CBCS SCHEME

USN

18CV72

Seventh Semester B.E. Degree Examination, Feb./Mar. 2022
Design of RCC and Steel Structures

Time: 3 hrs.   Max. Marks:100

Note: *1. Answer any ONE full question each from Module-1 and Module-2.*
2. Use of IS – 456, SP – 16, IS – 800, SP(6) and steel tables is permitted.
3. Missing data, if any, may be suitably assumed and same must be stated clearly.

Module - 1

1 Design slab type or slab beam type combined footing for two columns A and B spaced at 3.5m center to center. Cross section dimensions of column A is 400 × 400mm and carries an axial load of 1050kN. Cross section dimensions of column B is 500 × 500mm and carries an axial load of 1250kN. Safe bearing capacity of the foundation soil is 240 kN/m^2. The width of the combined footing is restricted to 2.00m. Use M-25 grade concrete and Fe-415 grade steel. Draw neat sketch of reinforcement details. **(50 Marks)**

OR

2 Design A cantilever retaining wall to retain soil embankment for a height of 3.5m above the average ground level. The back fill is horizontal at the top. The unit weight of soil is 16kN/m^3 and safe bearing capacity of the formation soil is 150 kN/m^2. The angle of repose of the soil is 30° and the coefficient of friction between concrete surface and soil may be taken as 0.55. Use M – 20 grade concrete and Fe – 415 steel. Draw a neat sketch of the designed reinforcement details. **(50 Marks)**

Module – 2

3 Design a welded gantry girder to be used in an industrial building for carrying a manually operated over head crane for the data as listed below :
Crane capacity = 200kN
Self weight of crab consisting trolley, motor, hooks, etc. = 40kN
Self weight of crane girder excluding crab (trolley) = 200kN
Minimum hook approach = 1.20m
Wheel base of crab (trolley) = 3.50m
Centre to centre distance between gantry rails = 16m(span of crane girder)
Centre to centre distance between columns = 8.00 (span for gantry girder)
Self weight of rail section = 300N/m
Diameter of crane wheel = 150mm
The steel used is Fe – 410 grade. Draw a neat sketch of the designed details. **(50 Marks)**

4. Design a bolted steel Howe truss having an effective span of 12.00m. The geometry of the truss is as shown in Fig.Q4. The forces induced in the members due to dead load, live load and wind load is tabulated in Table.Q4. Determine the design forces in the members due to various combination of loads as per IS – 800 provisions and hence design principal rafter, principal tie and main sling member with all the necessary safety checks including the reversal of stresses. Also design support joint 'A' by considering the size of supporting reinforced cement concrete column as 300 × 300mm, the design bearing pressure on the concrete is limited to 2N/mm^2 and the design bond stress between anchor bolt and concrete is limited to 1.2N/mm^2. Use M16 ordinary black bolts of grade 4.6 for designing member and connection with gusset plate and M25 bolt as anchor bolt at supports. List the design details.

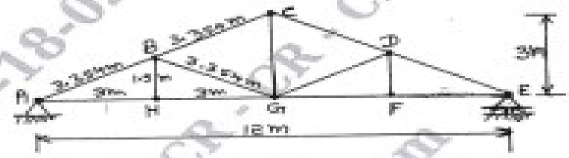


Fig.Q4

Members	Dead load kN	Live load kN	Wind load kN
Rafter AB, BC, CD and DE	–58.00	–52.52	+95.60
Tie member AH, HG, GF and FE	+52.00	+47.00	–76.00
Main sling BG, DG	+20.30	+18.40	–63.00

Table Q.4

Note :
i) (–) indicates compressive force
ii) (+) indicates tensile force
iii) Net support reaction ↑ = 45kN (at 'A')
iv) Net up-lift support reaction ↓ = 55kN (At 'A').

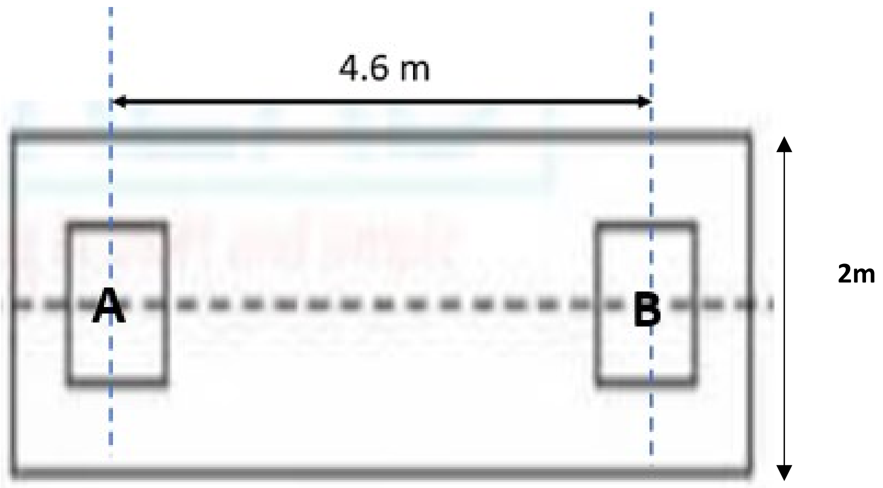
(50 Marks)

* * * * *

SOLUTIONS

1. Design a combined footing for 2 columns A and B of size 350 × 350 mm and 400 × 400 mm. The load carried by Column A is 700kN and Column B is 1200 kN respectively. The Safe bearing capacity of the soil is 130 kN/m^2. Use M20 concrete and Fe 415 steel. The center to center spacing between the columns is 4.6 m and width of footing is restricted to 2 m.

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,

Δ P = (700 + 1200) x 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

$$Area\ of\ the\ footing, A_{req} = \frac{P_1 + P_2 + \Delta P}{q_a} = 16.8 \quad m^2$$

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

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

- **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 ⇒ $P_{u1} + P_{u2} = 2850$ kN

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

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

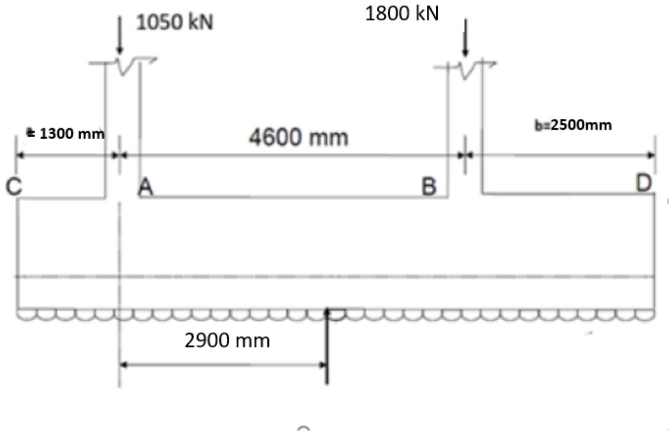
$$= \frac{1800 \times 4.6}{1050+1800} = 2.9 \text{ m}$$

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

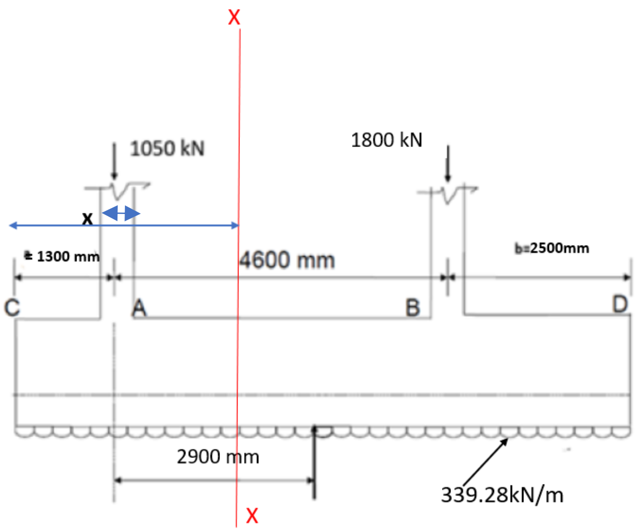


- **Uniformly distributed load acting in upward direction (soil pressure)**

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**



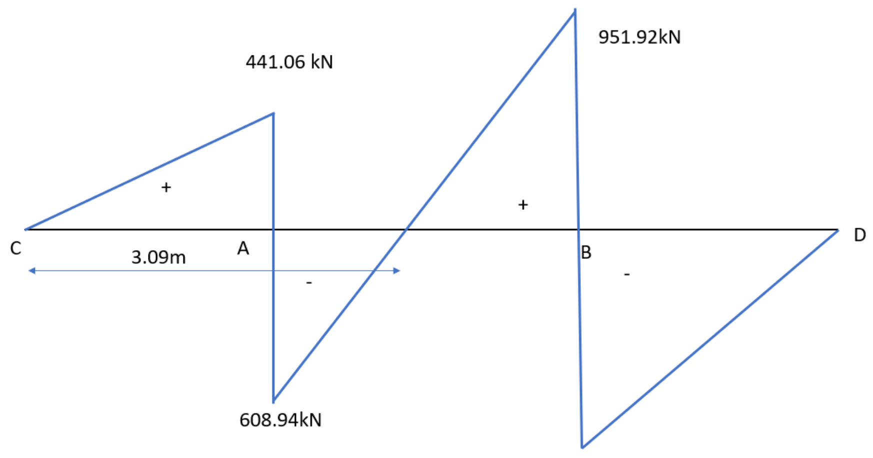
- Shear force at A, just before 1050 kN, left of section XX, V_{AC}= +339.28 x 1.3 = +441.06 kN
- Shear force at A, just after 1050 kN, left of section XX, V_{AB} = -1050+ 339.28 x 1.3 = -608.94 kN
- Shear force at B just after 1800kN, right of section XX , V_{BA} = +1800 – 339.28 x 2.5= +951.92 kN
-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

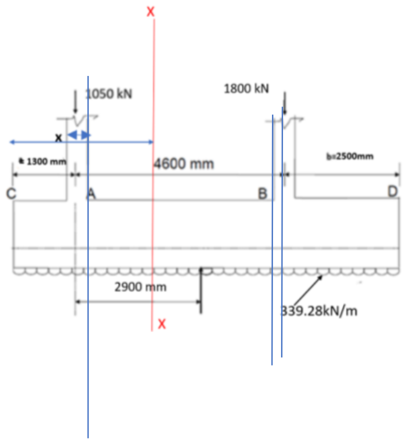
X = 3.09 m from C



- 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**

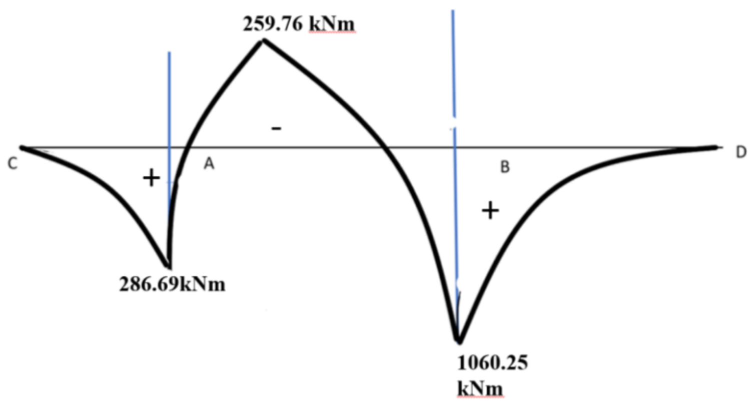


- BM at A, just before 1050kN, left of section XX, M_{AC}= 339.28 x 1.3 x1.3 /2 =+ 286.69 kNm
- BM at just at the inner face of Column A(1050kN), left of section XX,

  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

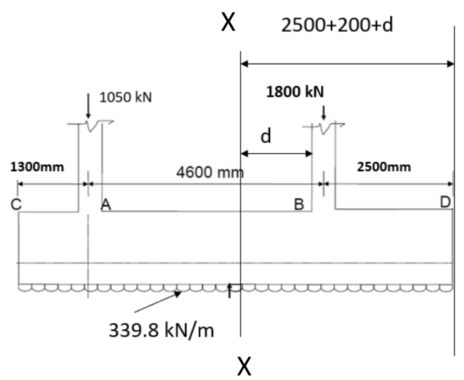


<u>Bending moment diagram</u>

- **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 (**<u>inner)</u>**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

$V_{uc} = 0.48 \times 2000 \times d$ = (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**

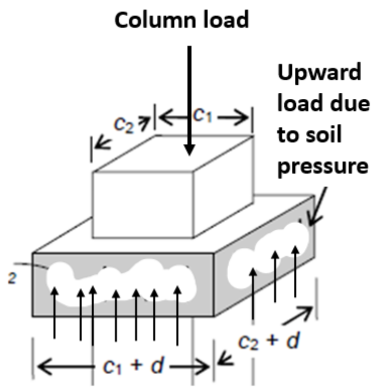
D = d + *75+ 20/2* = 680 + 75 + 20/2 = 765 mm

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

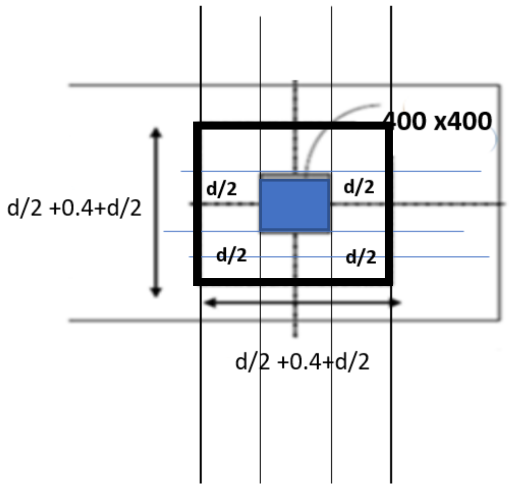
Two-way shear or punching shear ***(we need to consider the upward soil pressure not upward soil intensity) * Since it is acting on an area.***

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

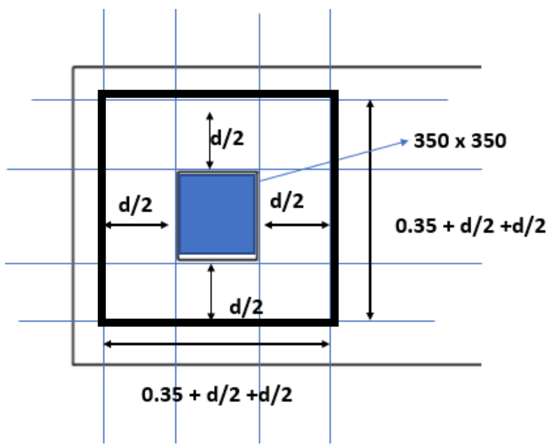


Punching shear or Two- way shear calculations for heavier Column B

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 V_{u2} = (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

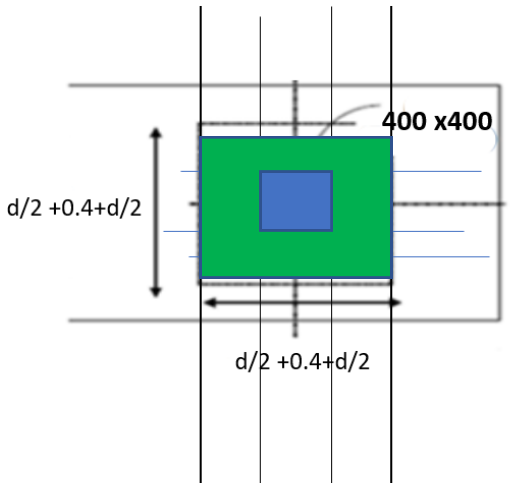
τ_c = $0.25\sqrt{f_{ck}}$ in limit state method of design, and $0.16\sqrt{f_{ck}}$ in working stress method of design.

