USN | | | | | | | | | |

Internal Assessment Test 2 – Nov. 2021

Sub:	Design of RC and steel structural elements				Sub Code:	18CV72/17CV72	Branch:	CIVIL
Date:	17.12.21	Duration:	90 min's	Max Marks: 50	Sem / Sec:	ALL		OBE

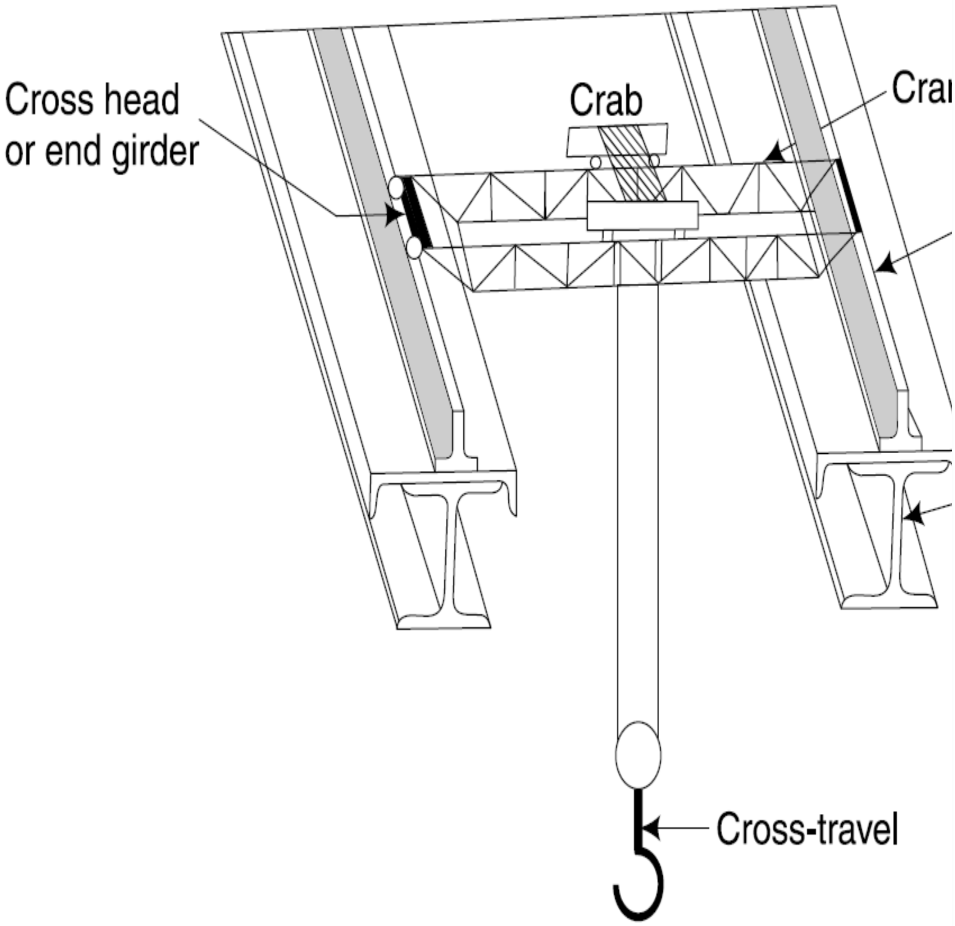
	Answer any one Questions- Use of IS 456 -2000 and IS 800-2007, Steel table is permitted	MARKS	CO	RBT
1 (a)	Design a simply supported gantry girder for the following dar. The girder is electrically operated. Take yield stress of steel as 250 MPa. i. Span of crane girder = 20 m ii. Span of gantry gider= 7m iii. Capacity of crane = 250 kN iv. Self weight of crane = 200 kN and self-weight of trolley = 60 kN v. Wheel base = 3.4 m vi. Minimum hook approach =1.1 m vii. Self weight of rail = 0.3 kN/m	[50]	CO2	L2

