 15kN. Draw elevation of truss greater than half span. **(50 Marks)**

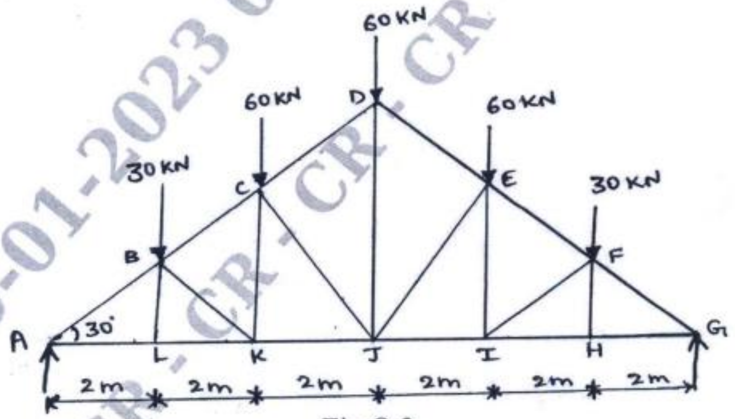


Fig.Q.3

Tabulation of member forces

Members	Length (m)	Force (kN)	Nature of Force
AB, GF	2.31	240.00	Compression
BC, FE	2.31	210.00	Compression
CD, ED	2.31	160.04	Compression
AL, GH	2.00	207.84	Tension
LK, HI	2.00	207.84	Tension
KJ, IJ	2.00	181.82	Tension
BL, FH	1.154	0.00	-
BK, FI	2.31	30.00	Compression
CK, EI	2.31	15	Tension
CJ, EJ	3.05	66.05	Compression
DJ	3.46	66.00	Compression

1. Design a roof truss with tabulation of member forces as shown in Figure. Design various member of roof truss along with their end connections with a gusset plateof 10 mm thick. The truss rest on 300 x 500 mm column made of M 20 grade concrete. Design the support bearing base plate for a reaction of 120 kN and anchor bolt for an uplift force of 18 kN. Draw to suitable scale i) Elevation of trussgreater than half span ii) Enlarged view of support joint iii) Elevation of upper joint of truss.

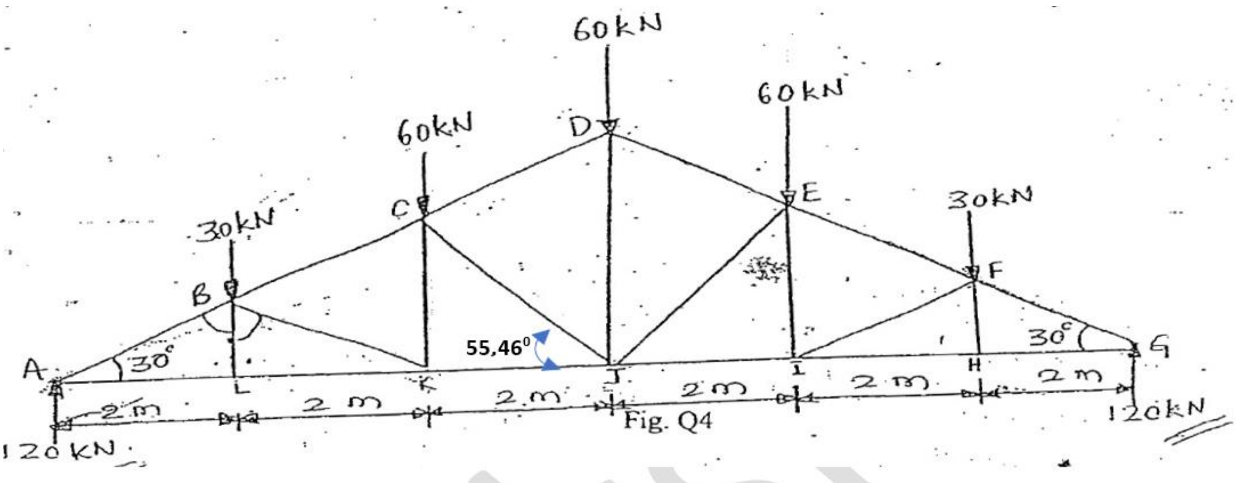


Fig. Q4

Member	Force	Nature of force (N)
AB, GF	240 kN	Compression, C
BC, FE	210 kN	C
CD, ED	160 kN	C
AL, GH	208 kN	Tension, T
LK, HI	208 kN	T
KJ, IJ	182 kN	T
BL, FH	0	-
BK, FI	30 kN	C
CK, EI	18 kN	T
CJ, EJ	66 kN	C
DJ	60 kN	C

Important points to be noted

- Top and Bottom chord – Two angles or Double angles
- Inner members - Single angles
- Use at least two bolts
- Gusset plate thickness should be uniform

- **Design of Top chord members (AB, BC, CD) (Compression members)**

AB, BC, CD are top chord member, maximum load is 240 kN

Factored load or $Force$ = 240 x 1.5 = 360 kN

Length of members AB/ BC/ CD

cos 30 = 2/ AB , AB = 2/ cos 30 = 2.3 m = 2300 mm

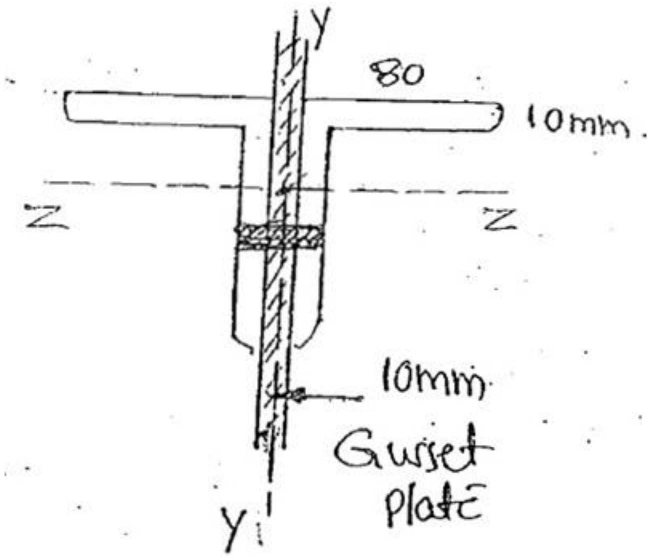
Assume design compressive stress, f_{cd} = 110 N/mm^2

(Assume f_{cd} = 40 -120 N/mm^2 based on load and experience)

Gross area, A_g = $\frac{Force}{f_{cd}} = \frac{360 \times 10^3}{110}$ = 3272.7 mm^2 = 32.73 cm^2

- Select double angle section from steel table

  From Steel Table 6, Page 18, Try 2 ISA 80 x 80 x 10 mm



Area = 30.10 cm^2 , r_{xx} = 2.41 cm Taking gusset plate of 10 mm thickness,

r_{yy} = 3.73 cm

r_{min} = 2.41 cm or 24.1 mm

- Effective length of section - Page 48, CL7.5.2.1, IS 800- 2007

7.5.2 ***Double Angle Struts***

7.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, *KL*, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, *KL*, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive

48

Effective length for Top chord members (AB) , L_{eff} = 0.85 x L = 0.85 x 2.3 = 1.955 m

Slenderness ratio, $\lambda = \frac{L_{eff}}{r_{min}} = \frac{1955}{24.1} = 81.12$

Since it is a built-up member it will come under buckling class "c" (IS 800 – 2007, Page 44, Table 10). If it is class "c", Use Table 9(c)

From Table 9 (c) Page 42 – IS 800 2007, through interpolation, for λ = 81.12 we get design Compressive Stress, $f_{cd} = 134.32\ N/mm^2$

(Page 34) Design compressive strength, $P_c = f_{cd} \times A_g = 134.32 \times 3010$

$= 404.3$ kN > 360 kN

So selected section is safe.

- Connections

Using M 22 Property Class 5.6 bolts

- Shear strength of bolts - Page 75 CL10.3.3 , IS 800 2007

Assume fully threaded bolts, number of shear planes $n_n = 2$, $n_s = 0$ (no shank portion)

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2$, $A_{sb} = 0$, $f_{ub} = 500\ N/mm^2$, $\gamma_{mb} = 1.25$

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

$$= \frac{500}{\sqrt{3} \times 1.25}(2 \times 296.5) = 136.94 \text{ kN….. (1)}$$

- Bearing strength of bolts - Page 75 CL10.3.4 , IS 800 2007

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

Pitch, p = 2.5× d = 2.5 × 22 = 55 mm

Edge distance e = 1.7 × d_o= 1.7× 24 = 40.8 ≈ 45 mm (d_o *is the dia of bolt hole*, (22 + 2)

where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

$f_{ub} = 500\ N/mm^2$, $f_u = 410\ N/mm^2$, t = 10 mm, d = 22 mm

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0

$V_{dpb} = 2.5 \times 0.513 \times 22 \times 10 \times \frac{410}{1.25} = 92.54$ kN……. (2)

Bolt value = Minimum of (1) and (2) = 92.54 kN

No of bolts = $\frac{360}{92.54} = 3.8 \approx 4$

Hence provide 2 ISA 80 × 80 × 10 mm with 4 bolts

- ### Design of Bottom chord members (AL, LK, KJ)-Tension members

  Taking Max Force = 208 kN

  Factored Tensile Force T_{dg} = 208 x 1.5 = 312 kN

  Tensile strength due to gross section yielding, Page 32, CL 6.2 (IS 800)

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

Gross area, $A_g = \frac{312 \times 10^3 \times 1.1}{250} = 1372.8\ mm^2$

Since it is bottom member, increase the area by 30% = 1.3 × 1372.8 = 1784.64 mm^2= 17.84 cm^2

ISA 8080	80 x 80	$\boxed{6.0}$	14.6	143.2	$\boxed{18.58}$	112.0	19.2	2.46
		8.0	19.2	188.4	24.42	145.0	25.2	2.44
		10.0	23.6	231.5	30.10	175.4	31.0	2.41
		12.0	28.0	274.7	35.62	203.8	36.6	2.39

Try 2 ISA 80× 80 × 6 mm (two angles back to back) *with* 10 *mm gap*

$A_g = 18.58\ cm^2 = 1858\ mm^2$

- Connections

Use M 22, class 5.6 (Same bolt diameter for all connection design)

- Shear strength of bolts

Assume fully threaded bolts, number of shear planes $n_n = 2$,(double angle), $n_s = 0$ (no shank portion)

$A_{ns} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2$, $A_{sb} = 0$

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

$$= \frac{500}{\sqrt{3} \times 1.25} (2 \times 296.5) = 136.94 \text{ kN} \ldots (1)$$

- Bearing strength of bolts

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

$$V_{dpb} = 2.5 \times 0.513 \times 22 \times 6 \times \frac{410}{1.25} = 55.53 \text{ kN} \ldots (2)$$

Bolt value = 55.53 kN (least of (1) or (2)

No of bolts = $\frac{312}{55.53} = 6$

Hence provide 2 ISA 80 × 80 × 6 mm with 6 bolts

(In case of High Strength Friction Grip Bolts (HSFG) - Shear capacity only needsto be calculated by CL 10.4.3, page 76 and then calculate no of bolts based on shearcapacity) No need of calculating "bearing strength of bolts".