For square columns, $ks = (0.5 + \beta c)$, 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)**



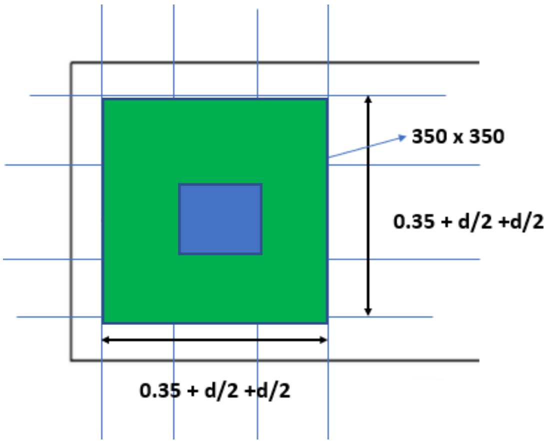
Perimeter of critical section (Green coloured area) = 4 X (400 + 680/2 + 680/2)

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.**

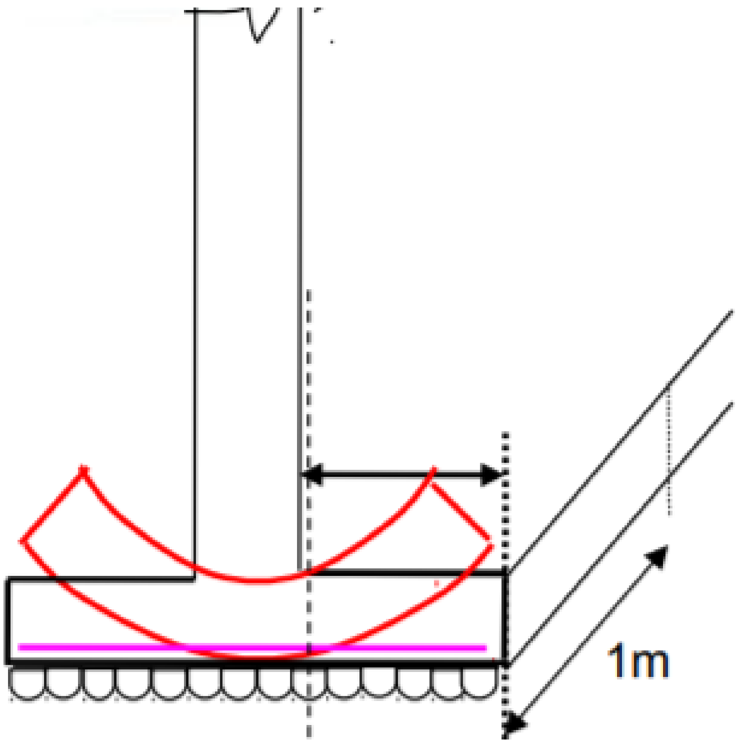
Hence safe against two way or punching shear, (if not provide shear reinforcement- stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

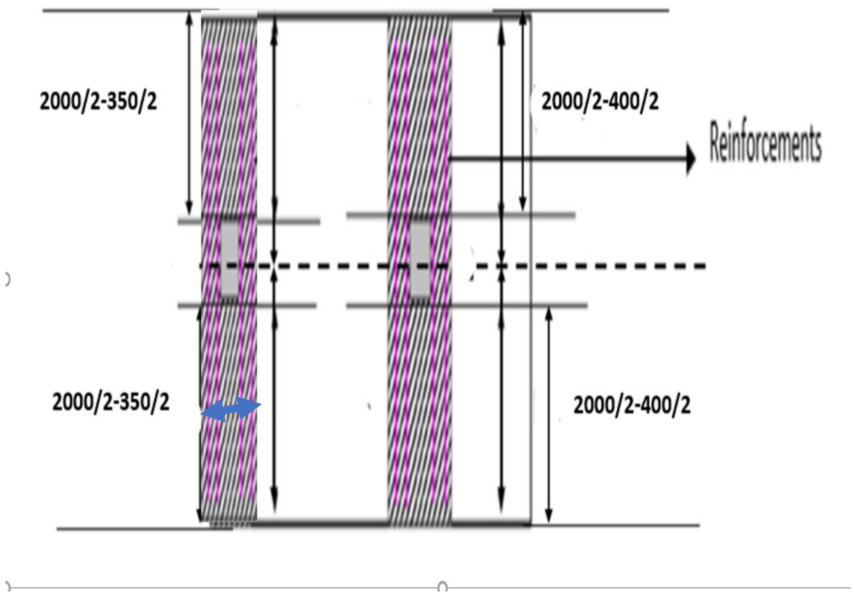
Maximum 'negative' moment: Mu = - 259.76 kNm	Maximum 'positive' moment: Mu = + 1060.25 kNm
$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$ Mu = 259.76 × 10^6 N mm	$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$ Mu = 1060.25 × 10^6 N mm

B = b = 2000 mm, 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 Ast provided < (*Ast*)*min, Hence provide (Ast)min* But we have assumed pt = 0.5 pt = 100 Ast, req/ (B × d) Choose 20 mm diameter bars, calculate no of bars = 1836 /(π/4 x 20^2) = 6 **Provide 6 # 20 mm diameter bars at top** • **Development length L_d = 47 x dia of bar** **= 47 x 20 = 940 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 No of 20 mm dia bars = 4648.12/(π/4 x 20^2) = 15 **Provide 15 # 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	• 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

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- Assume width of transverse beam, b =

  width of column + 2 x 0.75*d*

  b = 350 + 2 x 0.75 × 577 = 1215.5 mm

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

b = 1215.5 mm, d =577 mm

Mu = 178.6 x 10^6 N mm

Ast = 880.23 mm^2

- Width of transverse beam, b = width of column + 2 x 0.75d
- 400 + 2 ×0.75 ×577 = 1265.5mm

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

Mu = 384 x 10^6 N mm

b = 1265.5mm

d = 577 mm

Ast = 1956.2 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/ (π/4 x 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

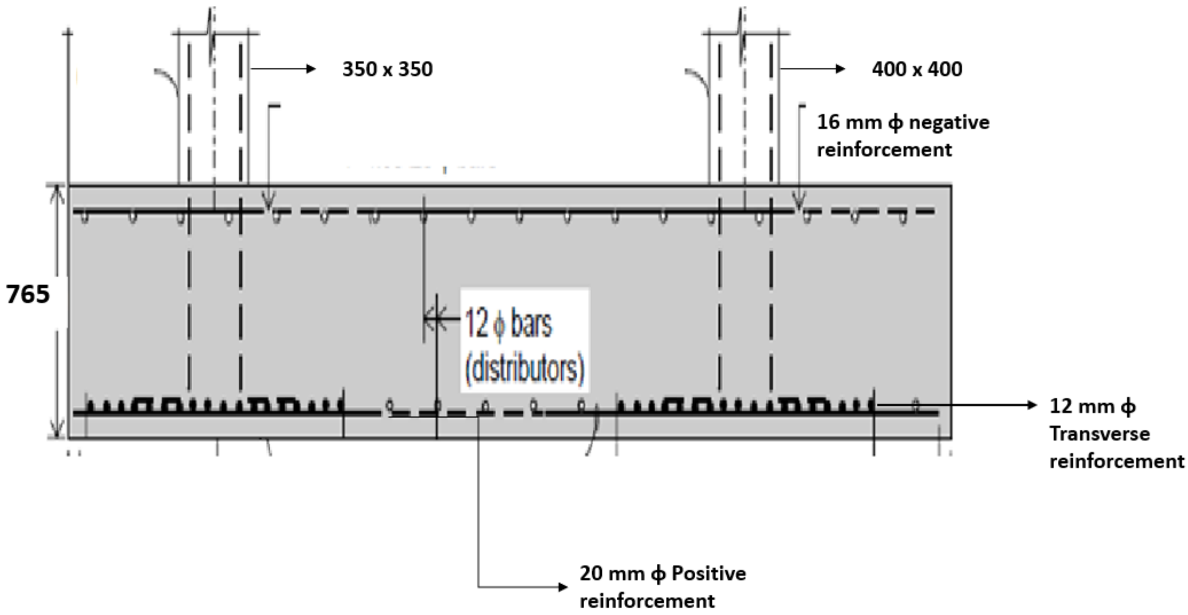
=1956.2 /(π/4 x 12^2) = 17.29 = 18

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. 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 = $0.45fck\sqrt{\frac{A1}{A2}}$ $A1 = 2000^2$ (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.*	• Limiting bearing stress at Permissible bearing stress = $0.45fck\sqrt{\frac{A1}{A2}}$ $[A1 = 2000^2 , A2 = 400^2 \text{ mm}^2]$ $\sqrt{\frac{A1}{A2}} = 5 > 2, \sqrt{\frac{A1}{A2}} = 2$ $= 0.45 \times 20 \times 2 = 18$ MPa *Permissible bearing resistance* = 18×400^2 = 2880 kN *2880kN > 1800kN , Hence safe*

Reinforcement detailing



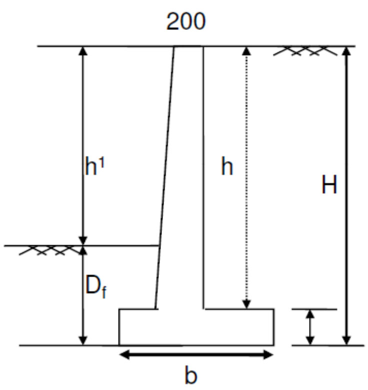
2. SOLUTIONS

Height of earth fill, h'= 4m, Safe bearing Capacity = 200 kN/m^2, Density of soil, γ = 18 kN/m^3, co-efficient of friction between concrete and soil, μ = 0.6, angle of repose φ = 30°

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

- **Depth of foundation, D_f**

Using Rankine's formula: find depth of foundation



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

$$k_a = \left[\frac{1-\sin 30^0}{1+\sin 30^0}\right]^2 = \frac{1}{3}$$

$$= \frac{200}{18} \times \frac{1}{3}$$

=1.23m say 1.2m, therefore H = 4 +1.2 = 5.2 m

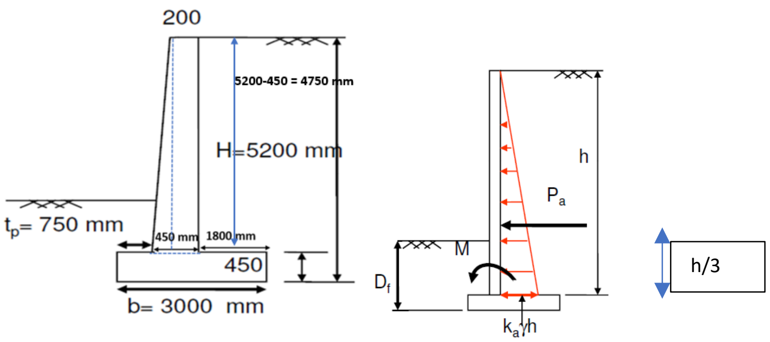
- **Proportioning of stem wall and base slab**

a. Thickness of base slab = **(1/10 to 1/14) H** = 1 /10 × 5.2 to 1 /14 × 5.2 = 0.52m to 0.43m, say 0.45 m or 450 mm
b. Width of base slab = b = **(0.5 to 0.6) H** = 0.5 × 5.2 to 0.6 ×5.2 = 2.6m to 3.12 m say 3m
c. Toe projection or width of toe slab = pt = (**1/3 to ¼) b** = 1 /3 × 3 to 1/4 × 3 = 1.0 m to 0.75 m say 0.75 m
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 ,Pa $= \frac{1}{2}(k_a \times \gamma \times h)\, h$

Pa = $\frac{1}{2} \times \frac{1}{3} \times 18 \times 4.75 \times 4.75 = 67.68$ kN

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

M = $67.68 \times \frac{4.75}{3} = 107.17$ kN-m

Mu = 1.5 × M = 160.6 kN-m

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

Check for effective depth

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

Put Mu,lim = 160.6 x 10^6, b = 1000mm , fck = 20N/mm^2

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

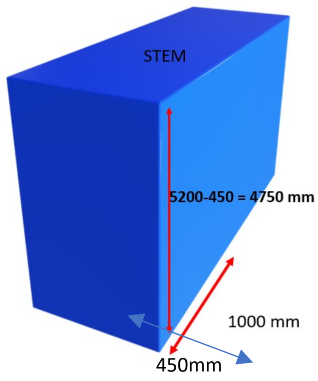
160.6 x 10^6 = 0.36 x 0.48 x (1 - 0.42 x 0.48) x 1000 x d^2 x 20

Find 'd' , d = 242 mm

Required, d = 242 mm

Assumed overall depth at base of the stem, D = 450 mm, effective, d = 450 - 50 = 400 mm >> 242 mm

Taking 1m length of stem wall,



b = 1000 mm, d = 450 - effective cover

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

= 40 + 12/2 = 46 ≈ 50 mm

d = 450 – 50 = 400 mm >> 242 mm , hence safe

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

d = 400 mm, b =1000 mm, Mu =160.6 x 10^6 Nmm, fy = 415 N/mm^2 , fck = 20 N/mm^2

Ast = 1184.6 mm^2

Ast, min = $0.0012 \times b \times D = 0.0012 \times 1000 \times 450 = 540\ mm^2$

Ast > Ast,min, hence Ok.