DESIGN OF GANTRY GIRDER

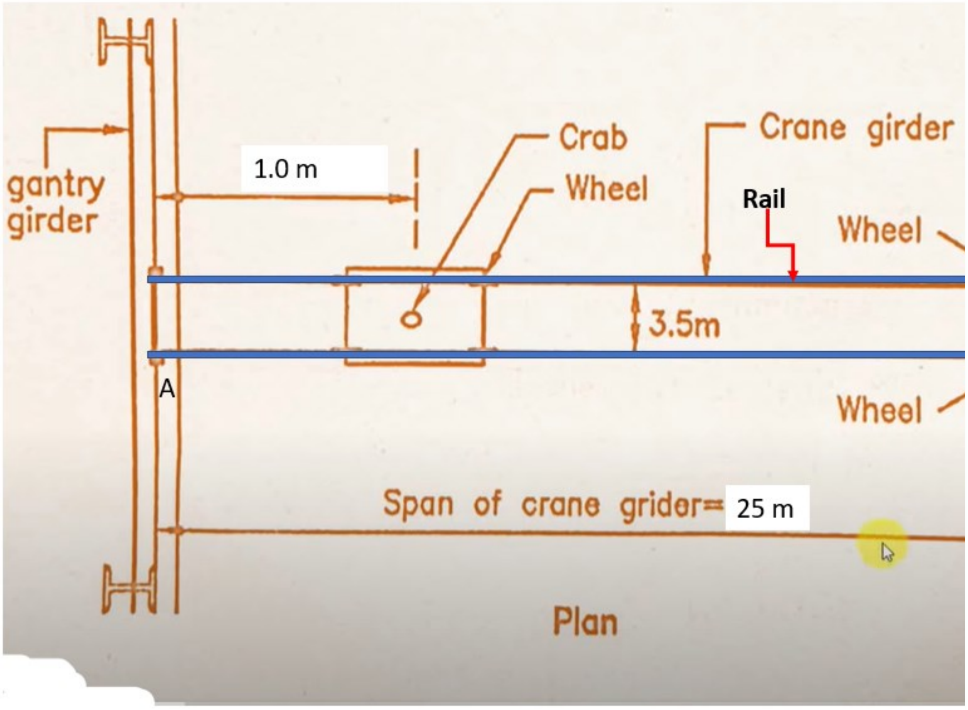
1. **Design a gantry girder to be used in an industrial building carrying an electrically operated overhead travelling crane, for the following data**

- **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 required.**

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

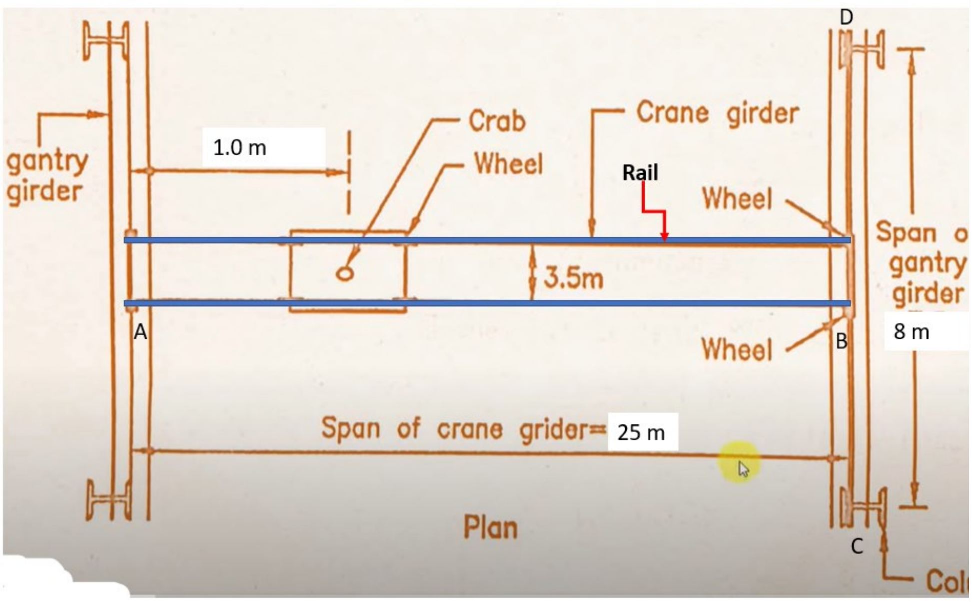


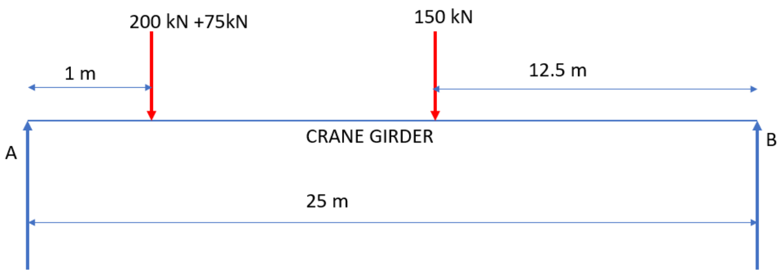
- ***Design Maximum load transferred from crane girder to gantry girder*** c



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, $\sum M_B = R_A \times 25 - 150 \times 12.5 - 275 \times 24 = 0$ $R_A = 339kN$ is the reaction developed at the end of Crane girder