$V_{dsf} = V_{nsf} / \gamma_{mf}$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o$$

where

μ_f = coefficient of friction (slip factor) as specified in Table 20 ($\mu_f = 0.55$),

n_e = number of effective interfaces offering frictional resistance to slip,

K_h = 1.0 for fasteners in clearance holes,

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot,

= 0.7 for fasteners in long slotted holes loaded parallel to the slot,

γ_{mf} = 1.10 (if slip resistance is designed at service load),

= 1.25 (if slip resistance is designed at ultimate load),

F_o = minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_0$,

A_{nb} = net area of the bolt at threads, and

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

To find partial safety factor

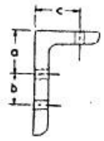
Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

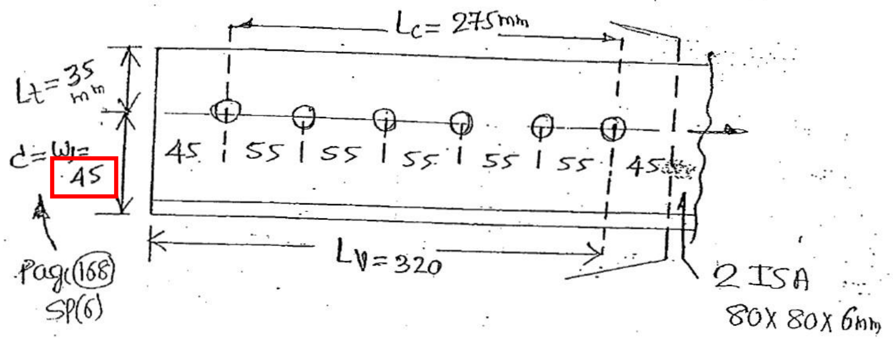
30

TABLE XXXI RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg Size	Double Row of Rivets		Single Row of Rivets	Maximum Rivet Size for Double Row of Rivets
	a	b	c	
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—
45	—	—	25	—
40	—	—	21	—
35	—	—	19	—
30	—	—	17	—
25	—	—	15	—
20	—	—	12	—

SP 6, Page 168



Longitudinal section of a double angle with bolts

- **Check for rupture (tension member)**

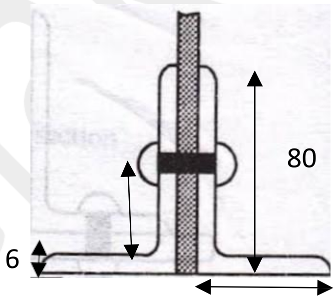
Page 33 CL 6.3.3 IS 800 - 2007

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta A_{go}\, f_y / \gamma_{m0}$$

where

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u \gamma_{m0} / f_y \gamma_{m1}) \ge 0.7$$



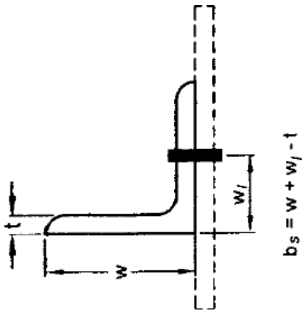
w = outstand leg width or **width of unconnected leg** = 80 mm

L_c = Distance between the outermost bolts in the end joint measured alongthe load direction = 275 mm

t = 6mm

$f_u = 410\ N/mm^2$, Ultimate strength of material

$f_y = 250\ N/mm^2$, Yield strength of material



$b_s = w + w_t - t = 80 + 45 - 6 = 119$ mm

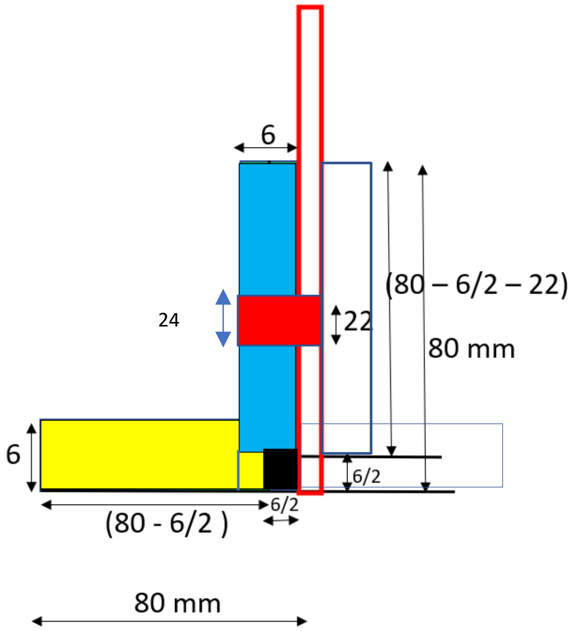
$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c)$$

$= 1.132$

Also find $\dfrac{f_u\,\gamma_{m0}}{f_y\,\gamma_{m1}}$ **= 1.44**

As per IS 800 – 2007, $\beta = 1.132 \geq 0.7 \leq 1.44$

Hence take β = 1.132



Angle section attached to gusset plate

A_{go} = Gross area of outstanding or unconnected leg (without bolt) = $(B - \frac{t}{2})\,t$ = $(80 - \frac{6}{2})\,6 = 462\ mm^2$

A_{nc} = Net area of connected leg (subtract area of bolt hole) = $(A - d_o - \frac{t}{2})\,t$ = $(80 - 24 - \frac{6}{2})\,6 = 318\ mm^2$ where $diameter\ of\ bolt\ hole, d_o = 22 + 2 = 24\ mm$

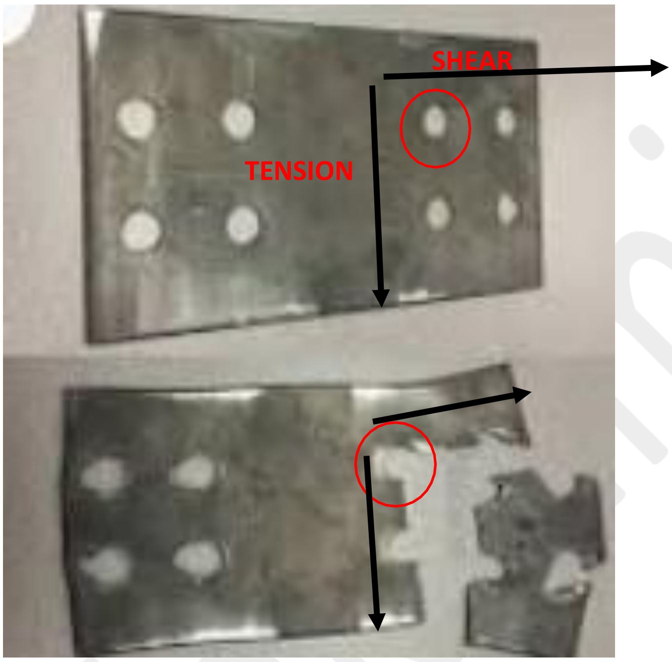
$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta A_{go}\ f_y / \gamma_{m0}$$

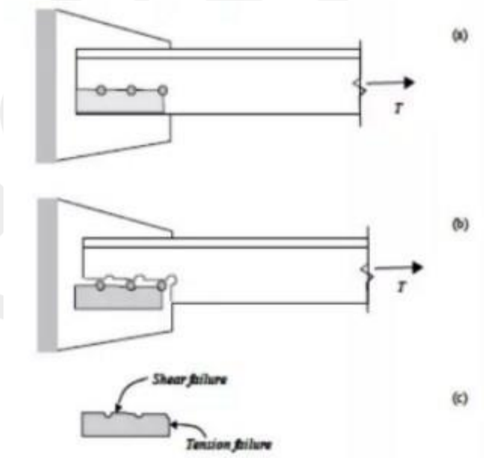
For double angle section multiply the value of T_{dn} by "2"

For double angle $T_{dn} = 2 \times \left(0.9 \times 318 \times \frac{410}{1.25} + 1.132 \times 462 \times \frac{250}{1.1}\right)$

$= 425.47$ kN > 312 kN , It is safe.

- Check for block shear





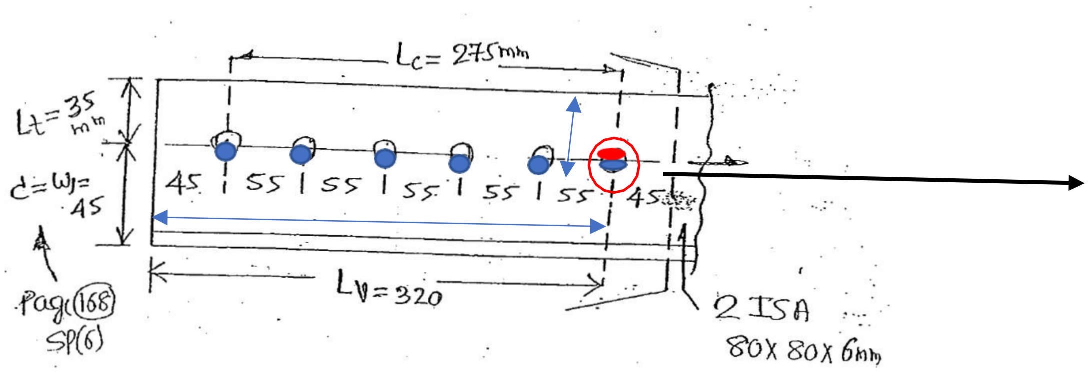
Page 33 CL 6.4.1

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y / (\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

or

$$T_{db} = (0.9 A_{vn} f_u / (\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$



Length of shearing action, L_v = 45 + 5 × 55 = 320 mm

Length of tensile action, L_t = 35 mm

Gross area in shear parallel to force , $A_{vg} = L_v \times t = 320 \times 6 = 1920\ mm^2$

Net area in shear parallel to force, $A_{vn} = A_{vg} - 5.5 \times d_o \times t$

$$= 1920 - 5.5 \times 24 \times 6 = 1128\ mm^2$$

Gross area in tension perpendicular to force, $A_{tg} = L_t \times t = 35 \times 6 = 210\ mm^2$

Net area in tension perpendicular to force, A_{tn} = 210 – 0.5 × 24 × 6 =138 mm^2

$$T_{db} = [A_{vg} f_y/(\sqrt{3}\gamma_{m0}) + 0.9 A_{tn} f_u/\gamma_{m1}]$$

For two angles, $T_{db} = 2 \times [\,1920 \times \frac{250}{\sqrt{3} \times 1.1} + 0.9 \times 138 \times \frac{410}{1.25}\,]$ = 585.3 kN > 312 kN

$$T_{db} = (0.9 A_{vn} f_u/(\sqrt{3}\gamma_{m1}) + A_{tg} f_y/\gamma_{m0})$$

For two angles, $T_{db} = 2 \times [\,0.9 \times 1128 \times \frac{410}{1.25 \times \sqrt{3}} + 210 \times \frac{250}{1.1}\,]$ = 479.9 kN> 312 kN

Hence 2 ISA 80 × 80 × 6 mm is safe .