Provide 12 mm φ bars as main steel

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{1184.6}$ = 96 mm ≈ 100 mm or 95 mm (Your wish!!)

Main steel #12 mm φ @ 100 mm c/c < 300 mm or 3 times effective depth "d" (Check!!!) IS 456 2000

Distribution steel

= 0.12% Gross Area = 0.12 × 450 × 1000/100 = 540 mm^2

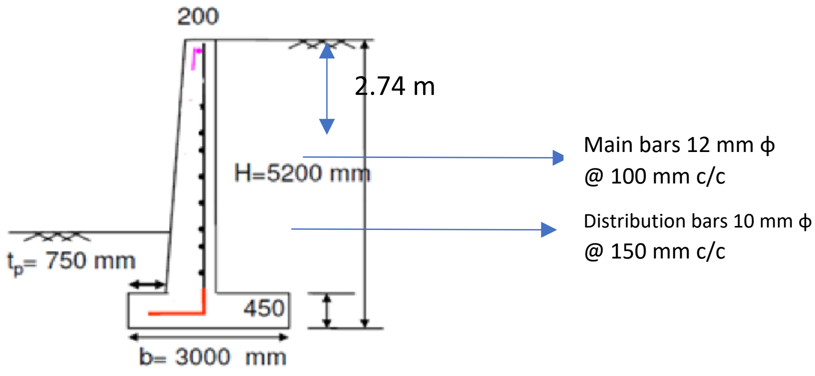
Use 10 mm φ bars , spacing required

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{540}$ = 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

$L_d = 47\ \Phi_{bar} = 47 \times 12 = 564$ **mm =0.564 m**



Curtailment of bars

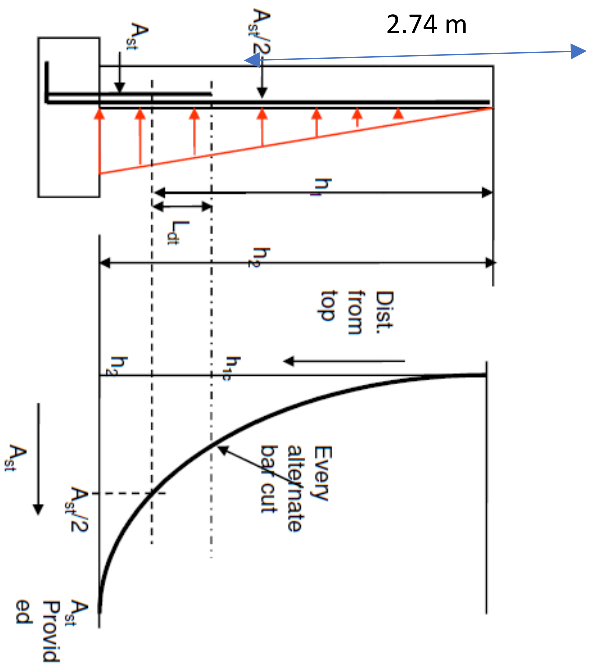
Curtail 50% steel from top, Ast = $\frac{50}{100} \times 1184.6\ = 592.3\ mm^2$

$\left(\frac{h_1}{h}\right)^2 = \frac{1}{2}, \left(\frac{h_1}{4.75}\right)^2 = \frac{1}{2}, \ \frac{h_1{}^2}{4.75^2} = \frac{1}{2}, h_1 \ \ = 3.36\ m$, is the curtailment length

Actual point of cut off or cutting position = 3.36 - L_d =3.36 - 0.564 = 2.74m from top.

Spacing required, s = $\frac{1000\ \times\frac{\pi}{4}\times 12^2}{592.3}$ = 190.9 mm ≈ 190 mm

Spacing of bars 12 mm φ @ 190 mm c/c < 300 mm and 3 x effective depth . Hence it is ok

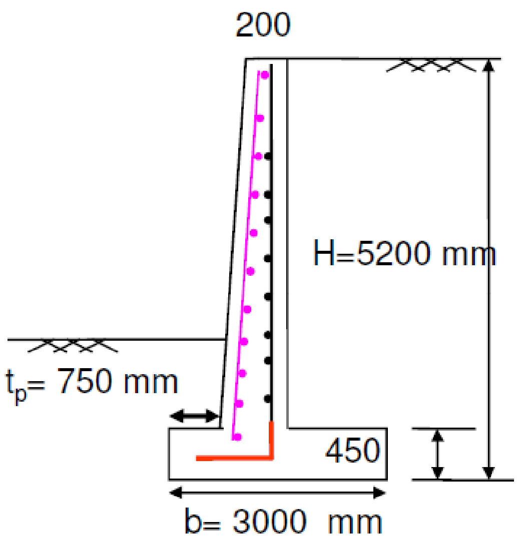


Secondary steel for stem at front (Temperature steel)

0.12% Gross Area = 0.12 × 450 × 1000/100 = 540 mm²

This can be copied from distribution steel calculations

#10 @ 150 mm c/c< 450 mm and 5d ok

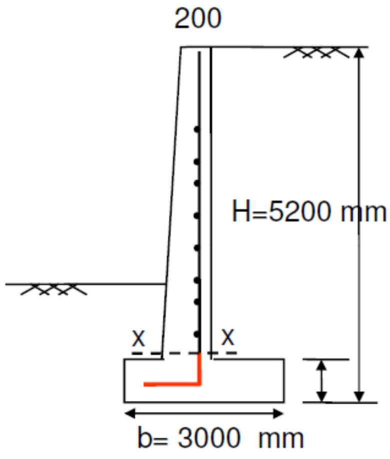


Check for shear

Max. Shear Force at Junction XX= P_a = 67.68 kN (Lateral earth pressure)

Ultimate Shear Force = Vu =1.5 x 67.68 = 101.52 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 pt $= \frac{100\,A_{st}}{b \times d} = \frac{100 \times 1184.6}{1000 \times 400} = 0.29\ \%$

Use IS:456-2000, Page 73, Table 19, pt = 0.29 %, M 20

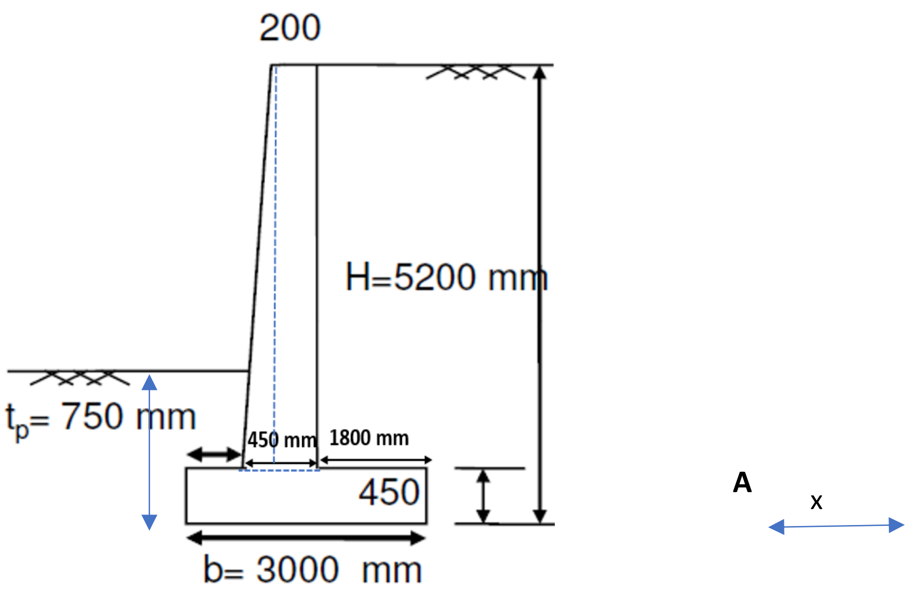
τc = 0.38 N/mm^2

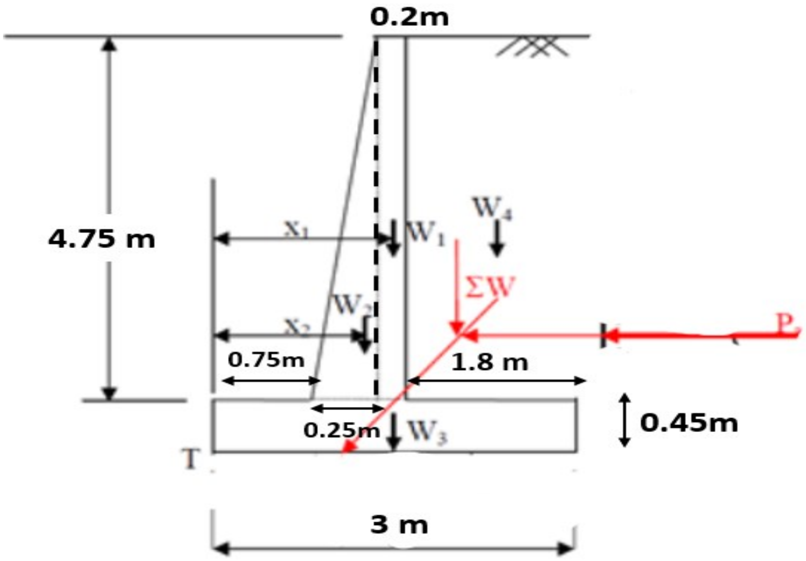
Compare τ_v and τ_c , 0.25 < 0.38

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

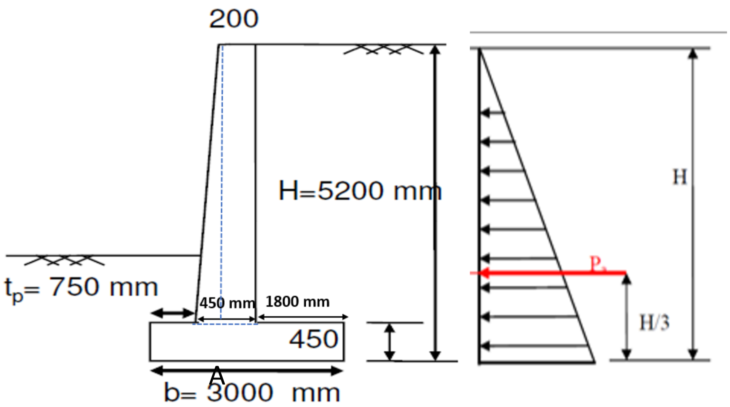
Stability analysis – 1. To find factor of safety against overturning

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





Load	Magnitude, kN	Distance from A, m	Bending moment about A kN-m
Stem W1	0.2 x 4.75x1x25 = 23.75	(0.75+0.25+0.2/2) =1.1	26.13
Stem W2	½ x 0.25x4.75x1x25 = 14.84	0.75 + 2/3 x 0.25 = 0.916	13.59
Base slab W3	3.0x0.45x1x25 = 33.75	3/2	50.63
Back fill, W4	1.8x4.75x1x18 = 153.9	0.75+0.45+1.8/2 =2.1	323.20
Total	**ΣW= 226.24 kN**		**ΣM_R = 413.55 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 = -140.60**

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 = 413.55 kNm, M_O = 140.05 kNm

(F.S) overturning = $\Sigma M_R / M_O$ = 2.94 >1.55 Hence it is safe

2. **Check for Sliding**:

ΣW= 226.24kN

P_H =81.12 kN

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

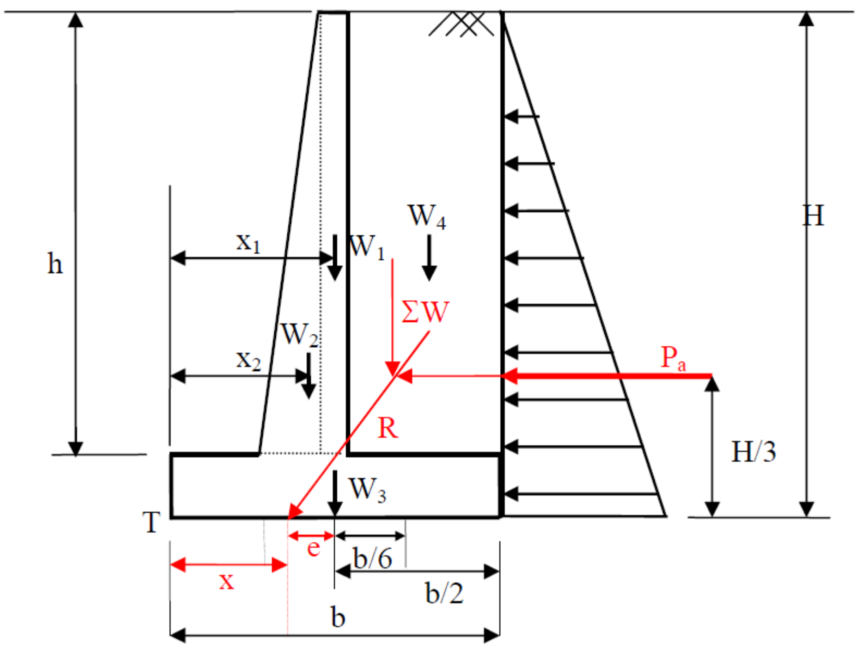
$$\frac{\mu \sum W}{P_H} = \frac{0.6 \times 226.24}{81.12} = 1.67$$

(F.S) sliding = 1.67 ≥1.55 Hence it is safe

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 = 413.55 -140.05 =273.5 kNm



$$x = \frac{273.5}{226.24} = 1.2 \text{ m} \quad , b = 3\text{m}$$

- **Eccentricity e = b/2 – x** = 3/2 – 1.2 = 1.5 – 1.2 **= 0.3m < b/6** (0.3 < 0.5)(Eccentricity of force should not exceed one sixth of base)

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

Pressure below the base slab , SBC =200 kN/m²

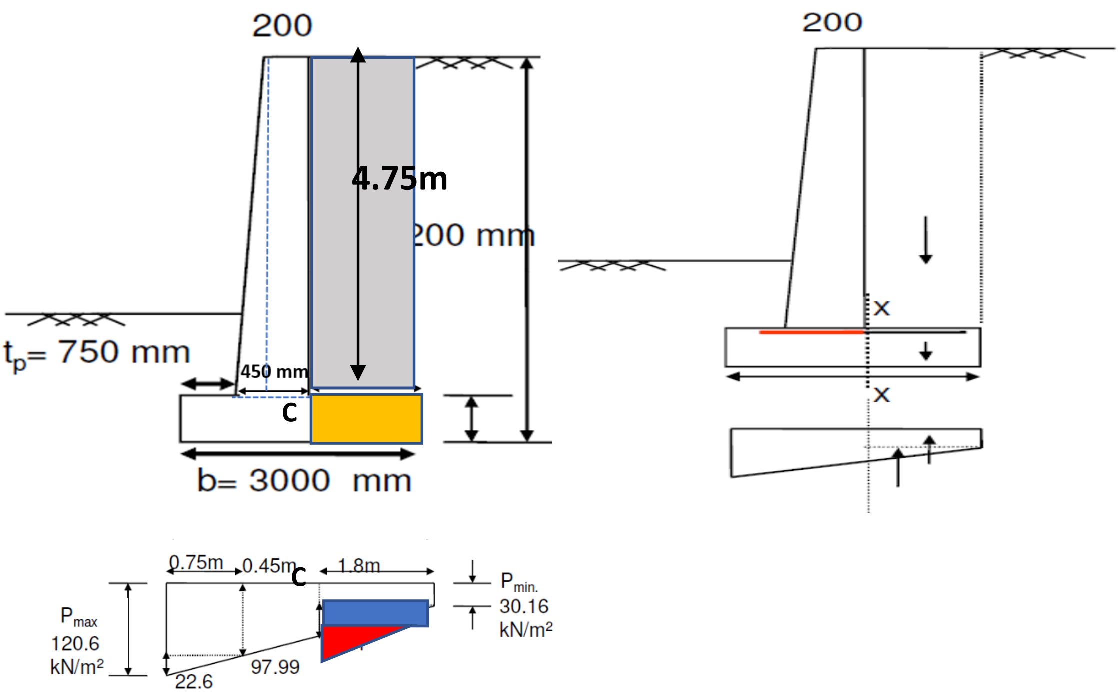
$$\text{Max. pressure} = P_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right]$$

120.66 kN/m² < SBC, safe

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

30.16 kN/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.