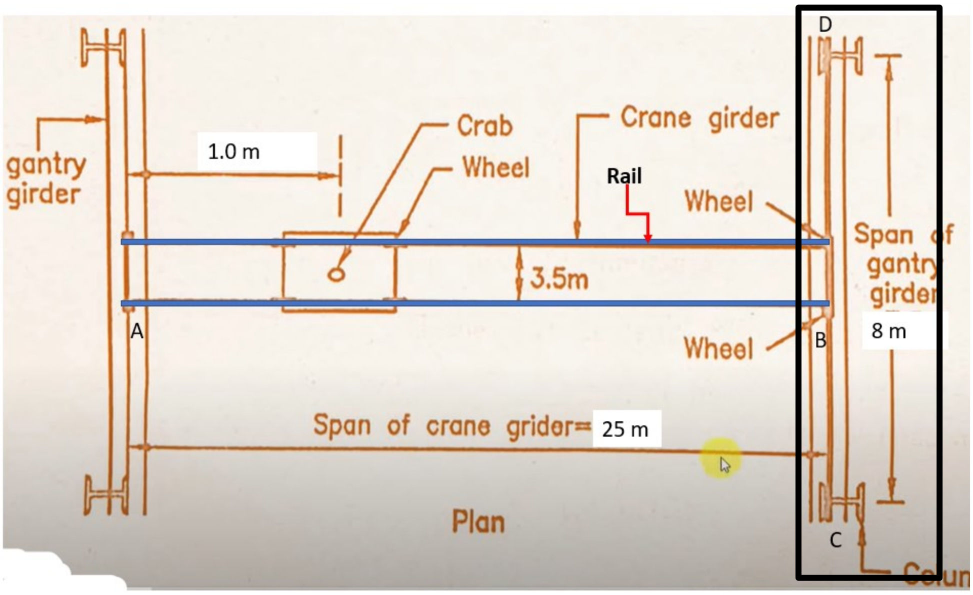
Since there are two wheels at each end of crane bridge, = 339/2 = 169.5kN

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

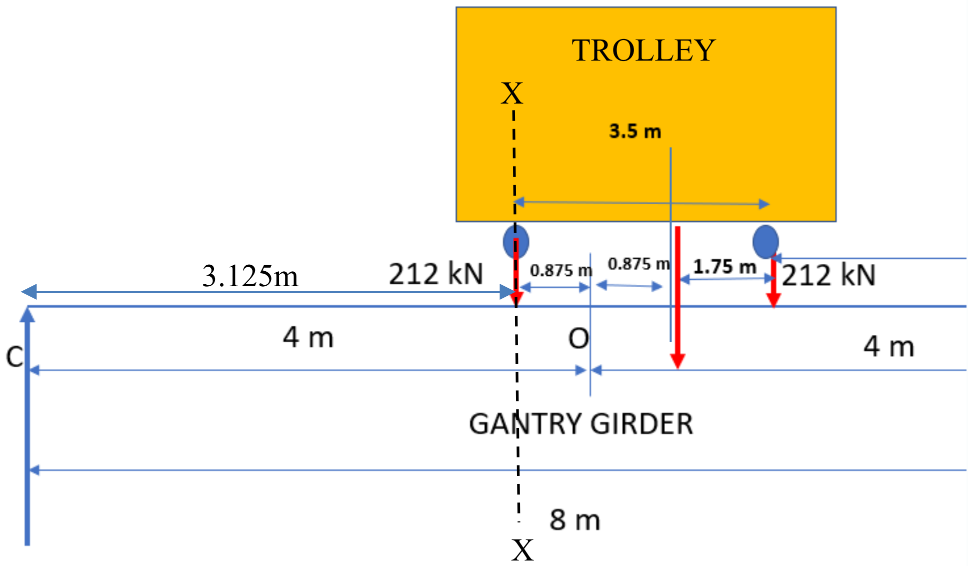
Vertical forces transferred through rails = 25% of **maximum static** wheel load (25% means 1.25)

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 × 1.5 = 776.4 kNm- Live load

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

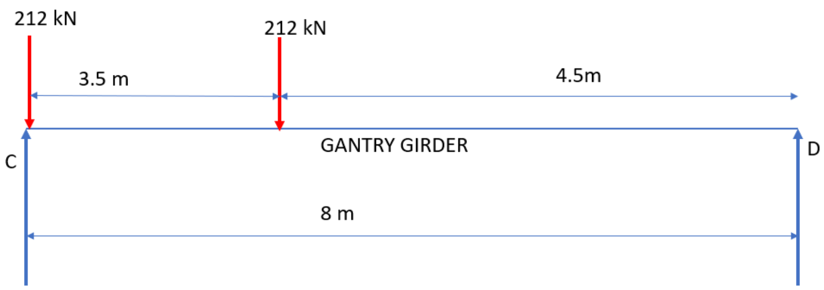
Assume self-weight of gantry girder as 1.6kN/m
Self-weight of rail = 300N/m = 0.3 kN/m (given)
Total self-weight = 1.6 + 0.3 = 1.9 kN/m, factored self-weight = 1.5 x 1.9 = 2.85 kN/m