- Design of "Inner Tension members"

  Member Force CK = 18 kN

  Factored Force, T_{dg}= 1.5 × 18 = 27 kN

  Tensile strength due to gross section yielding, Page 32, CL 6.2 (IS 800)

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

  A_g = **118.8 mm²** = 1.18cm²

  Use Steel table and Take single angle section (it is a inner member)

  Table 1, Page 2 , Choose ISA 50 X 50 X 6 mm ,

  Area = 5.68 cm² =568 **mm²**

- Connection – Bolts - M 12 Class 5.6

  Diameter of bolt hole $d_o = 12 + 1 = 13\ mm$

- Shear strength of bolts

  Assume fully threaded bolts, number of shear planes n_n = 1 (single angle section) , n_s = 0 (no shank portion)

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 12^2 = 88.2\ mm^2$, $A_{sb} = 0$, , f_{ub} = 500 N/mm²

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

0

$$= \frac{500}{\sqrt{3} \times 1.25}(1 \times 88.2) = 20.36 \text{ kN…..(1)}$$

- Bearing strength of bolts

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

Pitch, p = 2.5× d = 2.5× 12 = 30 mm, t = 6 mm

Edge distance e = 1.7 × d_o= 1.7× 13 = 22.1 = 25 mm

where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

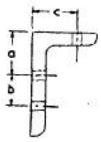
Smallest of : $k_b = \frac{25}{3 \times 13} = 0.64$, $k_b = \frac{30}{3 \times 13} - 0.25 = 0.519$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0

$V_{dpb} = 2.5 \times 0.519 \times 12 \times 6 \times \frac{410}{1.25} = 30.64$ kN…..(2)

Bolt value = 20.36 kN (LEAST OF (1) and (2))

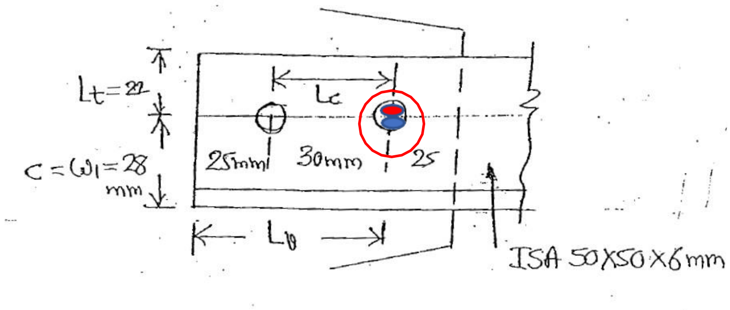
No of Bolts = 27/ 20.36 = 2 (Minimum no of bolts =2)

TABLE XXXI RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg Size	Double Row of Rivets a	Double Row of Rivets b	Single Row of Rivets c	Maximum Rivet Size for Double Row of Rivets
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—

Use SP6, Page 168, Note down gauge distance for leg size 50 mm, which is 28 mm



- Check for rupture

  w, width of unconnected leg = 50 mm, t = 6 mm, distance between end bolts, Lc = 30 mm,

  $b_s = w + w_t - t$

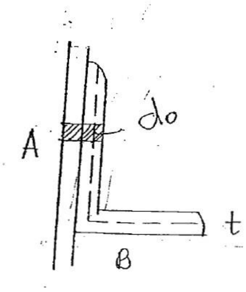
  bs = 50 +28 - 6 = 72 mm

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, $T_{dn,}$ as governed by rupture at net section is given by:

$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta A_{go}\, f_y / \gamma_{m0}$$

where

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u \gamma_{m0} / f_y \gamma_{m1})$$
$$\ge 0.7$$



A_{go} = Gross area of unconnected leg $= (B - \frac{t}{2})\, t = (50 - \frac{6}{2})6 = 282\ mm^2$

A_{nc} = Net area of connected leg $= (A - d_o - \frac{t}{2})\, t = (50 - 13 - \frac{6}{2})\,6 = 204\ mm^2$

Also calculate β = 1.4 – 0.076 x (50 /6) x (250/410) x (72/30) = 0.47 ≤ 0.7

$\frac{\gamma_{m0} f_u}{f_y \gamma_{m1}} = 1.44$, The value of β = 0.47 ≤ 1.44 .

Let us take β = 0.7, since calculated value of β is less than 0.7

For single angle $T_{dn} = 1 \times (0.9 \times 204 \times \frac{410}{1.25} + 0.7 \times 282 \times \frac{250}{1.1}) =$

= 105.08 kN >27 kN, it is safe.

- Check for block shear

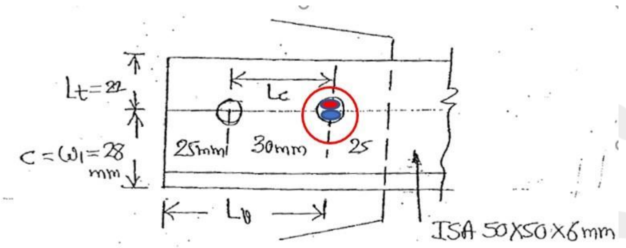
  Page 33 CL 6.4.1

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y /(\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

or

$$T_{db} = (0.9 A_{vn} f_u /(\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$



$A_{vg} = L_v \times \text{t} = (30 + 25) \times 6 = 330\ mm^2$ $A_{vn} = 330 - 1.5 \times 13 \times 6 = 213\ mm^2$

$A_{tg} = L_t \times \text{t} = 22 \times 6 = 132\ mm^2$ $A_{tn} = 132 - 0.5 \times 13 \times 6 = 93\ mm^2$

$$T_{db} = [A_{vg} f_y /(\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

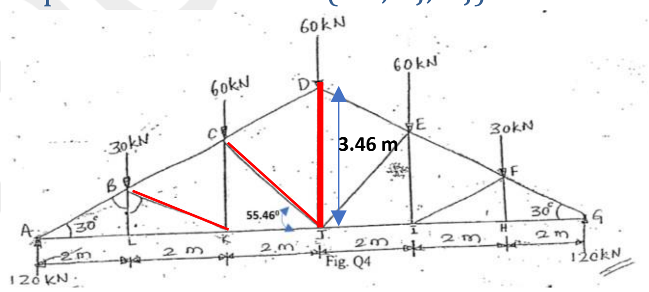
For single angles, $T_{db} = 1\times [\,330 \times \frac{250}{1.1\times\sqrt{3}} + 0.9 \times 93 \times \frac{410}{1.25}\,] = 70.75\text{kN} > 27\text{ kN}$

$$T_{db} = (0.9 A_{vn} f_u /(\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$

For two angles, $T_{db} = 1 \times [\,0.9 \times 213 \times \frac{410}{1.25\times\sqrt{3}} + 132 \times \frac{250}{1.1}\,] = 66.30\text{ kN} > 27\text{ kN}$

Hence ISA 50 × 50 × 6 mm is safe .

- Design of Inner compression members (BK, CJ, DJ)



Taking Maximum Force = 66.0 kN

Factored Force = 1.5 × 66.0 = 99. kN

Take the member which is having maximum length ie DJ = 3.46 m

Maximum Length = 3.46 m

Assume f_{cd} = 50 N/mm^2

(Assume f_{cd} = 40 -120 N/mm^2 based on load and experience)

Gross Area, $A_g = \frac{Factored\ Force}{f_{cd}} = \frac{99.07 \times 10^3}{50} = 1981.4\text{ mm}^2 = 19.81\text{ cm}^2$

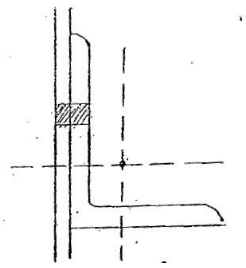
From Steel Table, Try Single ISA 100 x 100 x 10 mm (Table 1, Page 4)

Area = 19.03 cm^2

r_{xx} = r_{yy} = 3.05 cm = 30.5 mm

r_{uu} = 3.85 cm, r_{vv} = 1.94 cm

So r_{min} = 1.94 cm = 19.4 mm



Here load is acting through only one leg it will be subjected to torsional buckling

Using Page 48, Table 12 and using CL 7.5.1.2 Loaded through one leg (IS 800 2007)

7.5.1.2 *Loaded through one leg*

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e, as given below:

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

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio ($250/f_y)^{0.5}$.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

[1] Stiffeness of in-plane rotational restraint provided by the gusset/connecting member.
For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.

Assuming bolts ≥ 2 and hinged end conditions with gusset plate, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$
Assuming Effective length, $L_{eff} = 0.85 \times L = 0.85 \times 3460 = 2940\ mm$
$\varepsilon = 1$, $E = 2 \times 10^5$ N/mm^2 $b_1 = b_2 = 100$ mm, $t = 10$ mm

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

$$\lambda_{vv} = \frac{\frac{Leff}{rvv}}{\varepsilon\sqrt{\frac{\pi^2\varepsilon}{250}}} = 1.705$$

$$\lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

$\lambda_\Phi = 0.1125$, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$

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

Equivalent slenderness ratio, $\lambda_e = 1.589$
From Table 10, Page No 44, Choose the buckling class based on type of section

IS 800 : 2007

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections		Any	c

Since it is single angle section, choose buckling class as 'c'. Based on Buckling class, find α from Table 7 Page No 35 as 0.49.

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

From Page 34, CL 7.1.2.1, IS 800-2007

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

Put $\alpha = 0.49$, $\lambda_e = 1.589$, $\Phi = 2.102$

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

(Single angle section) Design compressive stress, $f_{cd} = \frac{250}{1.1(2.102 + (2.102^2 - 1.589^2)^{0.5})} =$ 108.47 N/mm^2

The design compressive strength of a member is given by:

Load $P_d = A \times f_{cd} = 1903 \times 108.47 = 206.42$ kN > 99.07 kN

Hence it is safe.