Pressure below the Retaining Wall

Using similar triangles,

$\frac{120.6-30.16}{3} = \frac{30.16+y}{1.8}$, y =24.1 kN/m^2 (Sample calculations)

Design of Heel Slab

Calculations of Moment about heel slab C

Load	Magnitude, kN	Distance from C, m	BM, M_C, kN-m
Wt of Backfill or Earth fill	1.8 x 4.75 x 1x18 =153.9	1.8/2 = 0.9	=+138.51
Heel slab	0.45x1.8x25 = 20.25	1.8/2 =0.9	=20.25 x 0.9 =+
Pressure distribution,(below heel slab) rectangle	- 30.16 x 1.8 = -	1.8/2 = 0.9	-
Pressure distribution, Triangle	$\frac{1}{2}$ x -24.1 x1.8= -21.69	$\frac{1}{3}$x1.8 = 0.6	-13.01
Total Load at junction C	98.17	**Total BM at Junction C**	**ΣM_C = 94.87 kNm**

ΣM_C = 94.87

Mu = 1.5 x 94.87 =142.3 kNm = 142.3 x 10^6 N mm

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

b = 1000 mm, d = 400 mm, f_{ck} =20 N/mm^2 f_y = 415 N/mm^2

Ast = 1041.5 mm^2

Use 16 mm φ bars (it is base slab) (You can choose 12 mm also)

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{1041.5}$ = 193 mm ≈ 190 mm

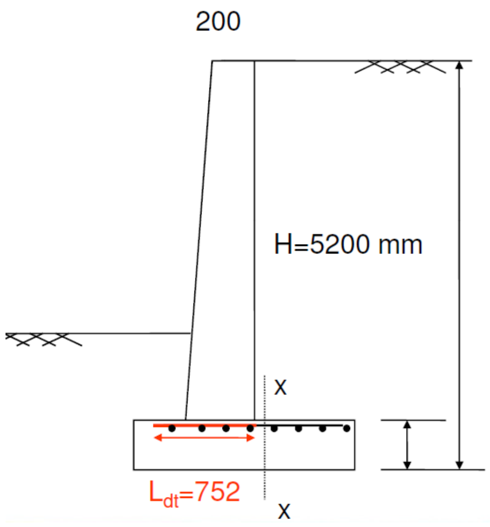
Main steel #16 mm Φ @ 190mm c/c < 300 mm and 3d ok. Hence it is safe.

Development length

L_d = 47 φ_{bar} = 47 x 16 = 752 mm

Distribution steel

Same, #10 dia @ 140 < 450 mm and 5d ok



Check for shear at junction (Tension)

The critical section for shear in the heel slab should be taken at the face of the support and not *d* away from it, because there is no compression introduced by the support reaction, and the probable inclined crack may extend ahead of the rear face of the stem

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

Maximum shear =V=98.17 kN, V_U, max= 98.17 x 1.5 =147.255 kN

$\tau_v = \frac{V_U}{b \times d} = \frac{147.255\times 10^3}{1000 \times 400} = 0.368$ N/mm^2

$\tau_v =$ 0.368N/mm^2

pt $= \frac{100\times 1041.5}{1000 \times 400} = 0.260$ % , Find τ_c

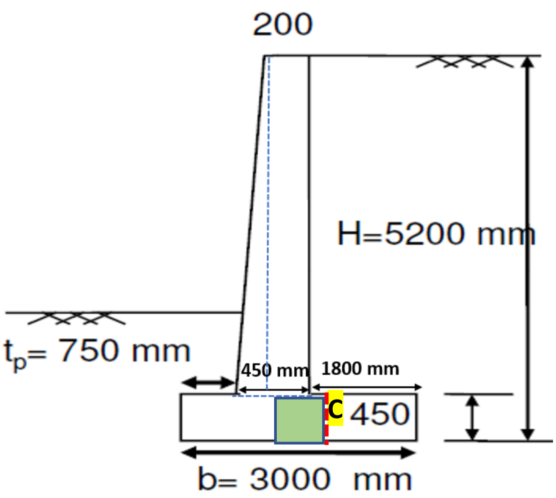
Use IS:456-2000, Page 73, Table 19, pt =0.26 %

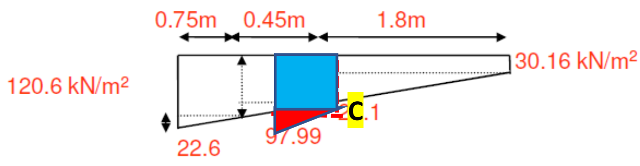
τ_c = 0.365 N/mm^2

Here τ_v and τ_c are almost close. There is no providing shear reinforcement. May be Ok.

Design of toe slab

To find the maximum bending moment





Pressure below the Retaining Wall

Load	**Magnitude, kN**	**Distance from C, m**	**BM, M_C, kN-m**
Self wt of Toe slab	0.75 x 0.45 x 25 = 8.44	0.75/2	+3.164
Pressure distribution, rectangle	-97.99 x 0.75 = -73.49	0.75/2	-27.56
Pressure distribution, Triangle	½ x -22.6 x 0.75 = -8.474	2/3 x 0.75=	-4.23
		Total BM at Junction C	**ΣM_C = -28.63**

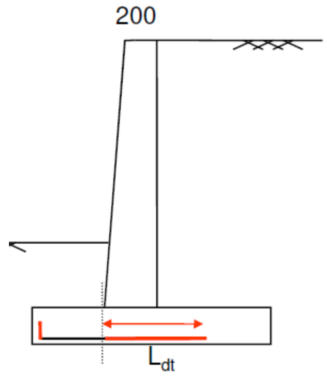
Mu = 1.5 x 28.63 =43 kN-m,

Ast= 302.48 mm^2

Provide #10mm @ 140mm < 300 mm and 3d ok (If you have time you can do calculation for spacing otherwise you proceed like this!)

Development length:

L_d = 47 ϕ bar = 47 x 10 = 470 mm



Check for shear: at “d” from junction of the toe slab (at XX as wall is in compression), d = 400 mm

Net shear force at the section XX

Net shear force for toe slab V= -(120.6 + 110.04) /2 x 0.35 + 0.45 x 0.35 x 25 = -36.42kN

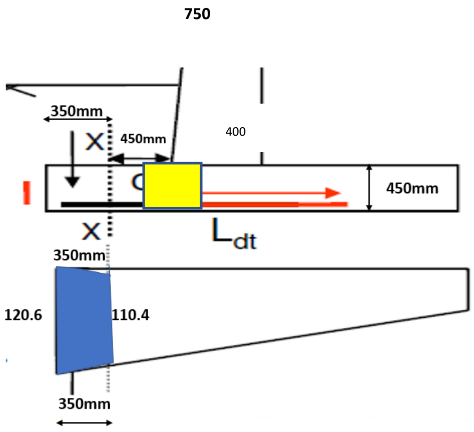
$V_{U,max}$ = 36.42 x 1.5 =54.63 kN

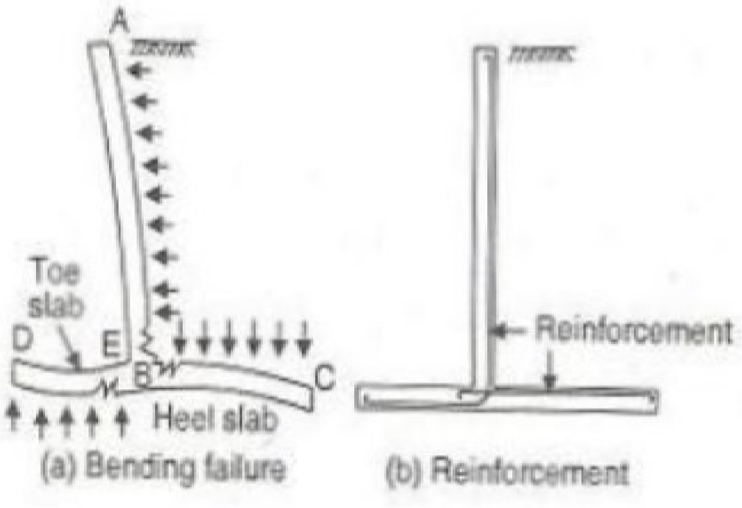
ζv = (54.63x1000) / (1000 x 400) = 0.13 N/mm^2

pt = (100 x 302.48) / (1000 x 400) = 0.075

From IS:456-2000, Page 73, pt≤0.15%, ζc = 0.28 N/mm^2

$\zeta v < \zeta c$, Hence safe in shear.





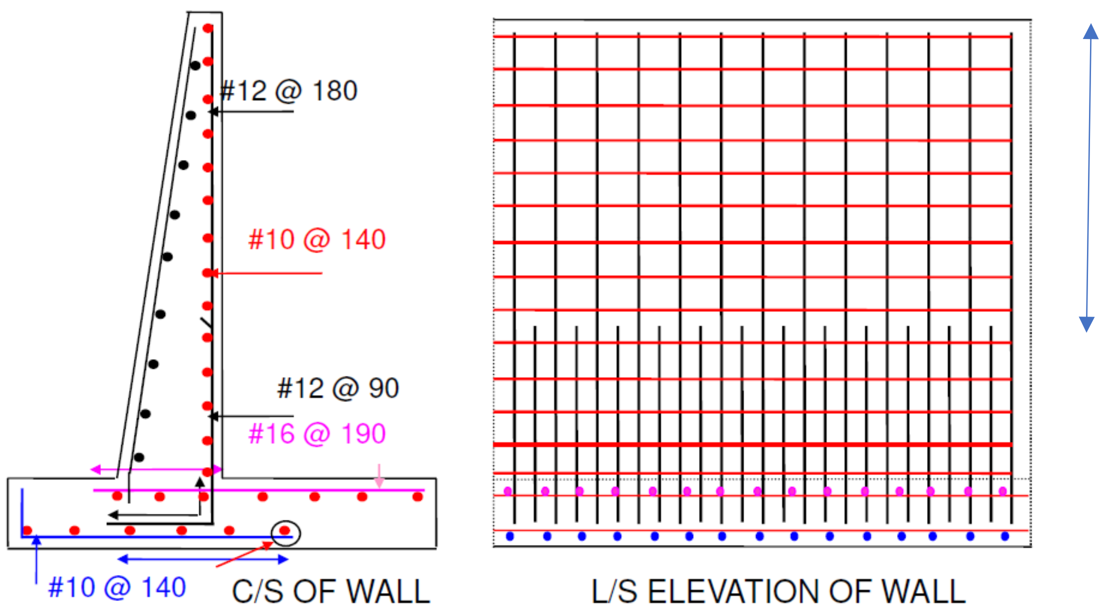
Construction joint

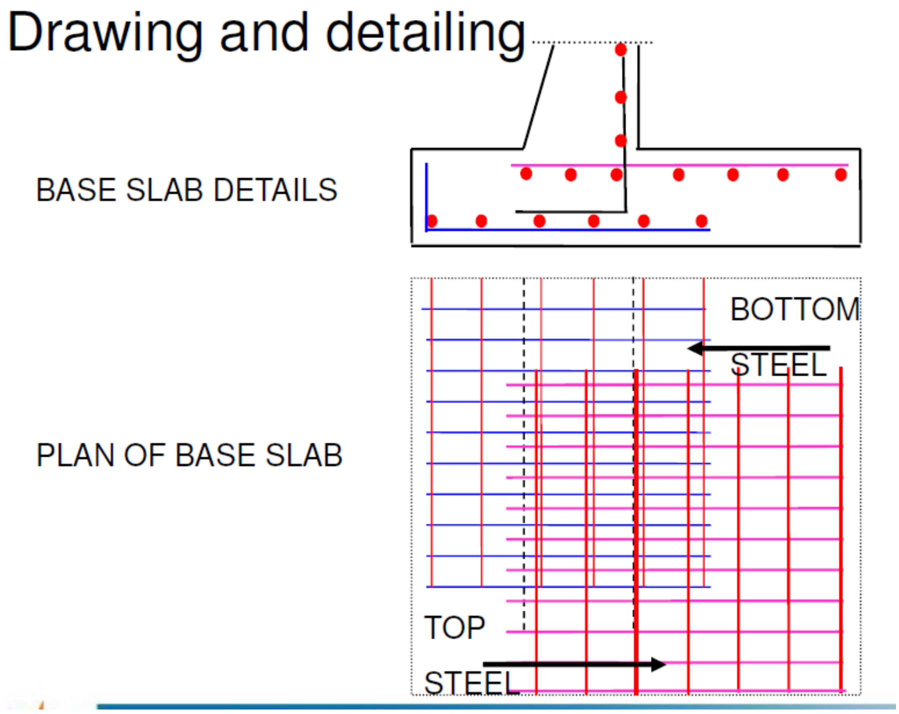
A key 200 mm wide x 50 mm deep with nominal steel #10 @ 250, 600 mm length in two rows.