✓ B M due to self-weight or dead load = 2.85 x $8^2/8$ = 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

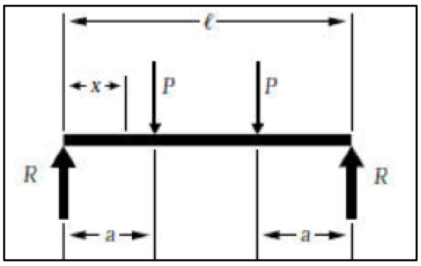
Table 6 Deflection Limits

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

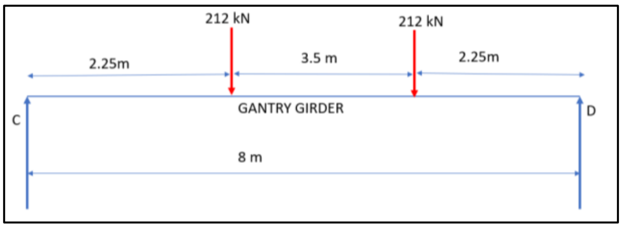
Maxi deflection for electrically operated crane, $\delta_{\max deflection}$ = Span of gantry girder/750

= 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))



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



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

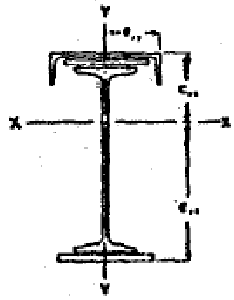


TABLE 12 *(Contd.)*

SINGLE JOIST WITH CHAN PLATES ON THE FLANGES (

Joist Designation	Joist w (Kg / N)	Top Flange Channel Designation	Channel w (Kg / N)	Top Flange Plate Width × Thickness (mm × mm)	Bottom Flange Plate Width × Thickness (mm × mm)	Weight per Metre (W) kg	Weight per Metre (W) N	Sectional Area a (cm²)	Centre of Gravity C_{xx} (cm)
ISWB 500	95.2 933.9	ISMC 400	49.4 484.6	320 × 10.0	320 × 20.0	219.9	2157.2	280.15	24.4
				12.0	25.0	237.5	2329.9	302.55	25.6
				16.0	32.0	265.1	2600.6	337.75	26.9
				20.0	40.0	295.3	2896.9	376.15	28.3
		ISMC 350	42.1 413.0	250 × 10.0	320 × 20.0	207.1	2031.7	263.88	25.7
				12.0	25.0	223.6	2193.5	284.88	27.0
				16.0	32.0	249.1	2443.7	317.28	28.5
				20.0	40.0	277.0	2717.4	352.88	30.0
		ISMC 400	49.4 484.6	—	320 × 10.0	169.7	1664.8	216.15	22.8
				—	12.0	174.7	1713.8	222.55	23.6
				—	16.0	184.7	1811.9	235.35	25.2
		ISMC 350	42.1 413.0	—	320 × 10.0	162.4	1593.1	206.88	23.6
				—	12.0	167.4	1642.2	213.28	24.5
				—	16.0	177.5	1741.3	226.08	26.1
ISMB 450	72.4 710.2	ISMC 300	35.8 351.2	—	250 × 10.0	127.9	1254.7	162.91	20.9
				—	12.0	131.8	1293.0	167.91	21.7
				—	16.0	139.7	1370.5	177.91	23.1
		ISMC 250	30.4 298.2	—	250 × 10.0	122.4	1200.7	155.94	21.7
				—	12.0	126.3	1239.0	160.94	22.4
				—	16.0	134.2	1316.5	170.94	23.9
		ISMC 225	25.9 254.1	—	200 × 10.0	114.0	1118.3	145.28	21.5
				—	12.0	117.2	1149.7	149.28	22.2
				—	16.0	123.5	1211.5	157.28	23.5