Design of connection using M22, class 5.6 (same diameter bolt)

Benefit- No need to do shear strength calculations, (for single angle, n_n = 1)

- Shear strength of bolts

Assume fully threaded bolts, number of shear planes n_n = 1 (Single angle section), n_s = 0 (no shank portion)

$$A_{ns} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2 \quad , A_{sb} = 0$$

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

$$= \frac{500}{\sqrt{3} \times 1.25}(1 \times 296.5) = 68.47 \text{ kN….. (1)}$$

- Bearing strength of bolts

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

Pitch, p = 2.5× d = 2.5× 22 = 55 mm

Edge distance e = 1.7 × d_o= 1.7× 24 = 40.8 = 45 mm

where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{fub}{fu} = \frac{500}{410} = 1.22$, 1.0

$$V_{dpb} = 2.5 \times 0.513 \times 22 \times 10 \times \frac{410}{1.25} = 92.54 \text{kN…..(2)}$$

Bolt value = Minimum of (1) and (2) = 68.47kN

No of bolts = $\frac{99}{68.47}$ = 2

- Design of Bearing Plate

Support Reaction = 120 kN

Factored Reaction = 1.5 × 120 = 180 kN

CL 34.4, Page 56 IS 456 2000

Using M 20 concrete, Bearing pressure of concrete = 0.45 f_{ck} = 0.45 × 20 = 9 N/mm^2

IS 456 : 2000

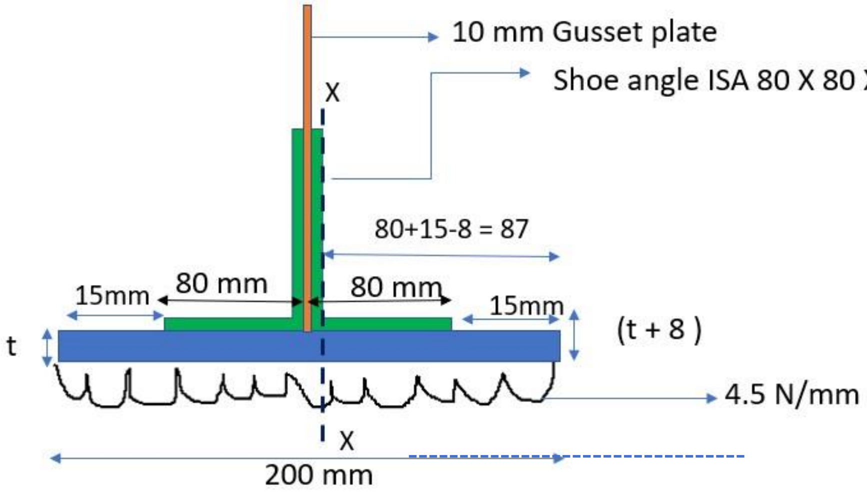
For working stress method of design the permissible bearing stress on full area of concrete shall be taken as $0.25 f_{ck}$; for limit state method of design the permissible bearing stress shall be $0.45\ f_{ck}$.

Area of Bearing plate = $\frac{180 \times 10^3}{9}$ = 20000 mm^2

Provide square plate = $\sqrt{20000}$ = 141.4 mm

Provide 200 mm × 200 mm as base dimensions

- Calculation of thickness ‘t’ of the bearing plate



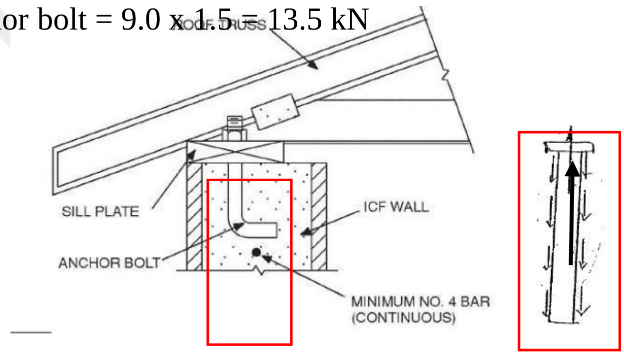
Upward pressure, q_o = Support Reaction / Size of plate = 180 x 10^3/ (200 x 200) = 4.5 N/mm^2
For 1mm, it is 4.5 N/mm
Moment at section XX (near the inner face of angle section), $M_{xx} = 4.5 \times 87 \times \frac{87}{2} =$ $17030.25\ Nmm$
Take b = 1mm, d = (t + 8)

Using Bending formula $M_{xx} = \sigma_b \times Z = \frac{f_y}{\gamma_{mo}} \times b \times \frac{d^2}{6}$ (Z is section modulus)

$17.03 \times 10^3 = \frac{250}{1.1} \times 1 \times \frac{(t+8)^2}{6}$
t = 13.20 mm ≈ 14 mm
Bearing plate 200 × 200 × 14 mm

- Design of Anchor Bolts

Given Uplift force = 18 kN at each support
Assuming two Anchor bolts at each support, Force on each anchor bolt = 18/2 =9.0 kN
Factored force on anchor bolt = 9.0 x 1.5 = 13.5 kN



From CL 26.2.1.1 Bond stress for M 20 (RCC column M 20) = **1.2** N/mm^2, increase by **60**% (Page 43, IS 456 2000) = 1.2 x 1.6

IS 456 : 2000

26.2.1.1 Design bond stress in limit state method for plain bars in tension shall be as below:

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress, τ_{bd}, N/mm²	1.2	1.4	1.5	1.7	1.9

For deformed bars conforming to IS 1786 these values shall be increased by 60 percent.

For bars in compression, the values of bond stress for bars in tension shall be increased by 25 percent.

The values of bond stress in working stress design, are given in B-2.1.

2) In the compression zone, from the mid depth of the beam.

b) *Stirrups*—Notwithstanding any of the provisions of this standard, in case of secondary reinforcement, such as stirrups and transverse ties, complete development lengths and anchorage shall be deemed to have been

To find length of Anchor bolt

Let us assume 18 mm diameter Anchor Bolt

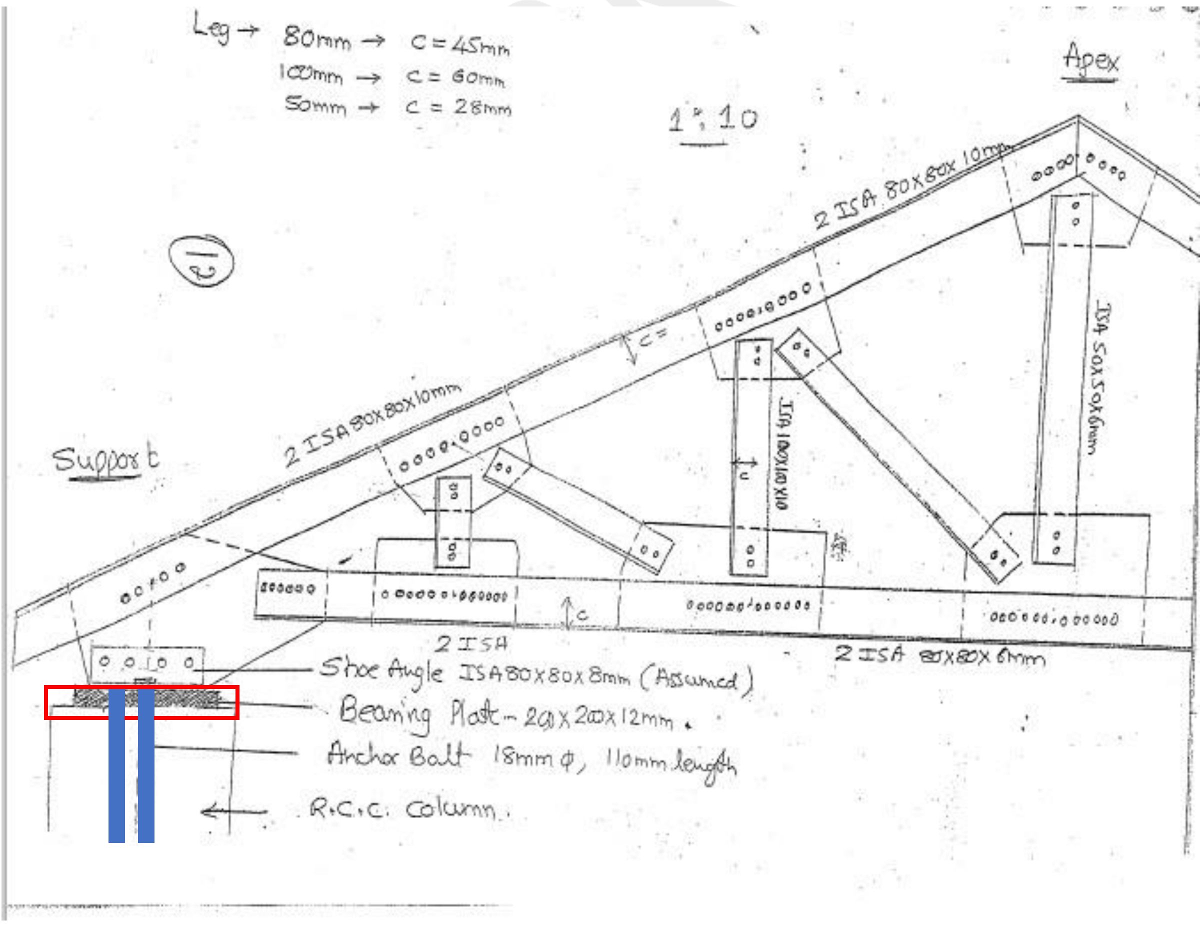
Factored Uplift Force = Force developed around the anchor bolt

$$= (\text{Circumference} \times \text{length}) \times \text{Bond stress}$$

$$13.5 \times 10^3 = \pi \times 18 \times \text{length} \times (1.2 \times \mathbf{1.6})$$

$\text{length} = 124.34 \text{ mm} \approx 125 \text{ mm}$

Provide 2, 18 mm Φ anchor bolts of length 125 mm at each end.



OR

4 Design a simply supported gantry girder to carry an electrically operated travelling crane with the following data:
Span of crane bridge = 25m
Column spacing = span of gantry girder = 8m
Wheel Base = 3.5m
Crane capacity = 200kN
Weight of crane bridge = 150kN
Weight of Trolley = 75kN
Min Hook Distance = 1.0m
Weight of Rail = 0.30kN/m
Height of Rail = 105mm
Also draw sectional elevation. (50 Marks)

*****

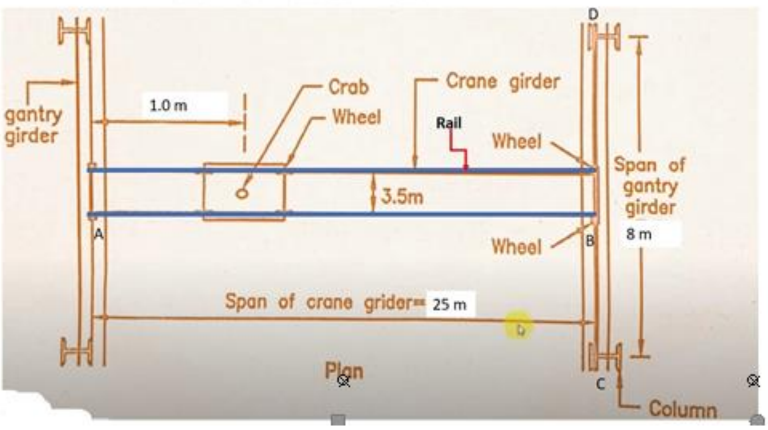
DESIGN OF GANTRY GIRDER

1. Design a gantry girder to be used in an industrial building carrying an electrically operated overhead travelling crane, for the following data

- **Centre to Centre between distance between gantry rails or span of crane girders 25 m**
- **Centre to Centre distance between columns (span of gantry girder) 8 m**
- **Crane capacity 200 kN**
- **Self-weight of the crane girder excluding trolley 150 kN at centre or (150/25 = 6kN/m)**
- **Self-weight of the crab or trolley, electric motor, hook, etc. 75 kN**
- **Approximate minimum approach of the crane hook to the gantry girder 1.0 m**
- **Wheel base 3.5 m**
- **Self-weight of rail section 300 N/m =0.3kN/m**
- **Diameter of crane wheels 150 mm**
- **Height of rail 105 mm**
- **Steel is of grade Fe 410. Design also the field welded connection if required.**
- **The support bracket connection need not be designed.**