Drainage

100 mm dia. pipes as weep holes at 3m c/c at bottom. Also provide 200 mm gravel blanket at the back of the stem for back drain.

Drawing and detailing





3. SOLUTIONS

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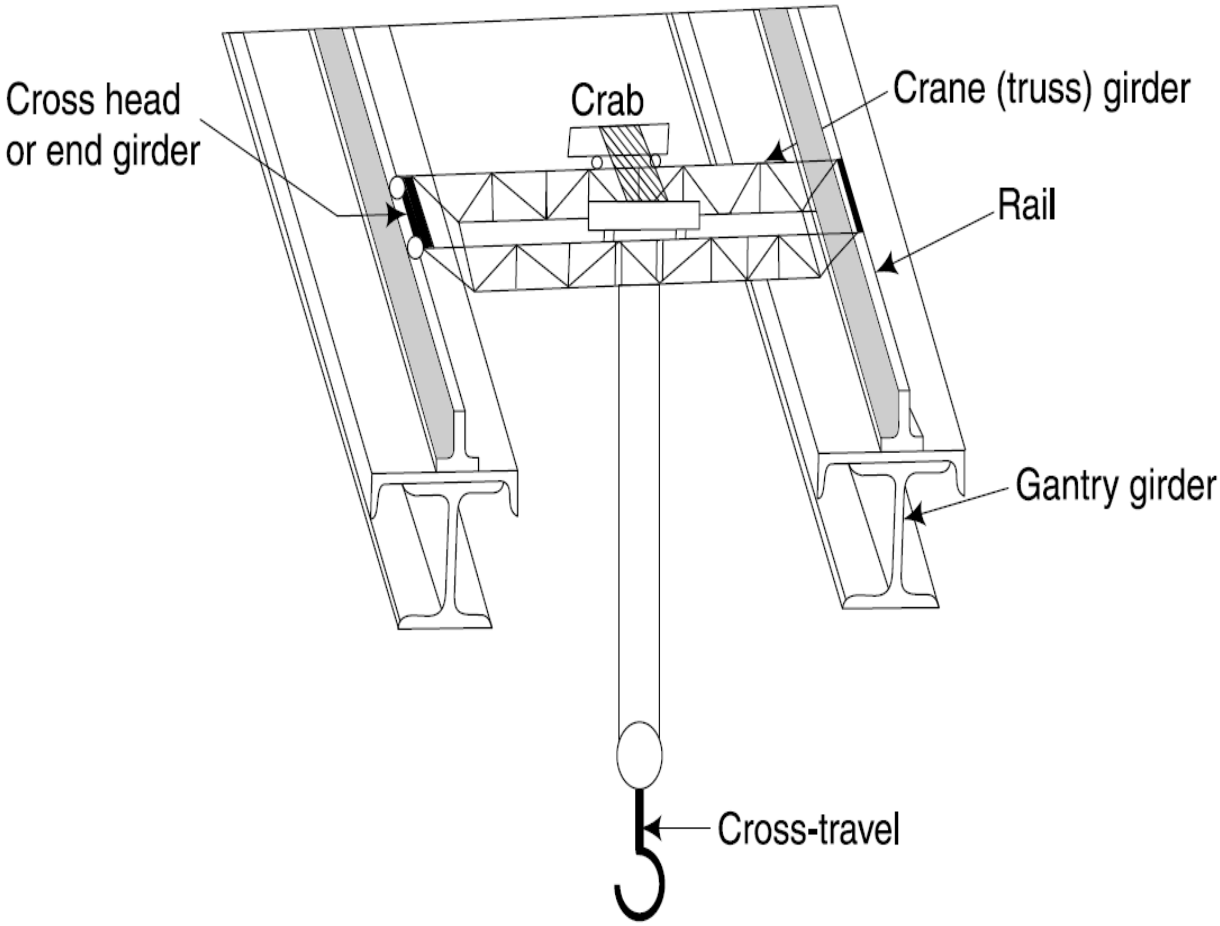
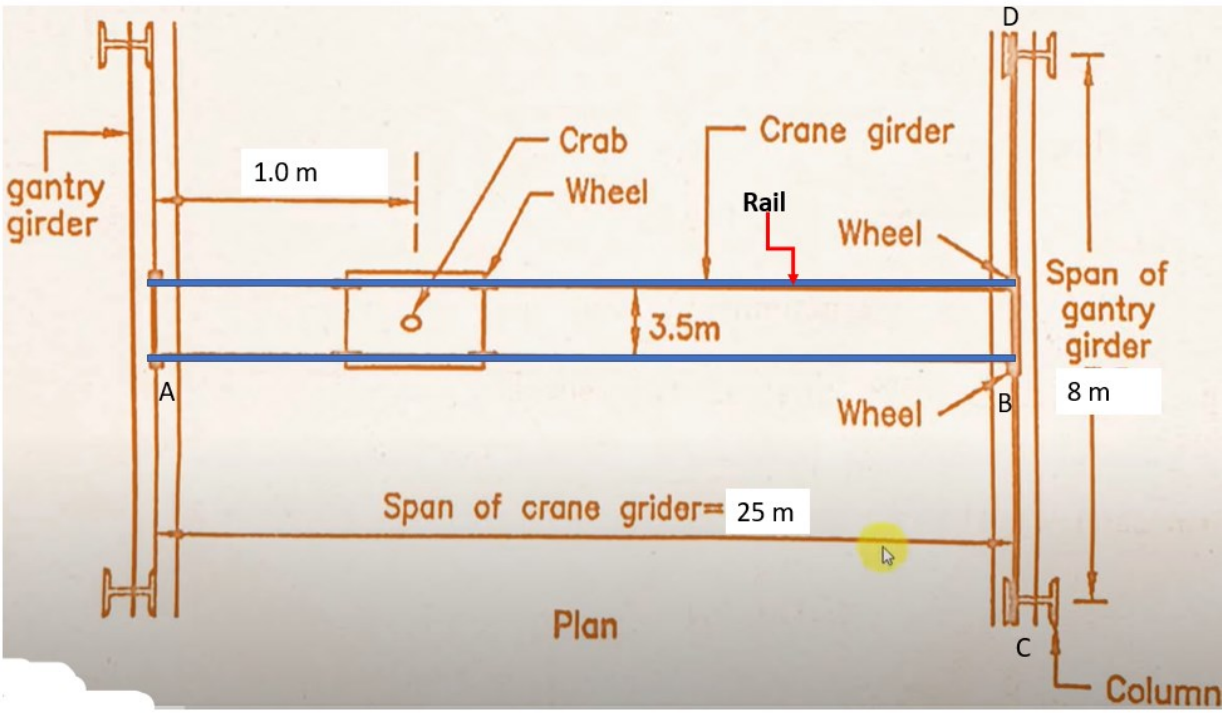
Solution For Fe 410 grade of steel: $f_u = 410$ MPa, $f_y = f_{yw} = f_{yf} = 250$ MPa

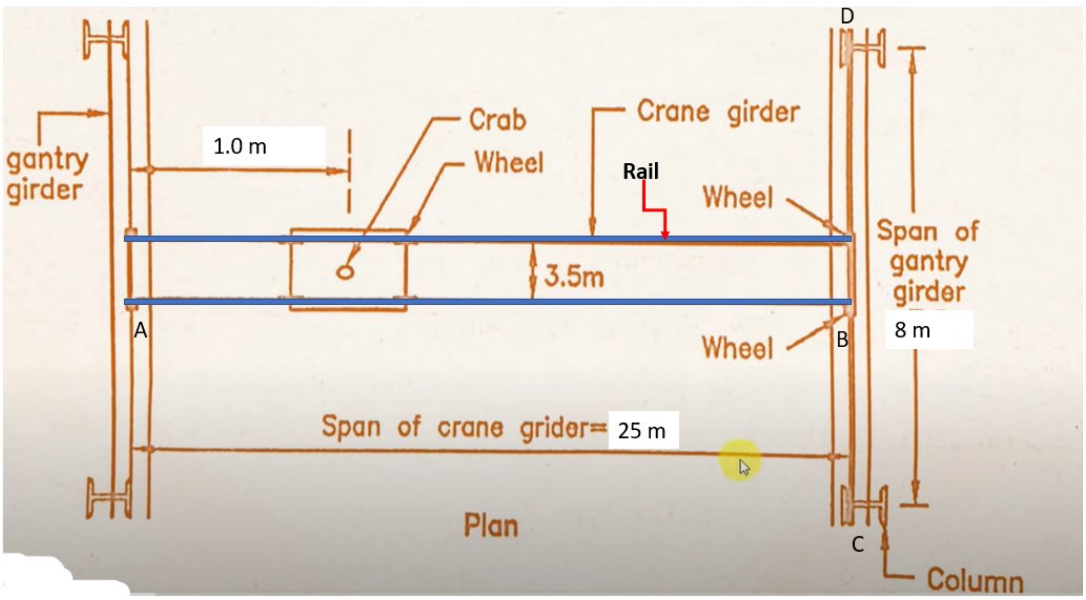


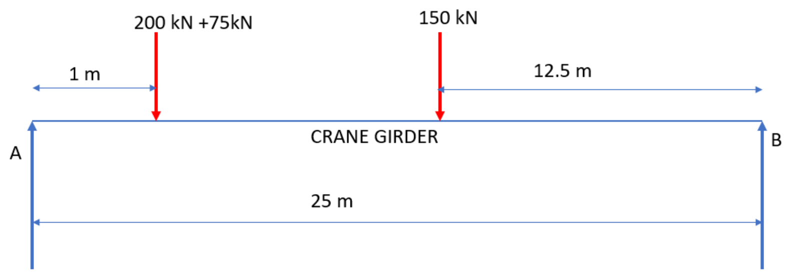
- ***Design Maximum load transferred from crane girder to gantry girder*** c



The maximum reaction/load in crane girder occurs when the crab or trolley along with hook if it is towards 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, $\sum M_B = R_A \times 25 - 150 \times 12.5 - 275 \times 24 = 0$ $R_A = 339$kN is the reaction developed at the end of Crane girder

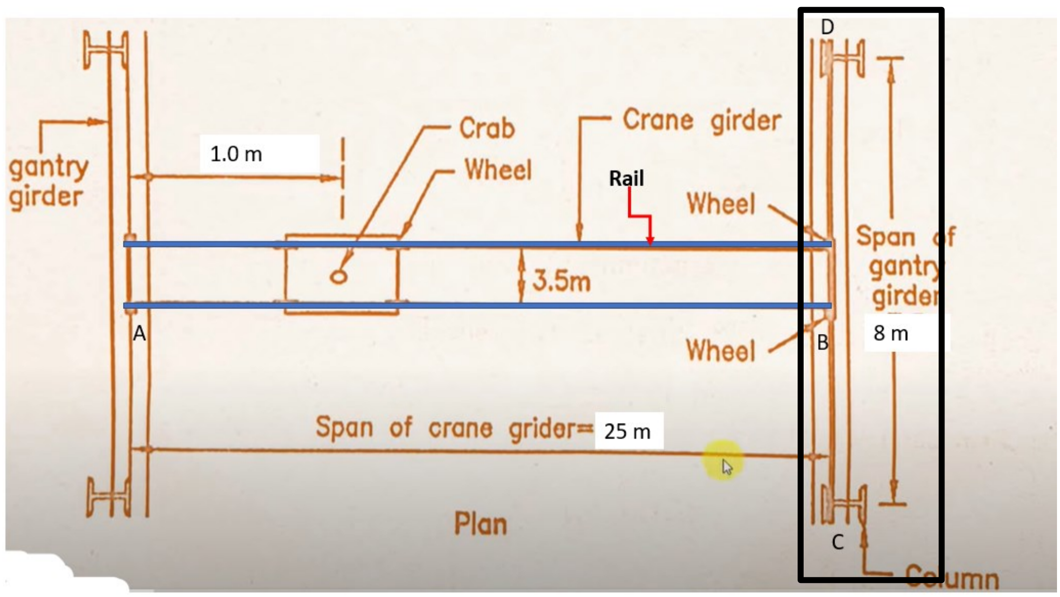
Since there are two wheels at each end of crane bridge, = 339/2 = 169.5kN

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

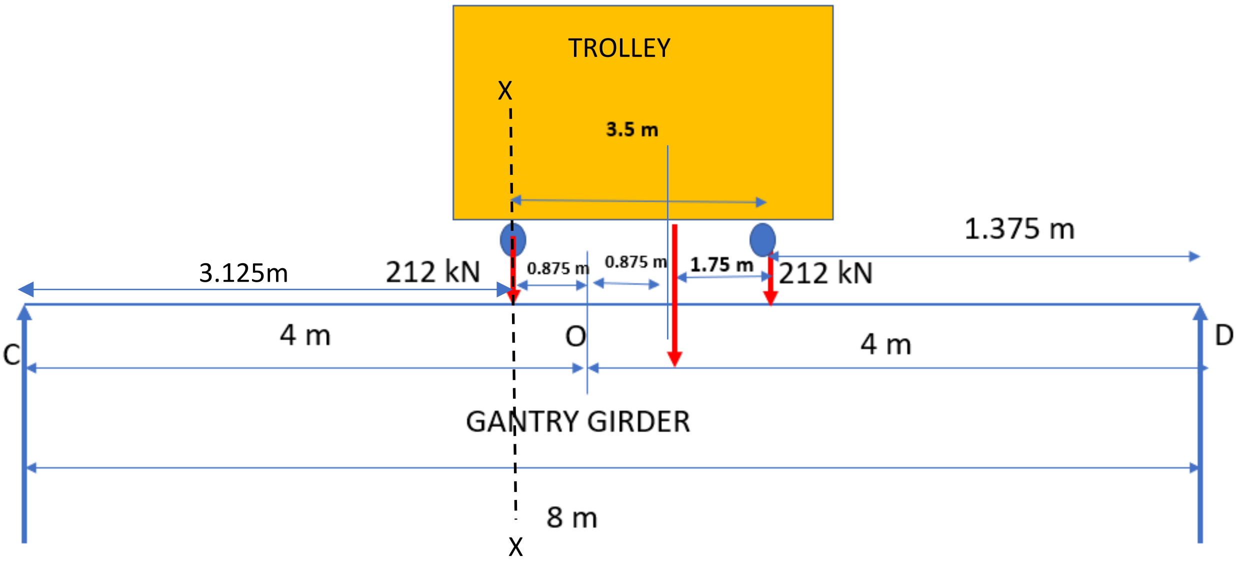
Vertical forces transferred through rails = 25% of **maximum static** wheel load (25% means 1.25)