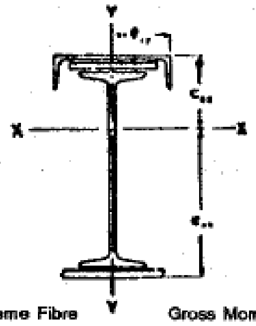


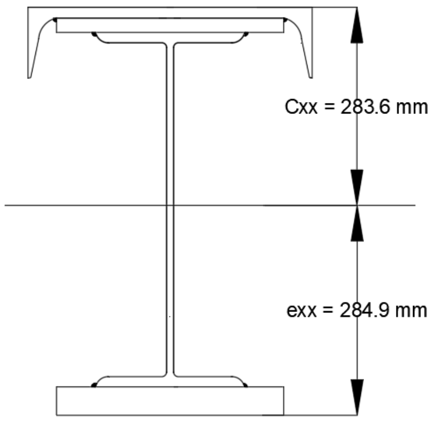
TABLE 12 *(Contd.)*

SINGLE JOIST WITH CH
PLATES ON THE FLANGE

Extreme Fibre Distance e_{xx}	Extreme Fibre Distance e_{yy}	Gross Moments of Inertia I_{xx}	Gross Moments of Inertia I_{yy} Whole Section	Gross Moments of Inertia I_{yy} Top Flange Only	Radius of Gyration r_{yy}	Moduli of Section Z_{xx} Zc	Moduli of Section Z_{xx} Zt
cm	cm	cm^4	cm^4	cm^4	cm	cm^3	cm^3
29.46	20.00	152781.2	26262.6	19307.2	9.68	6262.3	5185.5
28.93		170717.9	28174.1	19853.4	9.65	6661.7	5900.4
28.71		198797.6	31177.8	20945.7	9.61	7377.1	6923.9
28.49		230194.5	34454.6	22038.1	9.57	8114.6	8079.2
28.07	17.50	143834.0	19759.2	12803.9	8.65	5587.0	5124.9
27.45		159945.7	21385.0	13064.4	8.66	5911.3	5826.3
27.09		184963.8	23817.3	13585.3	8.66	6485.6	6827.6
26.75		212768.9	26522.6	14106.2	8.67	7077.7	7954.5
29.05	20.00	106172.6	20801.3	16576.5	9.81	4654.5	3654.9
28.41		111454.1	21347.4	16576.5	9.79	4712.8	3922.9
27.25		121362.3	22439.7	16576.6	9.76	4815.0	4452.9
28.12	17.50	101911.5	15726.5	11501.7	8.72	4301.4	3624.5
27.47		106854.4	16272.6	11501.8	8.73	4354.4	3889.8
26.30		116100.2	17364.9	11501.9	8.76	4447.3	4413.8
25.63	15.00	62983.5	8498.7	6779.4	7.22	3008.6	2438.8
25.25		66244.2	8759.1	6779.5	7.22	3051.8	2623.1
24.22		72359.1	9279.9	6779.6	7.22	3127.4	2987.2
25.00	12.50	60394.7	5952.9	4233.7	6.18	2781.6	2416.0
24.42		63446.4	6213.3	4233.8	6.21	2820.9	2598.3
23.38		69152.5	6734.1	4233.8	6.28	2889.5	2958.0
25.07	11.25	55138.5	4195.3	3111.5	5.37	2556.1	2199.5
24.59		57604.5	4328.6	3111.5	5.38	2589.5	2342.2
23.73		62272.6	4595.3	3111.6	5.41	2649.1	2623.8

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)
$A = 121{\cdot}22\ cm^2$	$A = 62.93cm^2 = 6293\ mm^2$
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