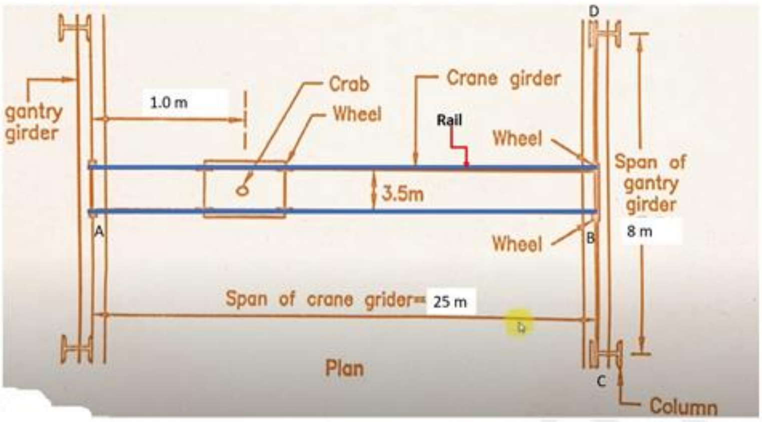
Solution For Fe 410 grade of steel: $f_u = 410$ MPa, $f_y = f_{yw} = f_{yf} = 250$ MPa

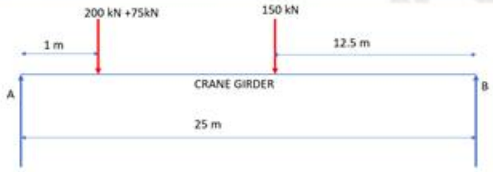
Design Maximum load transferred from crane girder to gantry girder



The maximum reaction/load in crane girder occurs when the crab or trolley along with hook if it istowards left or right of crane girder with a minimum hook distance of 1.0 m.

The Free body diagram is shown as below.





To find max Reaction R_A , take moment at B, $\sum M_B = R_A \times 25 - 150 \times 12.5 - 275 \times 24 = 0$

R_A = 339kN is the reaction developed at the end of Crane girder

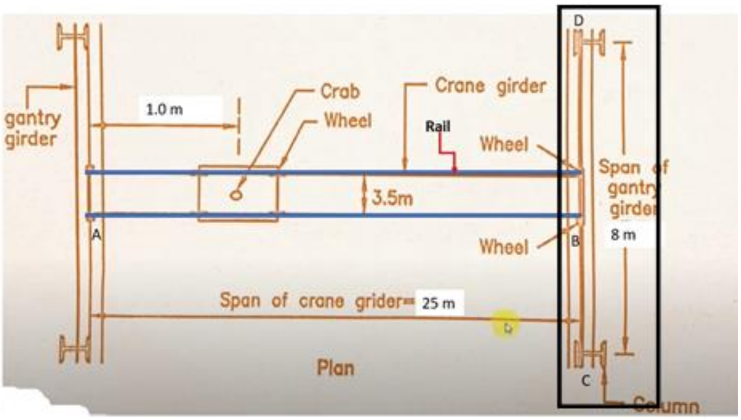
Since there are two wheels at each end of crane bridge, **load on each wheel of trolley or crab** = 339/2 = 169.5kN

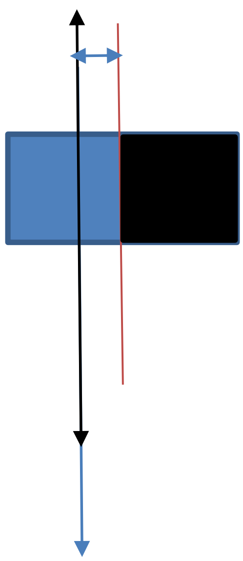
As per IS: 875 (part 2)-1987, for Electrically operated crane (EOT crane), (CL 6.1, Page 15)

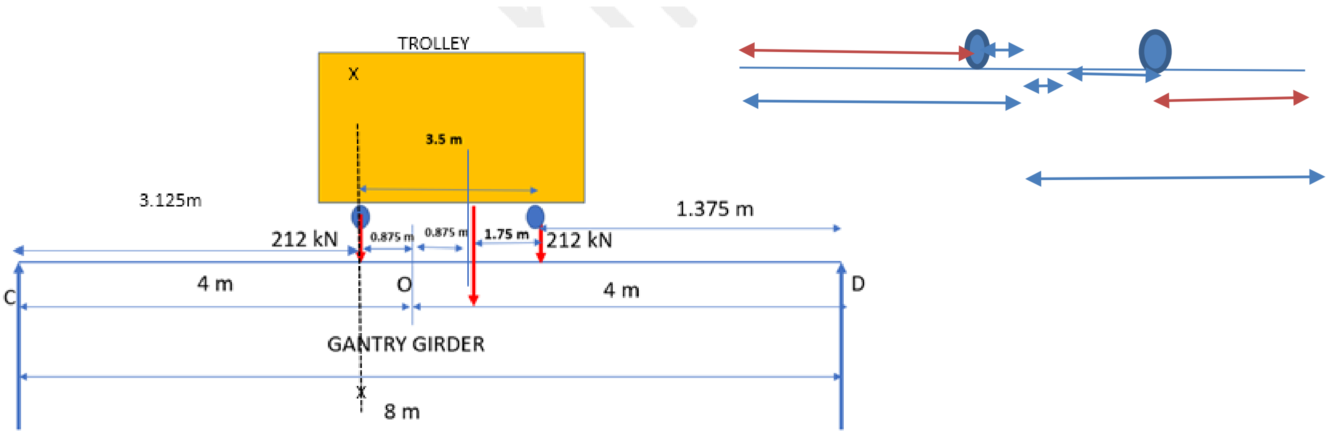
Vertical forces transferred through rails = 25% of **maximum static** wheel load (25% means 1.25)

Load on each wheel = $1.25 \times 169.5 = 211.87$ kN $\approx$ 212 kN

- Design Maximum bending moments on gantry girder







TROLLEY
X
3.5 m
3.125m
1.375 m
212 kN
0.875 m
0.875 m
1.75 m
212 kN
C
4 m
O
4 m
D
GANTRY GIRDER
X
8 m

The arrangement of wheel loads for Maximum bending moment is at **Centre of gravity of the wheelloads(trolley)** and one of the wheel load are **equidistant from the Centre of the gantry girder**

$\sum M_D = R_c \times 8 - 212 \times 1.375 - 212 \times (1.375 + 1.75 + 1.75) = 0$, $R_c = 165.63$ kN

B M under a wheel load or Maxi B M at XX = $R_c \times 3.125 = 165.63 \times 3.125 = 517.6$ kNm

Factored B M = 517.6 × 1.5 = 776.4 kNm

B M and S F due to self-weight or dead load

Assume self-weight of gantry girder as 1.6kN/m

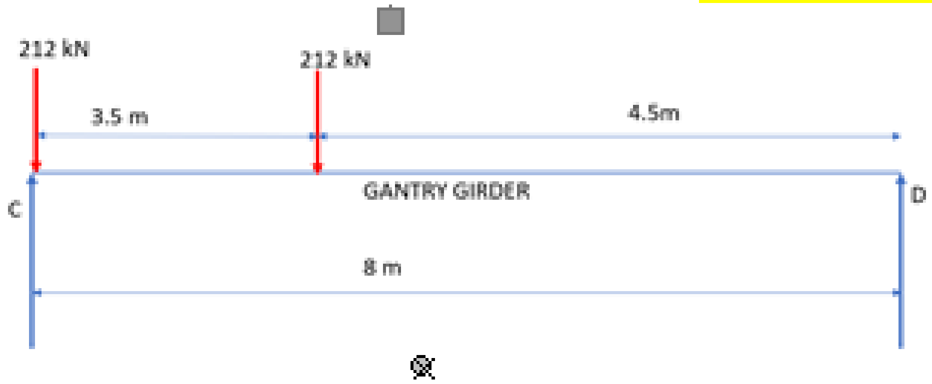
Self-weight of rail = 300N/m = 0.3kN/m (given)

Total self-weight = 1.6 + 0.3 = 1.9 kN/m, factored self-weight = 1.5 x 1.9 = 2.85 kN/mB M due to self-weight or dead load = 2.85 x 8^2/8 = 22.8kNm

S F due to self-weight or dead load = 2.85 x 8/2 = 11.4kN

- Design Maximum shear force

The shear due to the wheel load is maximum when one of the wheels is at the support



$\sum M_D = R_c \times 8 - 212 \times 8 - 212 \times 4.5 = 0$,

$R_c = 331.25$

Factored S F = 331.25 × 1.5 = 496.88 kN

- Lateral load and its moment

Lateral load is developed due to the application of brakes or sudden acceleration of the trolley

As per IS: 875 (part 2)-1987, Lateral load or Horizontal forces transverse to the rails = 10% of the weight of the crab or trolley and the weight lifted on the crane(crane capacity)

Lateral force or Horizontal force = 10 /100 × (200 +75) = 27.5 kN

Lateral load acting on each wheel (there are 4 wheels) = 27.5 /4 = 6.875 ≈ 7kN

Factored lateral load = 7 × 1.5 = 10.5kN

Bending Moment due to lateral load

For 212 kN , B.M = 776.4kNm

Then for 10.5 kN, B.M due to lateral load =?