Load on each wheel = 1.25 × 169.5 = 211.87 kN ≈ 212 kN

- ***Design Maximum bending moments on gantry girder***



The arrangement of wheel loads for Maximum bending moment is at **Centre of gravity of the wheel loads(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 = $517.6 \times 1.5 = 776.4$ kNm- Live load

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 = 0.3 kN/m (given)

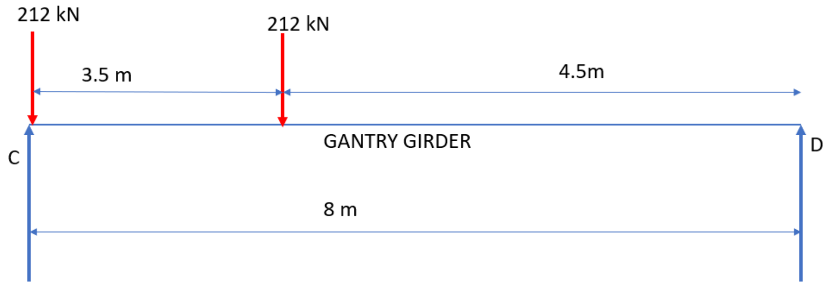
Total self-weight = 1.6 + 0.3 = 1.9 kN/m, factored self-weight = 1.5 x 1.9 = 2.85 kN/m

- ✓ B M due to self-weight or dead load = $2.85 \times 8^2/8$ = 22.8kNm- Dead load

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

- ***Design Maximum shear force***

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



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

$R_c = 331.25$kN

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 of trolley (there are 4 wheels) = 27.5 /4 = 6.875 ≈ 7kN

Factored lateral load = 7 × 1.5 = 10.5kN

Bending Moment due to lateral load

We know that 212 kN , B.M = 776.4kNm

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

212/10.5 = 776.4/ ? , ? =

$\frac{212}{10.5} = \frac{776.4}{?}$, 212 x ? = 10.5 x 776.4 , ? = 10.5 x 776.4/212 = 38.46kNm

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+10.5 = 518.49 kN

- **Preliminary trial section**

The trial section is selected based on deflection criteria, IS 800 2007, Table 6 Page 37

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

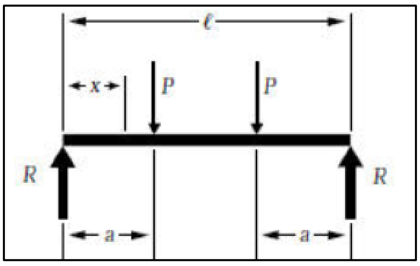
Maxi deflection for electrically operated crane, $\delta_{\max deflection}$ = Span of gantry girder/750

= 8000/750 = 10.67 mm(permissible deflection)

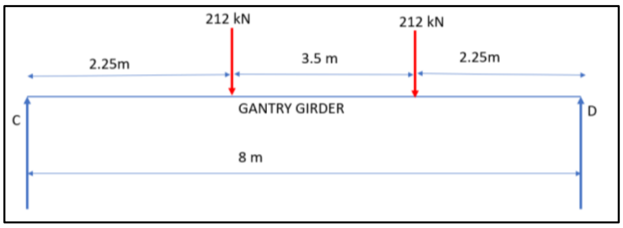
Maxi deflection = Deflection due to Dead load + Deflection due to Live load

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 of trolley (P))



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



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

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

Put $P = 212kN$, E = 2 $\times 10^5$ 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

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

$I = 1765 \times 10^6\ mm^4$

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

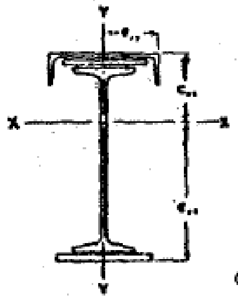


TABLE 12 *(Contd.)*

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

Joist: Designation	Joist: w	Top Flange: Channel Designation	Top Flange: Channel w	Top Flange: Plate Width	Top Flange: Plate Thickness	Bottom Flange: Plate Width	Bottom Flange: Plate Thickness	Weight per Metre (W)		Sectional Area a	Centre of Gravity C_{xx}	Mean Thickness of Flanges: Top	Mean Thickness of Flanges: Bottom
	Kg / N		Kg / N	mm	mm	mm	mm	kg	N	cm^2	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

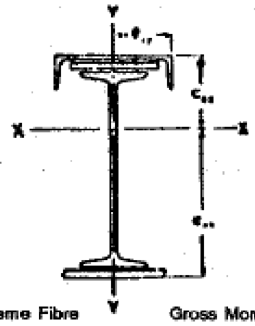


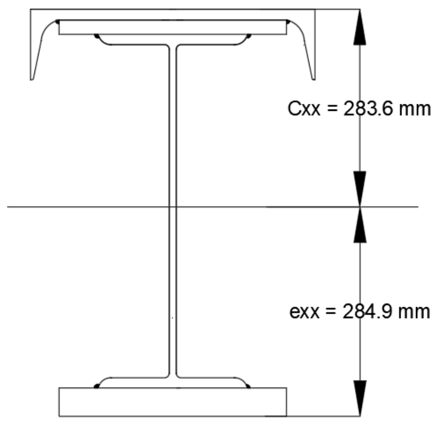
TABLE 12 *(Contd.)*

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
			I_{yy}			Z_{xx}			
e_{xx}	e_{yy}	I_{xx}	Whole Section	Top Flange Only	r_{yy}	Zc	Zt	M	S
cm	cm	cm^4	cm^4	cm^4	cm	cm^3	cm^3	kg.m × 10^3	kg × 10^3
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 (From Bottom)



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 = 121·22 cm^2	A = 62.93cm^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

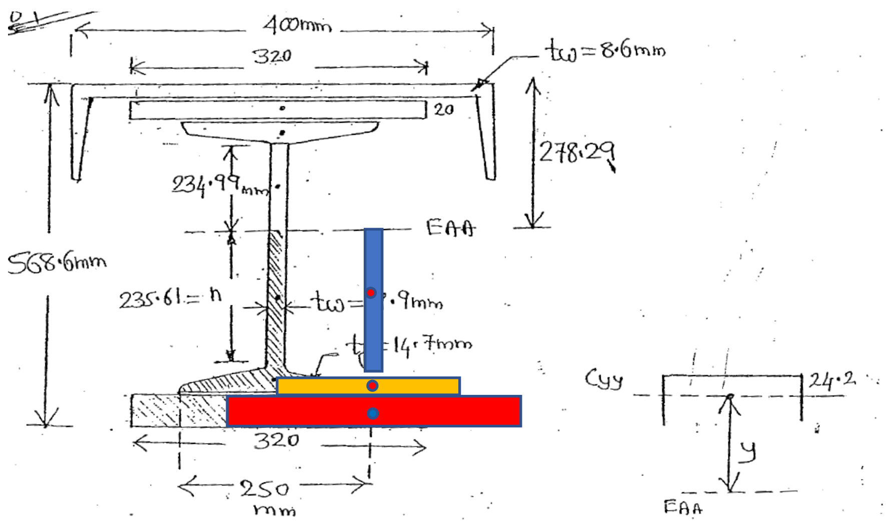
Location of Equal area axis

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

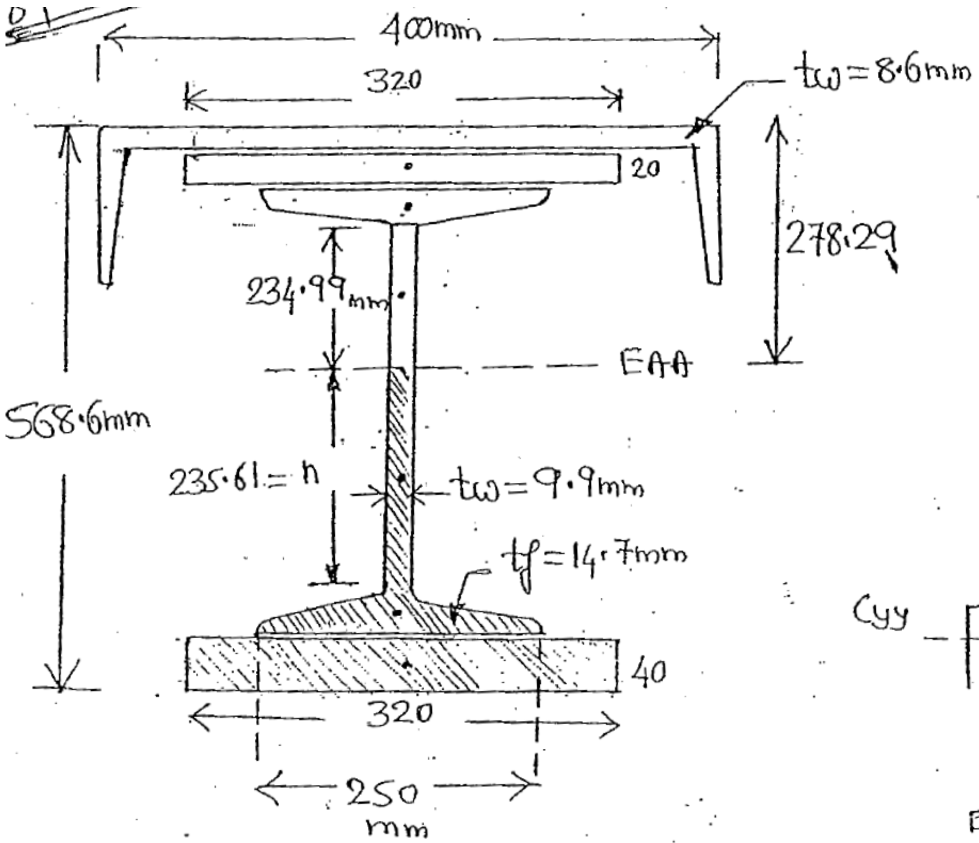
To locate Equal area axis '**n**' , Area of shaded portion = ½ total area of section

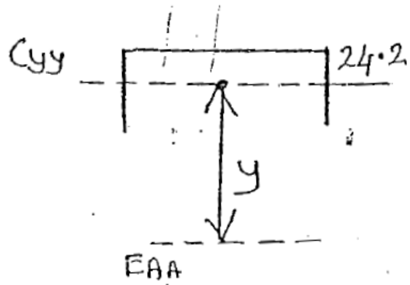
$9.9 \times \mathbf{n} + 250 \times 14.7 + 320 \times 40 = \frac{1}{2} \times (37615)$

n = 235.61 mm



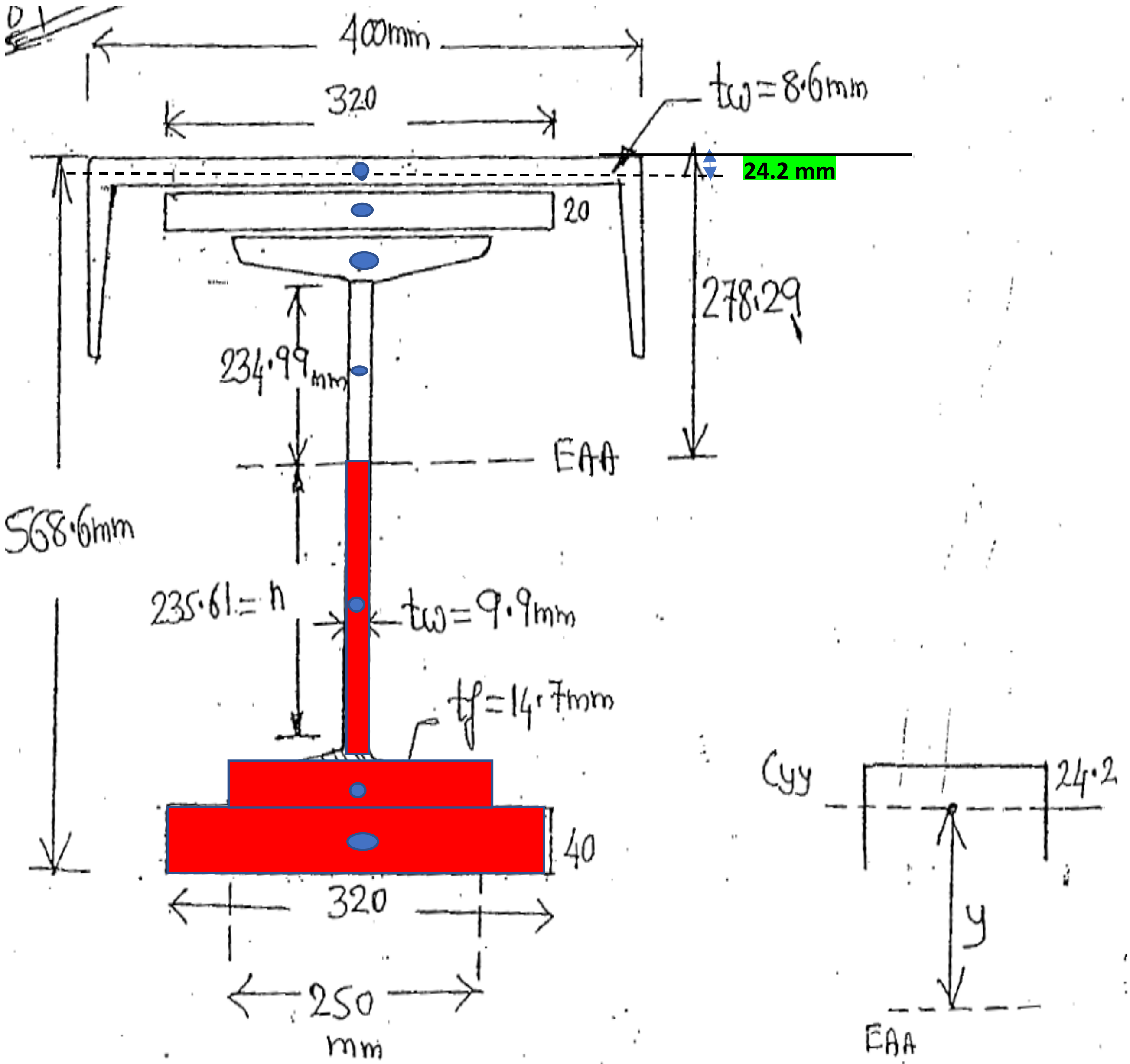
- **Plastic modulus** Zp – It is sum of areas of compression and tension zones multiplied by corresponding distance of the centroid of the compressive and tension area from the equal area axis





Calculation of Plastic modulus $Z_p = \sum a \times \overline{yi}$

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



Shaded area (a)	Centroidal distance from EAA $(\bar{y}i)$	Z_p (mm^3) $(a \times \bar{y}i)$	Unshaded area $(a)\ mm^2$	Centroidal distance from EAA $(\bar{y}i)$	Z_p (mm^3) $(a \times \bar{y}i)$
320×40	$235.61 + 14.7 + \frac{40}{2}$		234.99×9.9	$\frac{234.99}{2}$	
250×14.7	$235.61 + \frac{14.7}{2}$		250×14.70	$234.99 + 14.7/2$	
	$\frac{235.61}{2}$		320×20	$234.99 + 14.7 + \frac{20}{2}$	

9.9×235.61					
		$=\sum a \times \bar{y}i$	 Area of Channel section (From steel table) 6293	$278.29-24.2$	
Total	$=\sum a \times \bar{y}$	=	+		$=\sum a \times \bar{y}i$
Plastic Modulus $Z_p=\sum a \times \bar{y}$	$=9.05 \times 10^6 mm^3$				

- **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:

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

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:

α_{LT} = 0.21 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:

α_{LT} = 0.21 for rolled steel section

α_{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

55

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

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

Design bending strength or Moment of resistance, M_d = 1 × 9.05 × 10^6 × 187.96 = 1701.6 × 10^{6}N 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 / γ_{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

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

A_v = shear area, and

f_{yw} = yield strength of the web.