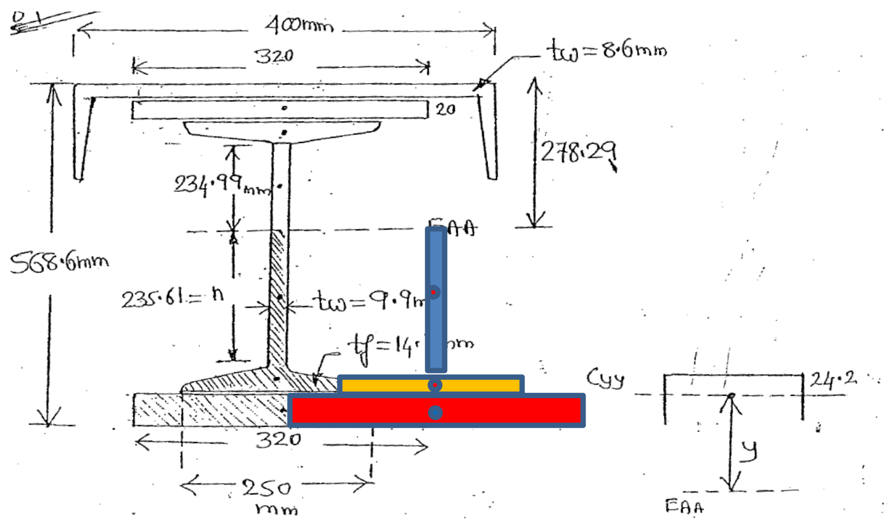
Location of Equal area axis

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

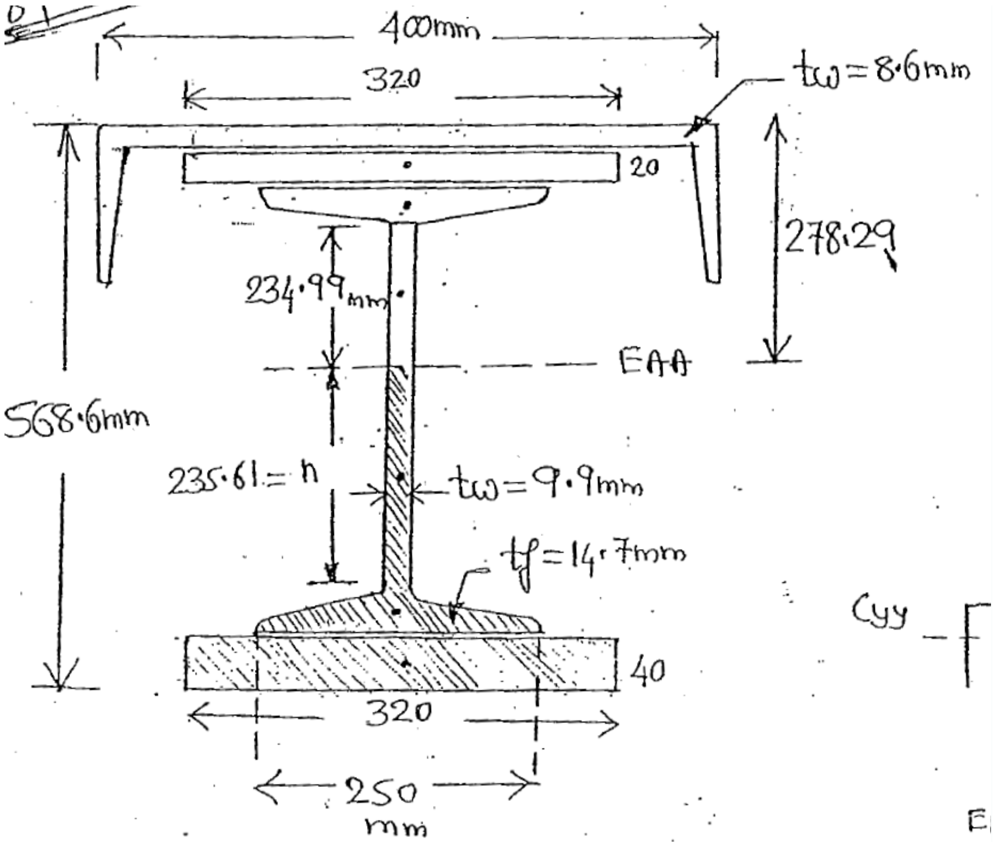
To locate Equal area axis '**n**' , Area of shaded portion = ½ total area of section