212/10.5 = 776.4/ ? , ? =

$\frac{212}{10.5} = \frac{776.4}{?}$

Bending moment due to lateral load? = 38.46kNm

Total Design bending moment (Live Load + Lateral load + Dead Load) = 776.4 + 38.46 + 22.8 = 837.76 kNm

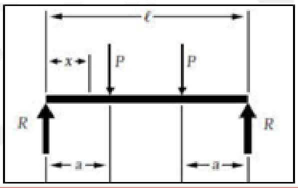
Total Design shear force (Live Load + Lateral load + Dead Load) = 496.88 +11.4+7 = kN

- **preliminary trial section**

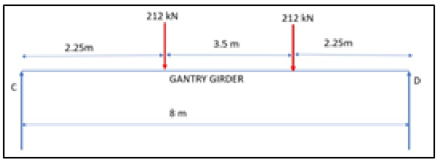
The trial section is selected based on deflection criteria, IS 800 2007, Table 6 Page 31

Table 6 Deflection Limits

Type of Building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm



$$\Delta_{max} \text{ (at center)} \ldots\ldots\ldots\ldots = \frac{Pa}{24EI}(3\ell^2 - 4a^2)$$



Maxi deflection for electrically operated crane, $\delta_{\max deflection}$ = Span of gantry girder/750

= 8000/750 = 10.67 mm

But Maxi deflection = Deflection due to Dead load + Deflection due to Live load

Let us assume deflection due to dead load $\delta_{dead\ load}$ as 1 mm (Since it is steel structure self-weight is very less, we are assuming very less deflection also)

Deflection due to live load $\delta_{live\ load}$ under two equal concentrated loads (two equal wheel loads)

Taking $\Delta_{max} = \delta_{live\ load}$

$$\delta_{live\ load} = \frac{Pa}{24EI}(3l^2 - 4a^2)$$

Put $P = 212kN$, E = 2 × 10 ^{5} N/mm^2 , a = 2.25 m, l = 8 m

$$\delta_{\max deflection} = \delta_{dead\ load} + \delta_{live\ load}$$

$$10.67 = 1.00 + \frac{Pa}{24EI}(3l^2 - 4a^2)$$

We need to calculate Moment of Inertia I for the selecting the section for gantry girder

$$10.67 = 1.00 + \frac{212\,000 \times 2250}{24 \times 2 \times 10^{5} \times I}(3 \times 8000^{2} - 4 \times 2250^{2})$$

I = 1765 x 10 ^{6} mm^{4}

Increase the value of I by 30% = 1.3 x 1765 x 10 ^{6} = 2294.5 x 10 ^{6} mm^{4} = 229450 cm^{4}

Take steel table and select a built-up section from Page 42 and Table 12 based on I_{xx} value

Selecting ISWB 500 @ 95.2kg/m, ISMC 400 @ 49.4kg/m as section for Gantry girder (Table 12, Page 42) for I_{xx} = 230194 cm^{4} = 2301.9 x 10 ^{6} mm^{4}

42 STEEL TABELS

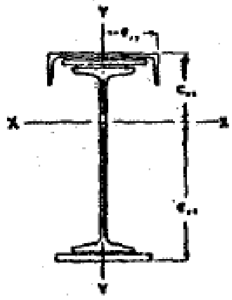


TABLE 12 *(Contd.)*

SINGLE JOIST WITH CHANNEL AND PLATES ON THE FLANGES (GIRDERS)

Joist Designation	Joist w	Top Flange Channel Designation	Channel w	Top Flange Plate Width × Thickness	Bottom Flange Plate Width × Thickness	Weight per Metre (W)		Sectional Area a	Centre of Gravity Cxx	Mean Thickness of Flanges Top	Bottom
	Kg/N		Kg/N	mm mm	mm mm	kg	N	cm²	cm	mm	mm
ISWB 500	95.2	ISMC 400	49.4	320 × 10.0	320 × 20.0	219.9	2157.2	280.15	24.40	25.8	31.5
	933.9		484.6	12.0	25.0	237.5	2329.9	302.55	25.63	27.4	36.5
				16.0	32.0	265.1	2600.6	337.75	26.95	30.6	43.5
				20.0	40.0	295.3	2896.9	376.15	28.37	33.8	51.5
		ISMC 350	42.1	250 × 10.0	320 × 20.0	207.1	2031.7	263.88	25.74	25.7	31.5
			413.0	12.0	25.0	223.6	2193.5	284.88	27.06	27.2	36.5
				16.0	32.0	249.1	2443.7	317.28	28.52	30.0	43.5
				20.0	40.0	277.0	2717.4	352.88	30.06	32.9	51.5
		ISMC 400	49.4	—	320 × 10.0	169.7	1664.8	216.15	22.81	17.8	21.5
			484.6	—	12.0	174.7	1713.8	222.55	23.65		23.5
				—	16.0	184.7	1811.9	235.35	25.21		27.5
		ISMC 350	42.1	—	320 × 10.0	162.4	1593.1	206.88	23.69	21.5	21.5
			413.0	—	12.0	167.4	1642.2	213.28	24.54	18.6	23.5
				—	16.0	177.5	1741.3	226.08	26.11	27.5	27.5
ISMB 450	72.4	ISMC 300	35.8	—	250 × 10.0	127.9	1254.7	162.91	20.93	16.3	20.4
	710.2		351.2	—	12.0	131.8	1293.0	167.91	21.71		22.4
				—	16.0	139.7	1370.5	177.91	23.14		26.4
		ISMC 250	30.4	—	250 × 10.0	122.4	1200.7	155.94	21.71	17.5	20.4
			298.2	—	12.0	126.3	1239.0	160.94	22.49		22.4
				—	16.0	134.2	1316.5	170.94	23.93		26.4
		ISMC 225	25.9	—	200 × 10.0	114.0	1118.3	145.28	21.57	18.0	23.0
			254.1	—	12.0	117.2	1149.7	149.28	22.25		25.0
				—	16.0	123.5	1211.5	157.28	23.51		29.0

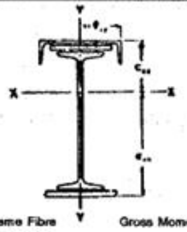


TABLE 12 *(Contd.)*

SINGLE JOIST WITH CHANNEL AND PLATES ON THE FLANGES (GIRDERS)

Extreme Fibre Distance		Gross Moments of Inertia			Radius of Gyration	Moduli of Section		Maximum Allowable Moment	Maximum Allowable Shear
e_{xx}	e_{yy}	I_{xx}	I_{yy} Whole Section	I_{yy} Top Flange Only	r_{yy}	Z_{xx} Zc	Z_{xx} Zt	M	S
cm	cm	cm⁴	cm⁴	cm⁴	cm	cm³	cm³	kg.m × 10³	kg × 10³
29.46	20.00	152781.2	26262.6	19307.2	9.68	6262.3	5185.5	81.7	46.8
28.93		170717.9	28174.1	19853.4	9.65	6661.7	5900.4	92.9	
28.71		198797.6	31177.8	20945.7	9.61	7377.1	6923.9	109.1	
28.49		230194.5	34454.6	22038.1	9.57	8114.6	8079.2	127.2	
28.07	17.50	143834.0	19759.2	12803.9	8.65	5587.0	5124.9	80.7	
27.45		159945.7	21385.0	13064.4	8.66	5911.3	5826.3	91.8	
27.09		184963.8	23817.3	13585.3	8.66	6485.6	6827.6	107.5	
26.75		212768.9	26522.6	14106.2	8.67	7077.7	7954.5	125.3	
29.05	20.00	106172.6	20801.3	16576.5	9.81	4654.5	3654.9	57.6	
28.41		111454.1	21347.4	16576.5	9.79	4712.8	3922.9	61.8	
27.25		121362.3	22439.7	16576.6	9.76	4815.0	4452.9	70.1	
28.12	17.50	101911.5	15726.5	11501.7	8.72	4301.4	3624.5	57.1	
27.47		106854.4	16272.6	11501.8	8.73	4354.4	3889.8	61.3	
26.30		116100.2	17364.9	11501.9	8.76	4447.3	4413.8	69.5	
25.83	15.00	62983.5	8498.7	6779.4	7.22	3008.6	2438.8	38.4	40.0
25.25		66244.2	8759.1	6779.5	7.22	3051.8	2623.1	41.3	
24.22		72359.1	9279.9	6779.6	7.22	3127.4	2987.2	47.0	
25.00	12.50	60394.7	5952.9	4233.7	6.18	2781.6	2416.0	38.1	
24.42		63446.4	6213.3	4233.8	6.21	2820.9	2598.3	40.9	
23.38		69152.5	6734.1	4233.8	6.28	2889.5	2958.0	46.6	
25.07	11.25	55138.5	4195.3	3111.5	5.37	2556.1	2199.5	34.6	
24.59		57604.5	4328.6	3111.5	5.38	2589.5	2342.2	36.9	
23.73		62272.6	4595.3	3111.6	5.41	2649.1	2623.8	41.3	

Selecting ISWB 500 @ 95.2kg/m, ISMC 400 @ 49.4kg/m as section for Gantry girder(Table 12,Page 43)

A = 376.15 cm^2 = **37615 mm^2** , r_{yy} = 95.7 mm, C_{xx} = 283.7 mm (from Top), e_{xx}= 284.9mm (FromBottom)

Sectional properties of I section and Channel section used in Gantry Girder

ISWB 500 @ 95.2kg/m (Table 4, Page 14 Steel Table)	ISMC 400 @ 49.4kg/m (Table 5, Page 16, Steel Table)
$A = 121{\cdot}22\ cm^2$	$A = 62.93 cm^2 = 6293\ mm^2$
h = 500 mm	h = 400 mm
b = 250 mm	b = 100 mm
t_f = 14.7 mm	t_f =15.3 mm
t_w = 9.9 mm	t_w =8.6 mm
r_{xx} = 207.7mm, r_{yy} = 49.6 mm	C_{yy} = 24.2 mm

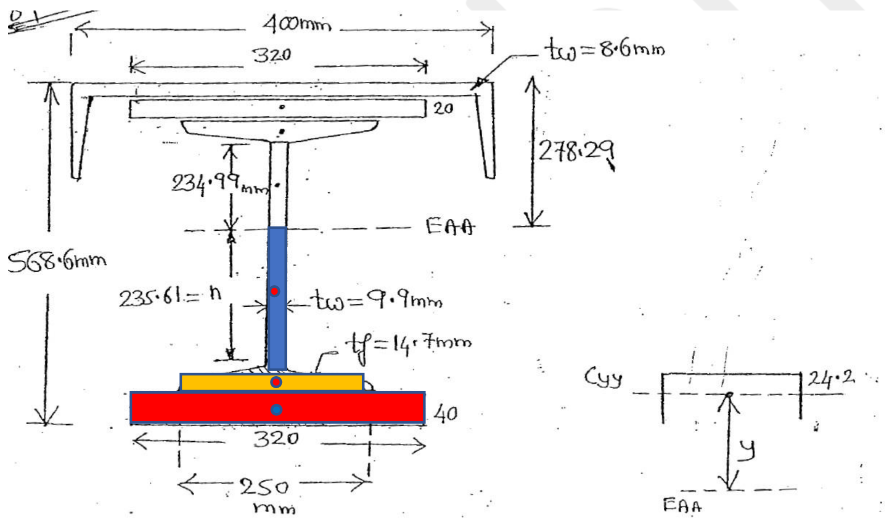
Location of Equal area axis

Equal area axis is the location of the **axis** which results in **equal** compressive and tensile forces when all fibres in a section have reached yield stress

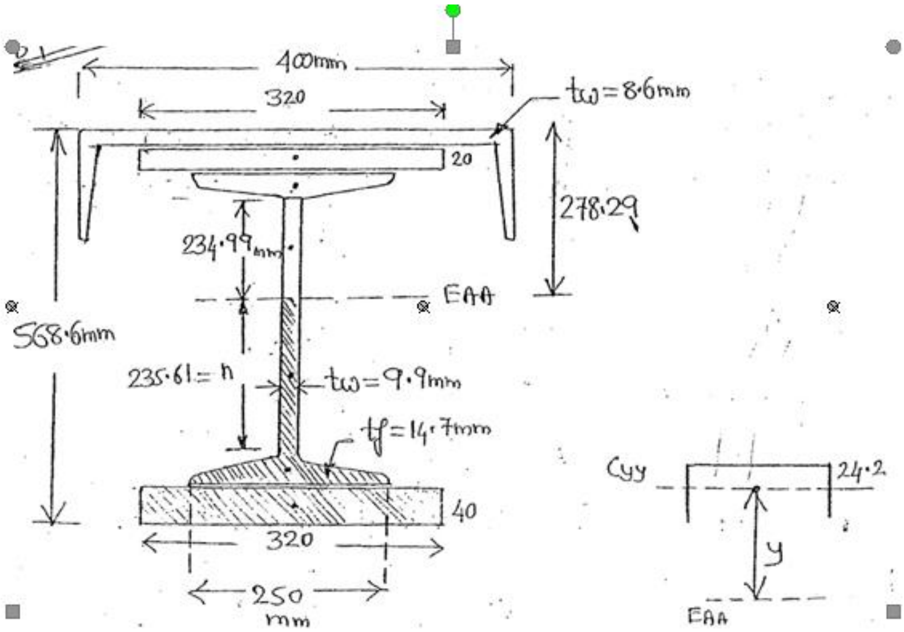
To locate Equal area axis '**n**' , Area of shaded portion = ½ total area of section