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

Major Axis Bending:

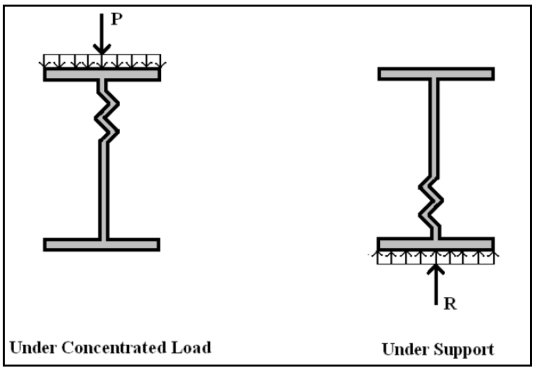
Hot-Rolled — $h\, t_w$

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>518

.28 kN (S F), it is safe.

- **Check for web crippling**



(**No need to write while solving**)

- 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 (check for web crippling)

8.7.4 *Bearing Stiffeners*

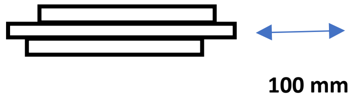
Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceeding the local capacity of the web at its connection to the flange, F_w, given by:

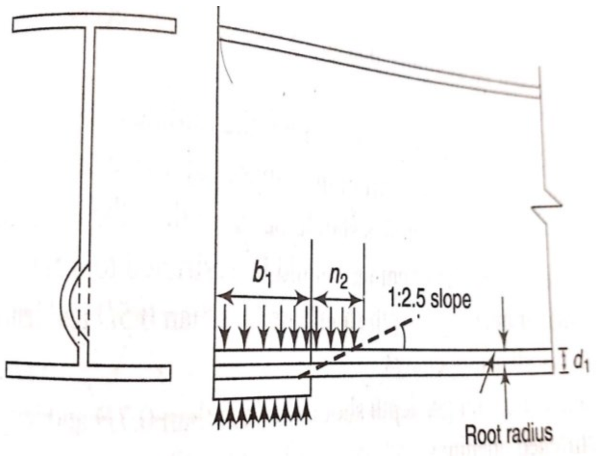
$$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0}$$

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 > 518.49 kN (Shear Force)

- **Check for buckling of web**

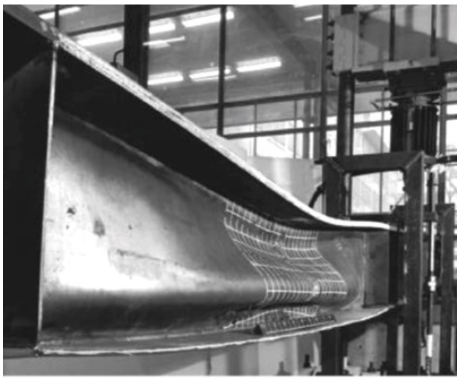
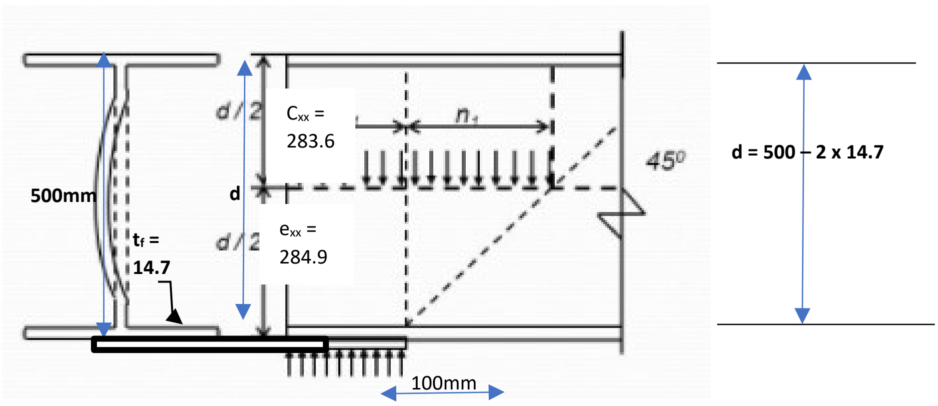


Figure 2. Vertical web buckling.

- 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,

Assume load dispersion of 45° at the mid depth of Gantry girder section n_1 = e_{xx} = 284.9 mm, (steel table of girder)

Thickness of web, t_w = 9.9 mm

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

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

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 (From steel table 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).

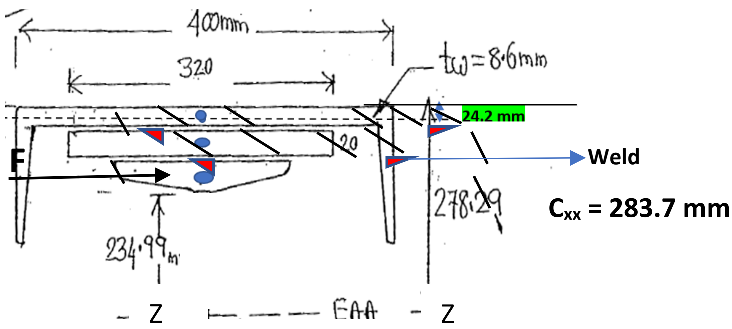
Since it is class "c", Use Table 9(c)

From Table 9 (c) Page 42 – IS 800 2007, for λ = 3.44 we do not have value of 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 > 518.49 kN (Shear Force)

- **Connections**

Using welded connections for Gantry girder



Shear Force at the junction for shaded portion = F = $\frac{V\, a\, \bar{y}}{I_Z}$, Where V is the shear force, $a\,\bar{y}$ is the area of shaded portion multiplied by centroidal distances measured from individual sections, I_Z is the moment of Inertia of the girder. [($I_Z = I_{xx}$) from Girder details - Steel table)]

$$= 518.28 \times \frac{[6293 \times (283.7 - 24.2\,) + (320 \times 20 \times (283.7 - 8.6 - 20/2)]}{2301.9 \text{ x } 10^6} = 0.745 \text{ kN/m} = 0.745 \text{ N/mm}$$

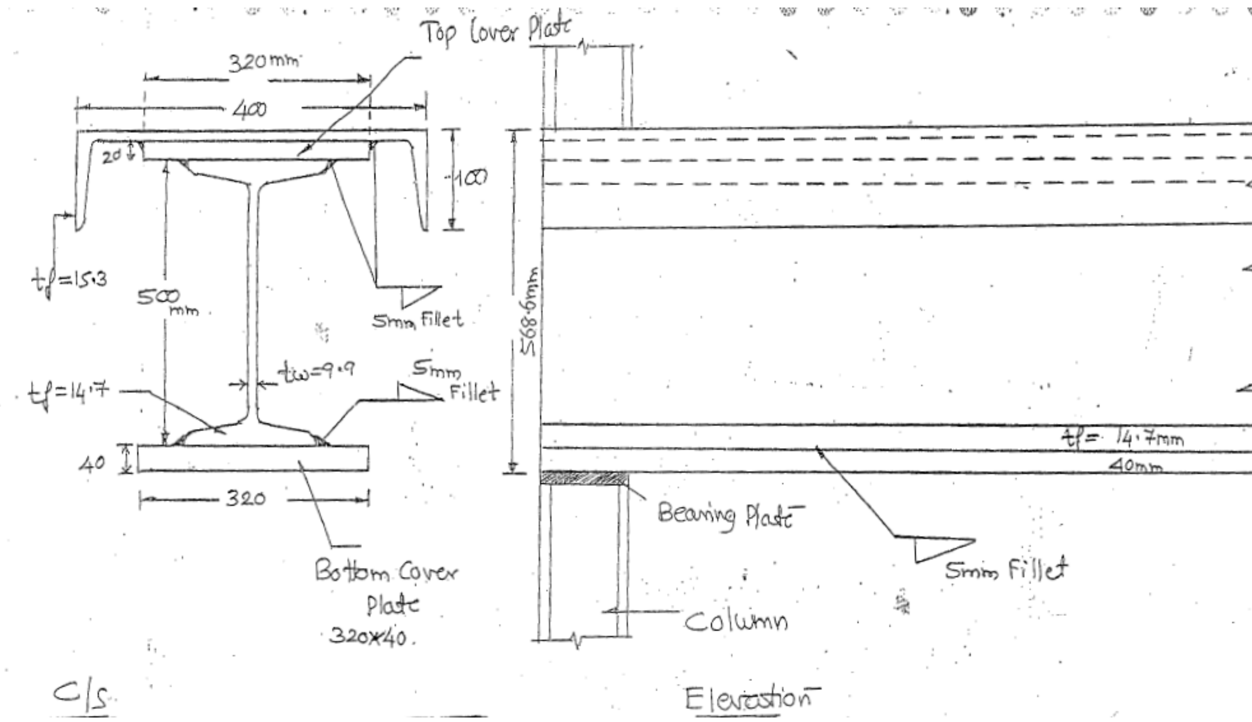
…. (1)

(Formula for weld)

Strength of weld = 2 × $\frac{[0.7 \times s \times 1 \text{ mm} \times 410\,]}{\sqrt{3} \times 1.25}$ where 's' is the size of weld…..(2)

(1) = (2), s = 2.8 mm

Provide 6 mm size weld



***Optional**

Bracket connections for Gantry girder

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

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

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

Use 8 number of bolts for bracket connections

4. SOLUTIONS

- **Load combinations**

Member	D L (kN)	L L(kN)	W L(kN)	DL+LL+WL (kN) (1)	DL+LL (kN) (2)	DL+WL (kN) (3)	Max Load (1) (2)and (3)
Rafter(AB)	-58.0	-52.5	+111.6	1.1	-110.5	53.6	-111.6
Tie(AH)	+52.0	+47.0	-102.4	-3.4	99	-50.4	99
Sling(BG)	+20.3	+18.4	-63.0	-24.3	38.7	-42.7	38.7

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

AB, BC are top chord member, maximum load is -111.6 kN

Factored load or $Force$ = 111.6 x 1.5 = 167.4 kN

Length of members AB/ BC/ CD

cos 30.96 = 2.5/ AB , AB = 2.5/ cos 30.96 = 2.9 m = 2900 mm

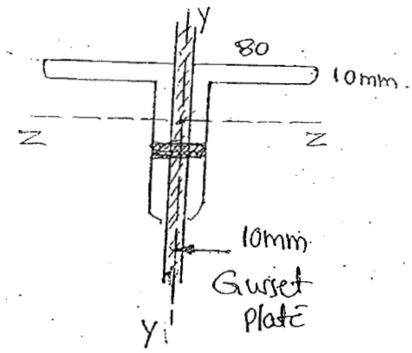
Assume design compressive stress, f_{cd} = 110 N/mm^2

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

Gross area, $A_g = \frac{Force}{f_{cd}} = \frac{111.6 \times 10^3}{110}$ = 1014.54 mm^2 = 10.14 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

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Effective length for Top chord members (AB) , $L_{eff} = K \times L = 0.85 \text{ x } L = 0.85 \text{ x } 2.9 = 2465$ m

Slenderness ratio, λ $= \frac{K\,L}{r_{min}} = \frac{L_{eff}}{r_{min}} = \frac{2465}{24.1} = 102.2$

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)

(*Clause* 7.1.2.1)

KL/r ↓	Yield Stress, f_y (MPa)																	
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6

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From Table 9 (c) Page 42 – IS 800 2007, through interpolation, for λ = 102.2 we get design Compressive Stress, $f_{cd} = 92.2\ N/mm^2$

(Page 34) Design compressive strength, $P_c = f_{cd} \times A_g$ =92.2 × 3010

= 277.5 kN > 167.4 kN

So selected section is safe.

- **Connections**

  Using M 22 Property Class 5.6 bolts (Try to use same diameter of bolts if possible)

- **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)

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

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

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

- **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 = $\frac{277.5}{92.54} = 2.99 \approx 3$

Hence provide 2 ISA 80 × 80 × 10 mm with 3 bolts

- **Design of Bottom chord members (AH)-Tension members**

  Taking Max Force = 99 kN

  Factored Tensile Force T_{dg} = 99 x 1.5 = 148.5 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, $A_g = \frac{148.5 \times 10^3 \times 1.1}{250} = 653.4\ mm^2$

  Since it is bottom member, increase the area by 30% = 1.3 × 653.4 = 849.4 mm^2= 8.49 cm^2

  From Steel table Page No 18, table 6 (Double angle)

ISA 8080	80 x 80	6.0	14.6	143.2	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**

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

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

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

No of bolts = $\frac{148.5}{55.53} = 3$

Hence provide 2 ISA 80 × 80 × 6 mm with 3 bolts

(In case of High Strength Friction Grip Bolts (HSFG) - Shear capacity only needs to be calculated by CL 10.4.3, page 76 and then calculate no of bolts based on shear capacity) 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 ($= 0.70 f_{ub}$).