$9.9 \times \mathbf{n} + 250 \times 14.7 + 320 \times 40 = ½ \times (37615)$

n = 235.61 mm

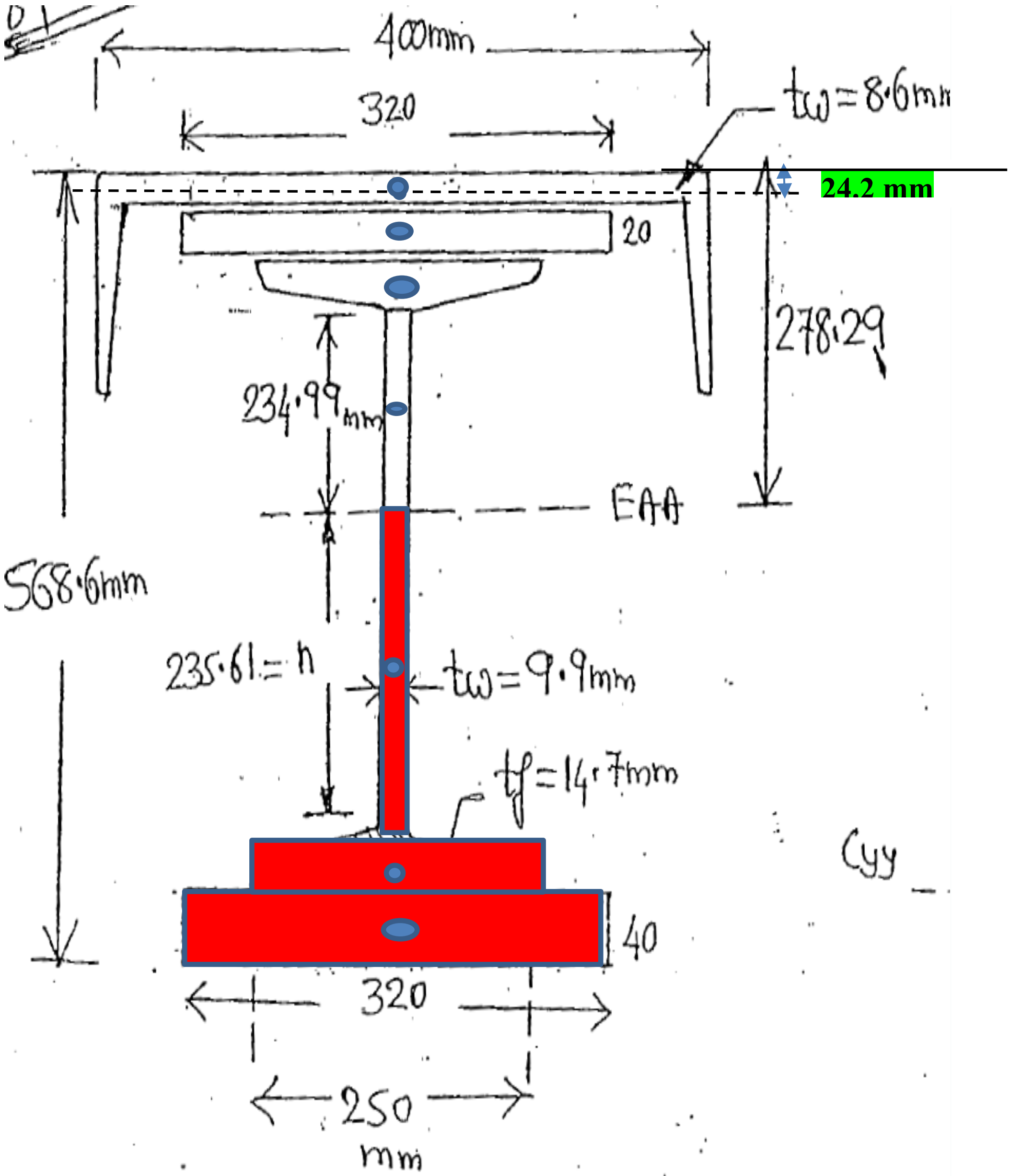


- **Plastic modulus** Zp – It is sum of areas of compression and tension zones multiplied by corresponding distance of the centroid of the compressive and tension area from the equal area axis



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



rea	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
)	$235.61 + 14.7 + \frac{40}{2}$		234.99×9.9	$\frac{234.99}{2}$	
.7	$235.61 + \frac{14.7}{2}$		250×14.70	$234.99 + 14.7/2$	
.61	$\frac{235.61}{2}$		320×20	$234.99 + 14.7 + \frac{20}{2}$	
		$= \sum a \times \bar{y}i$			

			 Area of Channel section (From steel table) 6293	$278.29 - 24.2$	
Total	$=\sum a \times \bar{y}$	=	+		$=\sum$
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 $\times 10^5$ N/mm², 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/mm²

Find f_{bd} , **Table 13 (a) Page 55**, IS 800- 2007, for $f_{cr,b}$ = 485.65 N/mm², α_{LT} = 0.21, f_y = 250 N/mm²

Table 13(a) Design Bending Compressive Stress Corresponding to Lateral Buckling, f_{bd}, α_{LT} = 0

(*Clause* 8.2.2)

$f_{cr,b}$	f_y														
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420
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
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
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
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
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
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
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
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
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
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
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
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
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

55

Through interpolation, the value of f_{bd} = 187.96 N/mm²

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

Design bending strength or Moment of resistance, M_d = 1 × 9.05 $\times 10^6 \times$ 187.96 = 1701.6 $\times 10^6$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 $V_d = \dfrac{V_n}{\gamma_{mo}} = \dfrac{A_v\, f_{yw}}{\sqrt{3}\, \gamma_{mo}}$

Major Axis Bending:

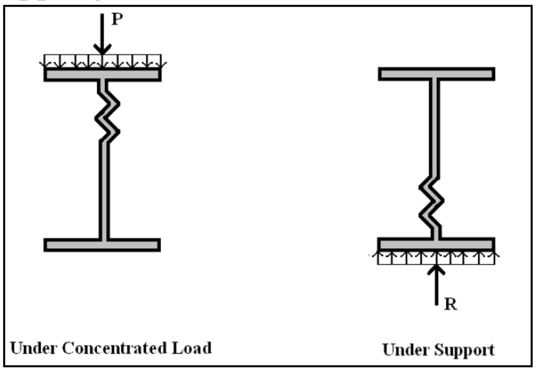
Hot-Rolled — $h\, t_w$

A_v is shear area = h × t_w = 568.6 × 9.9 = 5629 mm²

$V_d = \dfrac{5629 \times 250}{\sqrt{3} \times 1.1}$ =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)