$9.9 \times \mathbf{n} + 250 \times 14.7 + 320 \times 40 = \frac{1}{2} \times (37615)$

n = 235.61 mm

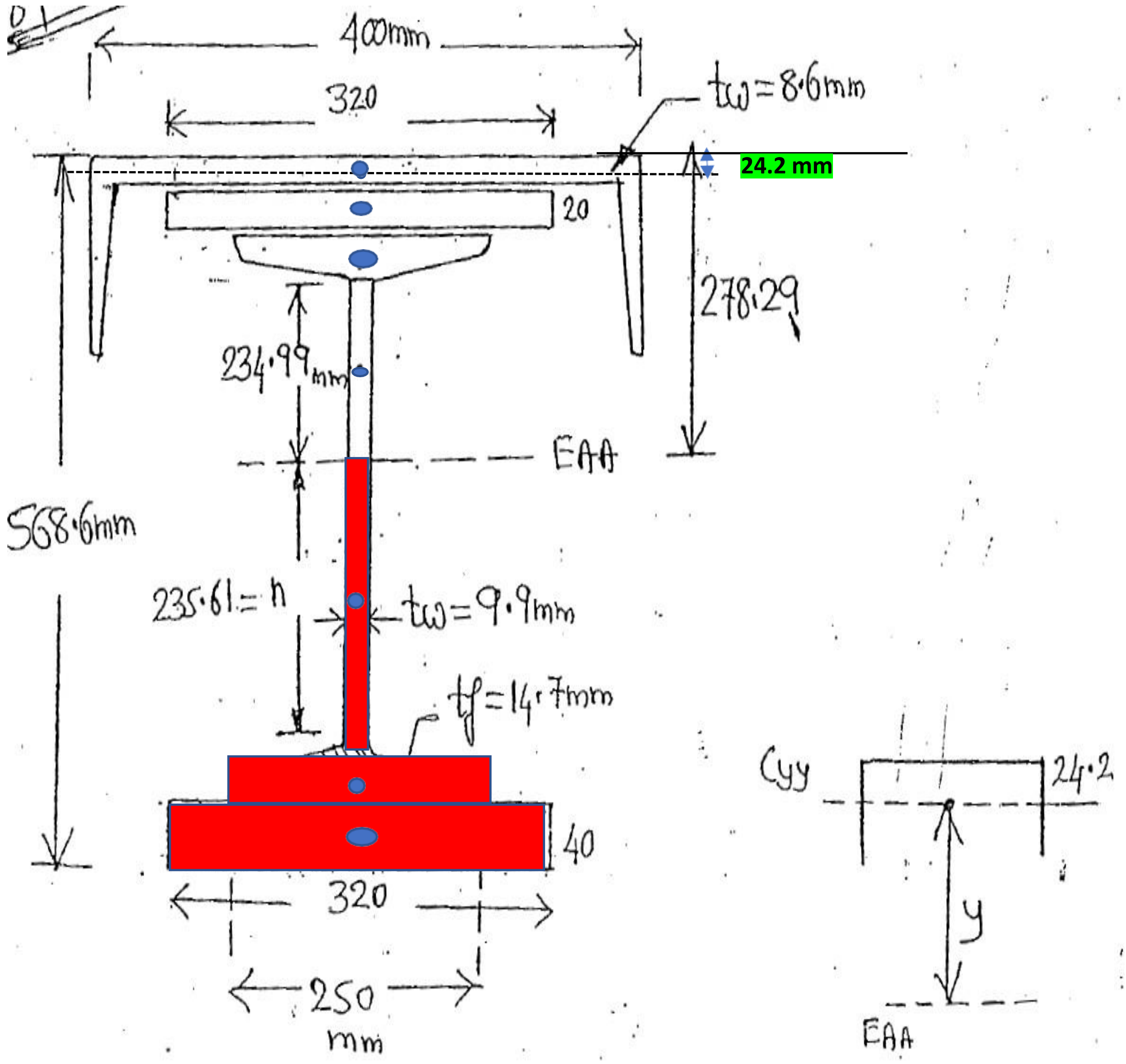


- **Plastic modulus** Zp – It is sum of areas of compression and tension zones multiplied by corresponding distance of the centroid of the compressive and tension area from the equal area axis



Calculation of Plastic modulus $Z_p = \sum a \times \bar{y}$

The distances $\bar{y}$ are measured from EAA axis to centroidal axis of the section



Shaded area (□)	Centroidal distance from EAA (□□)	Z_p (□□□) (□ × □□)	Unshaded area (□) □□□	Centroidal distance from EAA (□□)	Z_p (□□□) (□ × □□)
320 × 40	□□□. □□ + □□. □ □□ + □		□□□. □□ × □. □	□□□. □□ / □	
□□□ × □□. □	□□□. □□ + □□. □ / □		250 × □□. □□	□□□. □□ +14.7/2	
□. □ × □□□. □□	□□□. □□ / □		320 × □□	□□□. □□ + □□. □ + □□ / □	
		= ∑ □ × □□			

			Area of Channel section (From steel table) □□□□	□□□. □□ − □□. □	
Total	=∑ □ × □̄	=	+		= ∑ □ × □̄
Plastic Modulus Z_p =∑ □ × □̄	= □. □□ × □□□□□□				

- **Check for Moment of resistance**

Page 54, CL 8.2.2 IS 800 2007

8.2.2 *Laterally Unsupported Beams*

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, *see* **8.2.1**) in the following cases:

a) Bending is about the minor axis of the section,

b) Section is hollow (rectangular/ tubular) or solid bars, and

c) In case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by:

$$\boxed{M_d = \beta_b Z_p f_{bd}}$$

where

β_b = 1.0 for plastic and compact sections.

= Z_e/Z_p for semi-compact sections.

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given below [*see* Tables 13(a) and 13(b)]

α_{LT}, the imperfection parameter is given by:

$\boxed{\alpha_{LT} = 0.21 \text{ for rolled steel section}}$

α_{LT} = 0.49 for welded steel section

We need to calculate f_{bd} based on the values of $f_{cr,b}$ and imperfection factor α_{LT}

α_{LT}, the imperfection parameter is given by:

$\boxed{\alpha_{LT} = 0.21 \text{ for rolled steel section}}$

$\alpha_{LT} = 0.49$ for welded steel section

$$f_{cr,b} = \frac{1.1 \; \pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

E = 2×10^5 N/mm^2 , L_{LT} = 8000 mm (span of gantry girder), **[r_{yy} = 95.7 mm, t_f = 33.8 mm (Top flange mean thickness) (for Girder from Steel table**, **Table 12, Page 43**)]

h_f = centre to centre distance between the flanges = Overall depth of girder – ½ (Top and bottom mean flange thickness of girder)

= 568.6 - ½ × (33.8 + 51.5)

= 525.9 mm

$f_{cr,b}$ = 485.65 N/mm^2

Find f_{bd} , **Table 13 (a) Page 55**, IS 800- 2007, for $f_{cr,b}$ = 485.65 N/mm^2, α_{LT} = 0.21, f_y = 250 N/mm^2

Table 13(a) **Design Bending Compressive Stress Corresponding to Lateral Buckling, f_{bd}**, $\alpha_{LT} = 0.21$

(Clause 8.2.2)

$f_{cr,b}$	f_y																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
8 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
6 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
4 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
2 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
1 000	169.1	179.5	186	196.5	202.9	209.1	219.8	229.1	245.5	261.8	275.1	291.3	300.5	323.6	332.2	355.9	370.9	384.8	412.4
900	169.1	179.5	186	194.5	200.7	204.5	215.1	231.6	242.7	258.9	272	291.3	300.5	316.4	328.4	339.5	366.5	380.2	392.7
800	167.3	177.5	184	190.3	196.4	206.8	212.7	224	240	258.9	268.9	284.7	293.6	301.8	324.5	335.5	349.1	370.9	387.8
700	163.6	171.8	182	188.2	192	202.3	208	226.5	237.3	250.2	259.6	278.2	286.7	294.5	305.5	327.3	340.4	352.4	363.3
600	161.8	168	176	181.9	194.2	197.7	203.3	218.9	226.4	244.4	253.5	261.8	276.4	287.3	294	306.8	322.9	333.8	343.6
500	161.8	166.1	172	179.8	185.5	188.6	200.9	208.7	218.2	232.7	244.2	248.7	259.1	269.1	274.9	286.4	296.7	301.4	314.2
450	158.2	164.2	168	173.5	183.3	186.4	191.5	206.2	215.5	224	231.8	242.2	248.7	258.2	263.5	274.1	279.3	292.1	294.5
400	150.9	162.3	166	169.4	174.5	184.1	186.7	196	204.5	215.3	222.5	229.1	238.4	243.6	248.2	257.7	261.8	264.3	274.9

Through interpolation, the value of f_{bd} = 187.96 N/mm^2

$$\boxed{M_d = \beta_b Z_p f_{bd}}$$

Design bending strength or Moment of resistance, M_d = 1 × 9.05 × 10^6 × 187.96 = 1701.6 × 10^6N mm = 1701.6 kNm

1701.6 kNm > 839.4 kNm

It is safe.

- **Check for Shear resistance**

Page 59, CL 8.4, IS 800 2007

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \leq V_d$$

where

V_d = design strength

$= V_n / \gamma_{m0}$

where

γ_{m0} = partial safety factor against shear failure (*see* **5.4.1**).

The nominal shear strength of a cross-section, V_n, may be governed by plastic shear resistance (*see* **8.4.1**) or strength of the web as governed by shear buckling (*see* **8.4.2**).

8.4.1 The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p$$

where

$$\boxed{V_p = \frac{A_v f_{yw}}{\sqrt{3}}}$$

A_v = shear area, and

f_{yw} = yield strength of the web.

-

- **Check for Shear resistance**

Page 59, CL 8.4, IS 800 2007

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$$\boxed{V_p = \frac{A_v f_{yw}}{\sqrt{3}}}$$

A_v = shear area, and

f_{yw} = yield strength of the web.

Design shear strength $V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}}$

A_v is shear area = h × t_w = 568.6 × 9.9 = 5629 mm^2

$V_d = \frac{5629 \times 250}{\sqrt{3} \times 1.1}$ =738 kN>515.28 kN, it is safe.