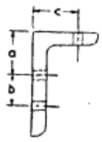
To find partial safety factor

Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

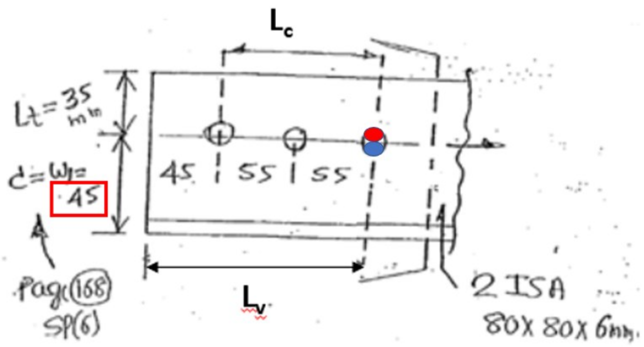
Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}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

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)**

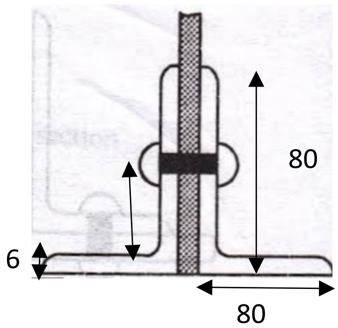
Page 33 CL 6.3.3 IS 800 - 2007

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

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



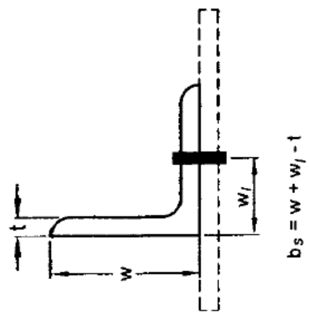
w = outstand leg width or **width of unconnected leg** = 80 mm

L_c = **Distance between the outermost bolts in the end joint measured along the load direction** = 55 +55 = 110 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 (CL 6.3.3)

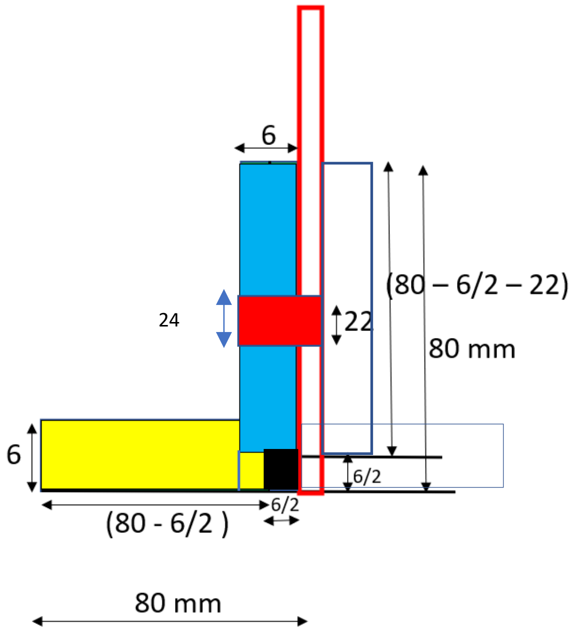
$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c)$$

$= 1.132$

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

As per IS 800 – 2007, β = 1.132 ≥ 0.7 ≤ 1.44

Hence take β = 1.132



Angle section attached to gusset plate

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

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

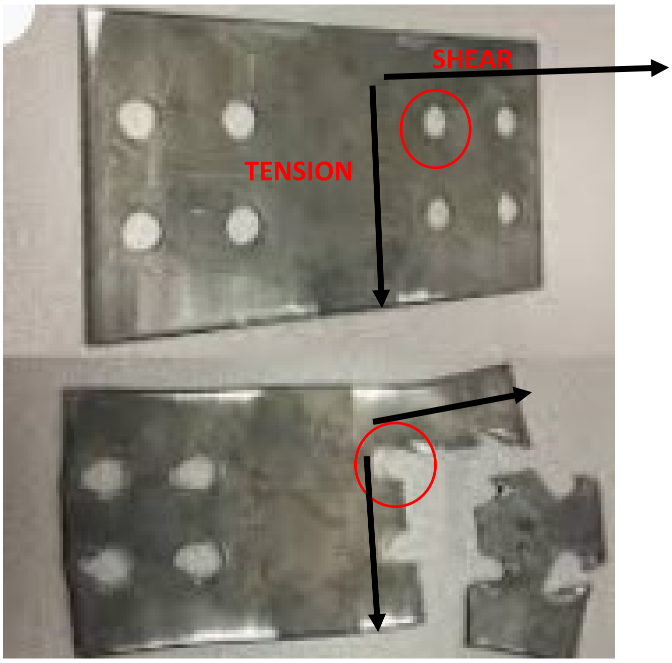
$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta\, A_{go}\, f_y / \gamma_{m0}$$

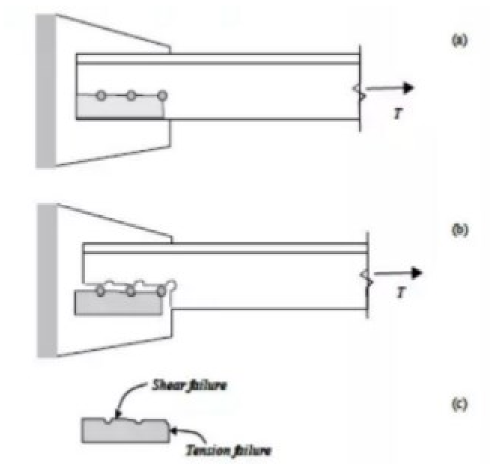
For double angle section multiply the value of T_{dn} by "2"

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 > 148.5 kN , It is safe.

- **Check for block shear**





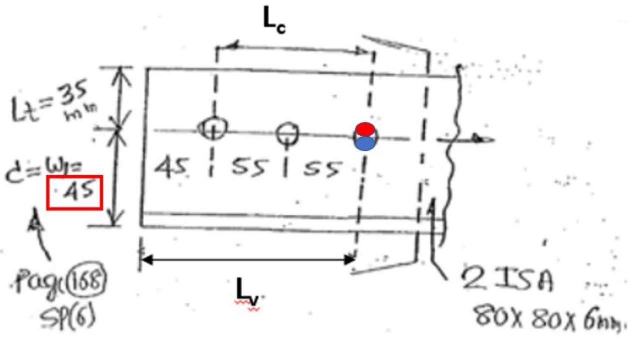
Page 33 CL 6.4.1

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

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

or

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



Length of shearing action, L_v = 45 + 2 × 55 = 155 mm
Length of tensile action, L_t = 35 mm
Gross area in shear parallel to force , $A_{vg} = L_v \times t = 155\times 6 = 930\ mm^2$
Net area in **shear** parallel to force, $A_{vn} = A_{vg} - \mathbf{2.5} \times d_o \times t$
$= 930 - 2.5 \times 24 \times 6 = 570\ mm^2$
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

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

For two angles, $T_{db} = 2 \times [\,930 \times \frac{250}{\sqrt{3}\times 1.1} + 0.9 \times 138 \times \frac{410}{1.25}\,]$ = 337.4 kN > 148.5 kN

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

For two angles, $T_{db} = 2 \times [\,0.9 \times 570 \times \frac{410}{1.25\times\sqrt{3}} + 210 \times \frac{250}{1.1}\,]$ = 289.7 kN> 148.5 kN

Hence 2 ISA 80 × 80 × 6 mm is safe .

- **Design of Inner compression members (BG)**

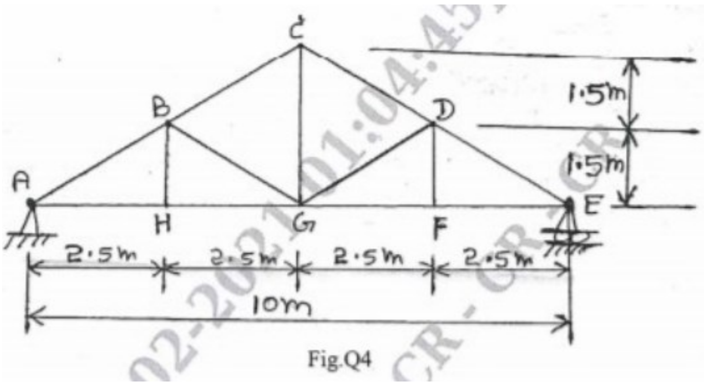


Fig.Q4

Taking Maximum Force = 38.7.0 kN
Factored Force = 1.5 × 38.7 = 58.05 kN
The length of BG is = $\sqrt{(1.5^2 + 2.5^2)}$ = 2.91 m

Maximum Length = 2.91 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 $= \frac{Factored\ Force}{f_{cd}} = \frac{58.05 \times 10^3}{50} = 1161$ mm^2 = 11.61 cm^2

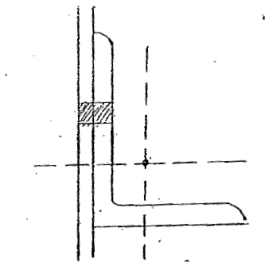
From Steel Table, Try Single ISA 100 x 100 x 10 mm (Table 1, Page 4)

Area = 19.03 cm^2 (You can choose higher area also)

$r_{xx} = r_{yy}$ = 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, $l = 0.85 \times L = 0.85 \times 2910 = 2473.5\ mm$
$\varepsilon = 1$, $E = 2 \times 10^5$ N/mm^2 $b_1 = b_2 = 100$ mm, t = 10 mm

$$\lambda_{VV} = \frac{\frac{L}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{2473.5}{30.5}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = \frac{81.09}{0.198} = 0.913$$

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

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

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

Equivalent slenderness ratio, $\lambda_e = 1.124$
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 $\alpha = 0.49$, $\lambda_e = 1.124$, $\Phi = 1.36$

To find f_{cd}

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

(Single angle section) Design compressive stress, $f_{cd} = \frac{250}{1.1(1.36+(1.36^2 - 1.124^2)^{0.5})} =$ 106.9 N/mm^2

The design compressive strength of a member is given by:

Load P_d = A × f_{cd} = 1903 × 106.9 = 203.42 kN > 58.500 kN
Hence it is safe.
Design of connection using M 22, 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)

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

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

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

- **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

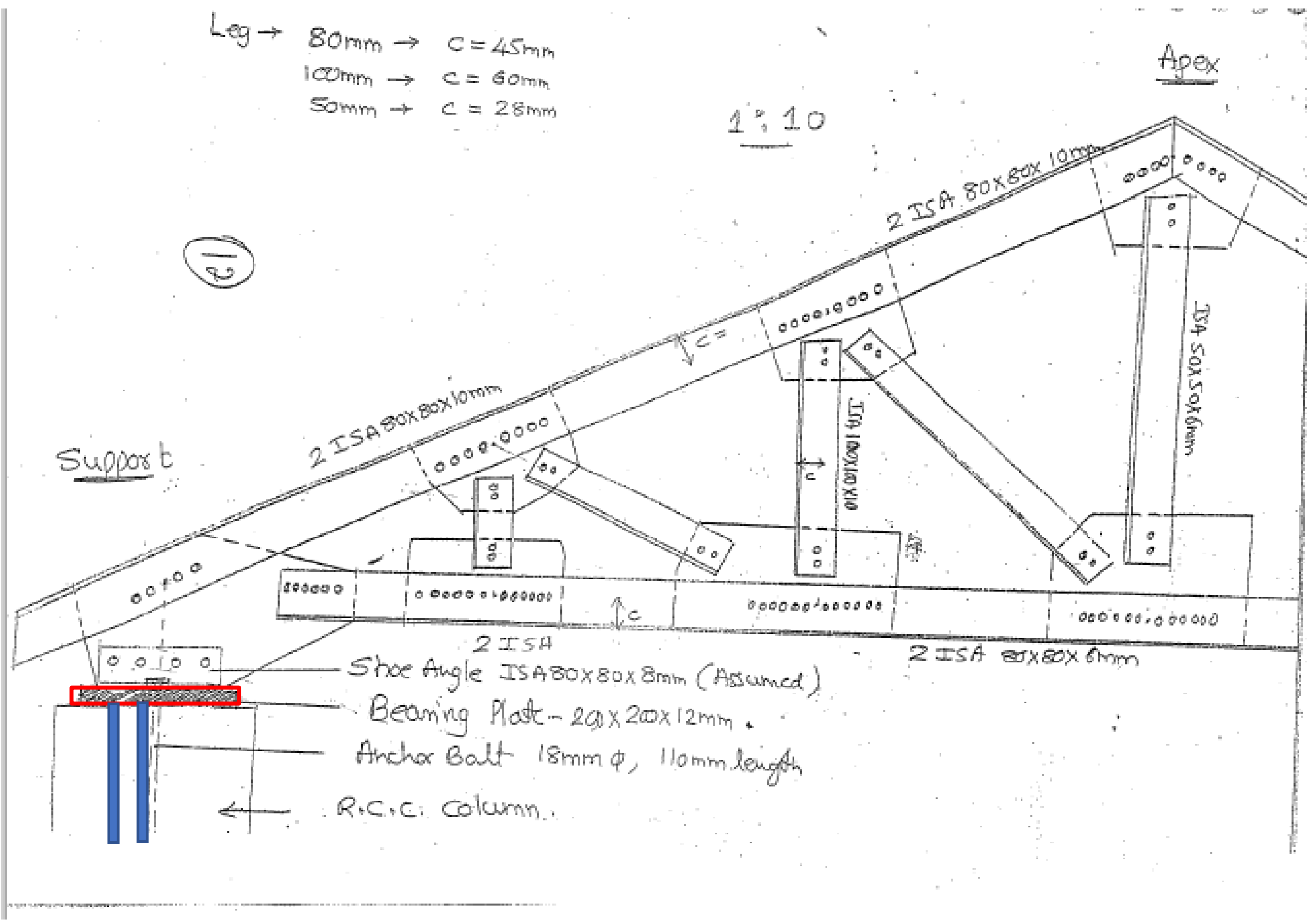
where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0 (try to copy value of k_b)

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

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

No of bolts = $\frac{58.5}{68.47} = 2$



Leg → 80mm → C = 45mm
100mm → C = 60mm
50mm → C = 28mm
1 : 10
Apex
Support
2 ISA 80x80x10mm
2 ISA 80x80x10mm
ISA 100x100x10
ISA 50x50x6mm
2 ISA
2 ISA 80x80x6mm
Shoe Angle ISA80x80x8mm (Assumed)
Bearing Plate - 200x200x12mm
Anchor Bolt 18mm ϕ, 110mm length
R.C.C. Column