- **Check for web crippling**

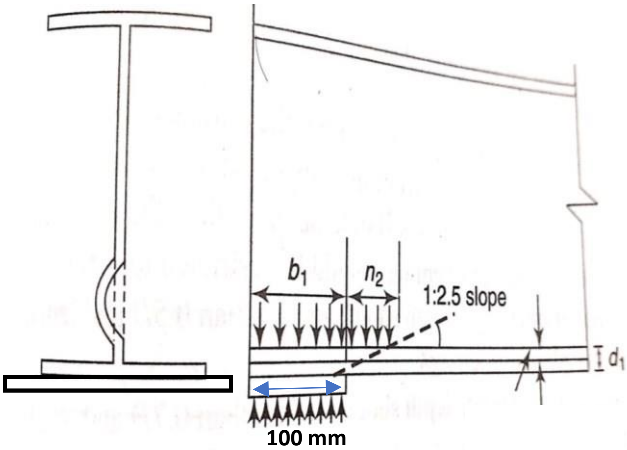
- Web crippling causes local crushing failure of web due to large bearing stresses under reactions at supports or concentrated loads.
- This occurs due to stress concentration because of the bottle neck condition at the junction between flanges and web.
- It is due to the large localized bearing stress caused by the transfer of compression from relatively wide flange to narrow and thin web.

Use CL 8.7.4, Page 67, IS 800-2007

8.7.4 *Bearing Stiffeners*

Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceeding the local capacity of the web at its connection to the flange, F_w, given by:

$$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0}$$



where

b_1 = stiff bearing length (*see* **8.7.1.3**),

n_2 = length obtained by dispersion through the flange to the web junction at a slope of 1 : 2.5 to the plane of the flange,

t_w = thickness of the web, and

f_{yw} = yield stress of the web.

Let us assume bearing length width, b_1 = 100 mm,

n_2 = 2.5 × (thickness of bottom plate + thickness of flange)

= 2.5 × (40 + 14.7) = 136.75 mm

F_w = (100 + 136.75) × 9.9 × 250/1.1 = 532.68kN > 515.28 kN (Shear Force)

- **Check for buckling of web**

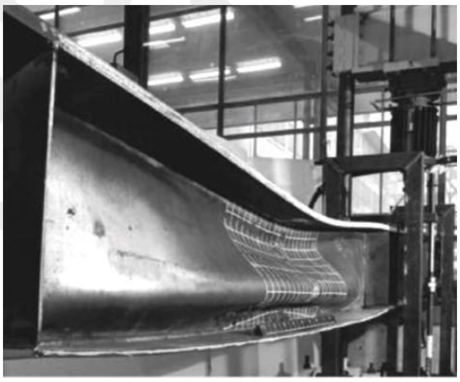
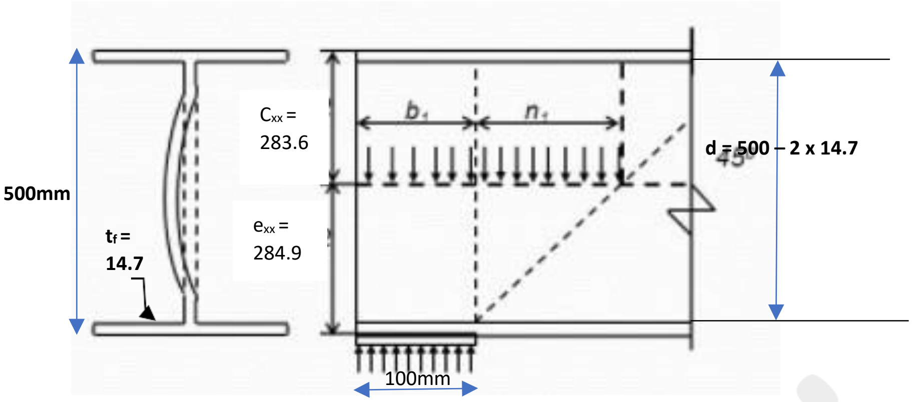


Figure 2. Vertical web buckling.

- The web of the beam is thin and can buckle under reactions and concentrated loads with the web behaving like a short column fixed at the flanges.
- The unsupported length between the fillet lines for I sections and the vertical distance between the flanges or flange angles in built up sections can buckle due to reactions or concentrated loads. This is called web buckling.



Buckling strength of web, F_{wb} = (b_1 + n_1) $t_w f_{cd}$

Breadth of bearing stiffener, b_1 = 100 mm,

Dispersion of 45° line at the mid depth of section n_1 = e_{xx} = 284.9 mm,

Thickness of web, t_w = 9.9 mm

To find design compressive stress f_{cd} , we need to calculate Slenderness ratio λ

Slenderness ratio, $\lambda = \frac{L_{eff}}{r_{min}}$

where L_{eff} is the effective length of the strut(compression) taken as, L_{eff} = 0.7 × d

where 'd' is the depth of the web portion (strut) between the flanges = 500 – 2 × 14.7 = 470.6 mm

$r_{min} = r_{yy}$ = 95.7 mm (girder)

$\lambda = \frac{0.7 \times 470.6}{95.7} = 3.44$

Since it is a built-up member it will come under buckling class "c" (IS 800 – 2007, Page 44, Table 10).

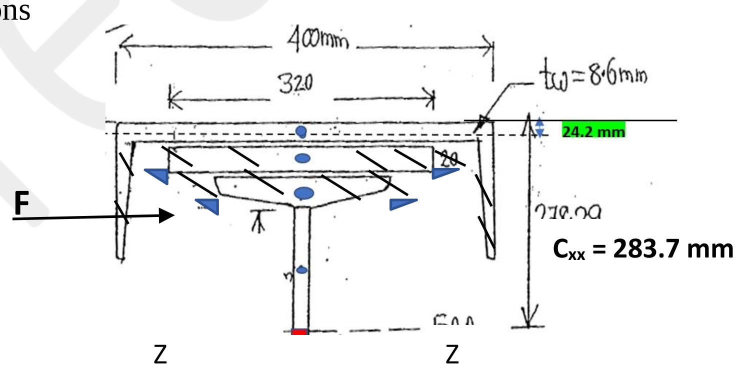
Since it is class "c", Use Table 9(c)

From Table 9 (c) Page 42 – IS 800 2007, for λ = 3.44 we get design Compressive Stress, f_{cd} = 227 N/mm^2

Hence Buckling strength of web, F_{wb} = (100 + 284.9) × 9. 9 ×227 = 865 kN > 515.28 kN (Shear Force)

- **Connections**

Using welded connections



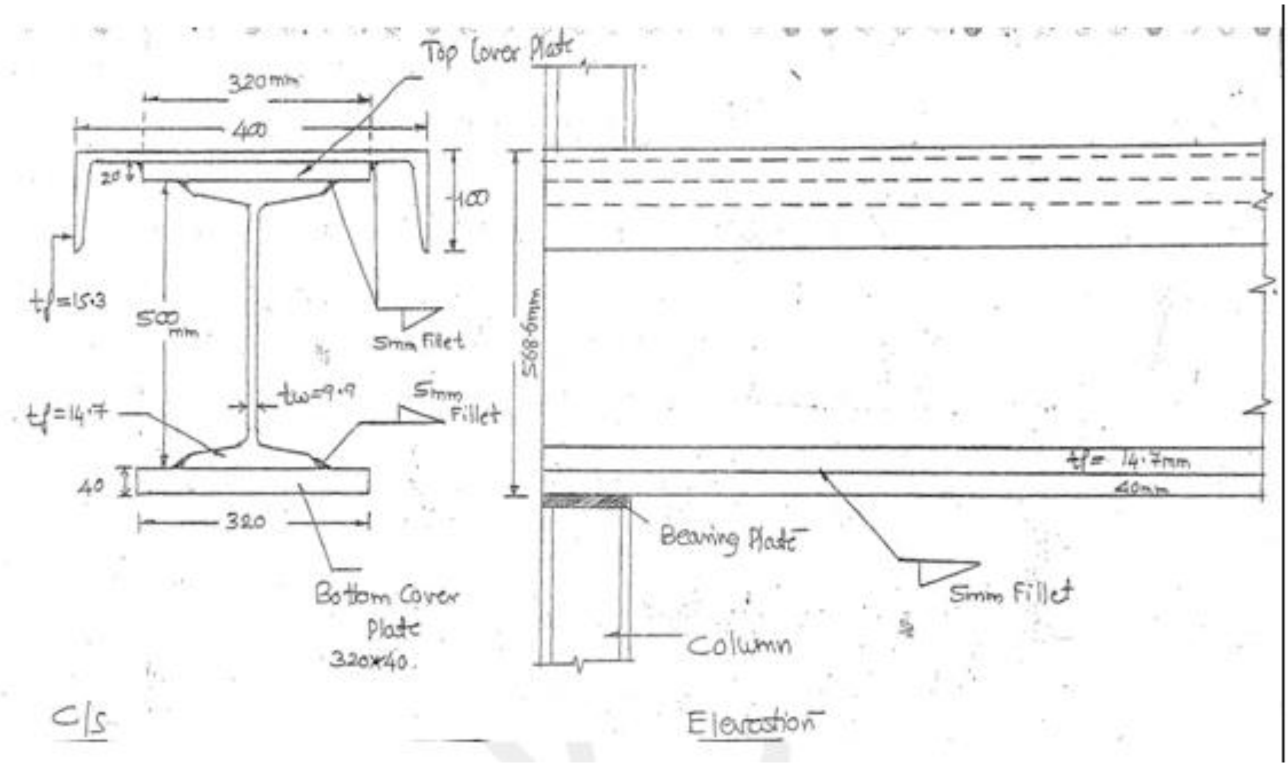
Force at the junction = F = $\frac{V\, a\bar{y}}{I_Z}$,

Where $a\bar{y}$ is the area of shaded portion multiplied by centroidal distances measured from individual section

$= 515.28 \times \frac{[6293 \times (283.7-24.2) + (320 \times 20 \times (283.7-8.6-10)]}{2301.9 \times 10^6} = 718.7$ kN/m

Strength of weld = 2 $\times \frac{[0.7 \times s \times 1 \text{ mm} \times 410]}{\sqrt{3} \times 1.25}$, s = 2.71 mm

Provide 5 mm size weld



Optional

Bracket connections for Gantry girder

No. of bolts = n = $\sqrt{\frac{6M}{l\,p\,R}}$

Where l = number of bolt lines = 4

Assume 20 mm diameter bolts

p is the pitch = 2.5 x diameter of bolt = 2.5 x 20 = 50 mm

R is the bolt value = 60.38 KN

Moment M=P x e

P= Max. SF in Gantry Girder = 515.28 kN

Assume , e= 200mm

M= (515.28 X10^3) x (200)=103056 X10^3 N-mm

$= n = \sqrt{\frac{6M}{l\,p\,R}} = \sqrt{\frac{6 \times 103056 \times 10^3}{4 \times 50 \times 515280}} = 8$

Use 8 number of bolts for bracket connections