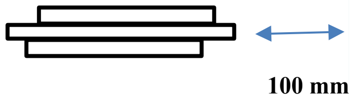
8.7.4 *Bearing Stiffeners*

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:

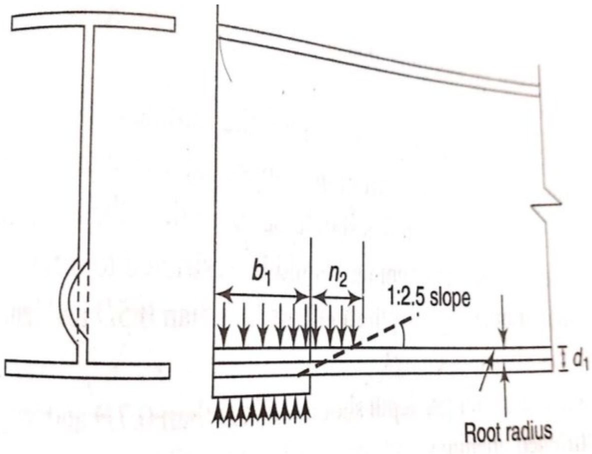
$$F_w = (b_1 + n_2)\, t_w f_{yw} / \gamma_{m0}$$

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**

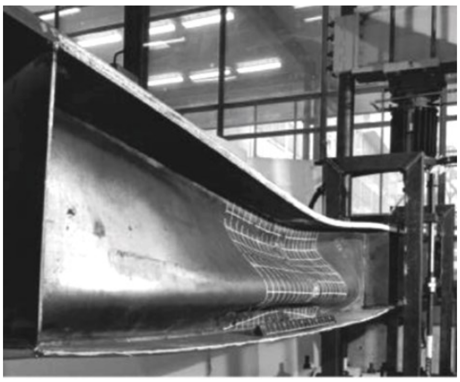
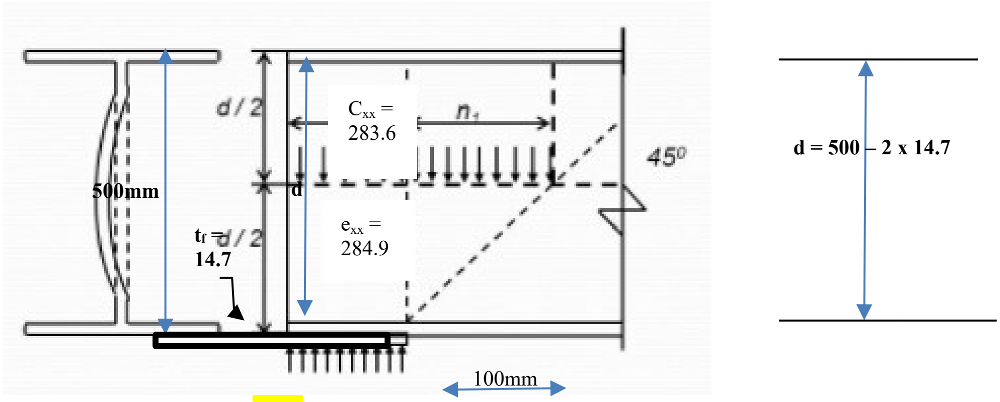


Figure 2. Vertical web buckling.

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

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 (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).

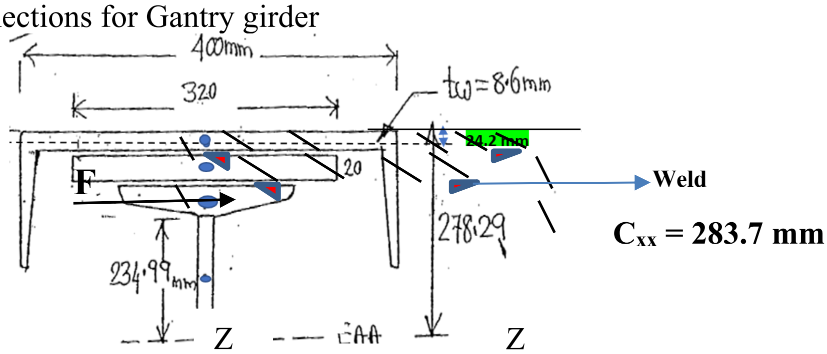
Since it is class "c", Use Table 9(c)

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)]

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

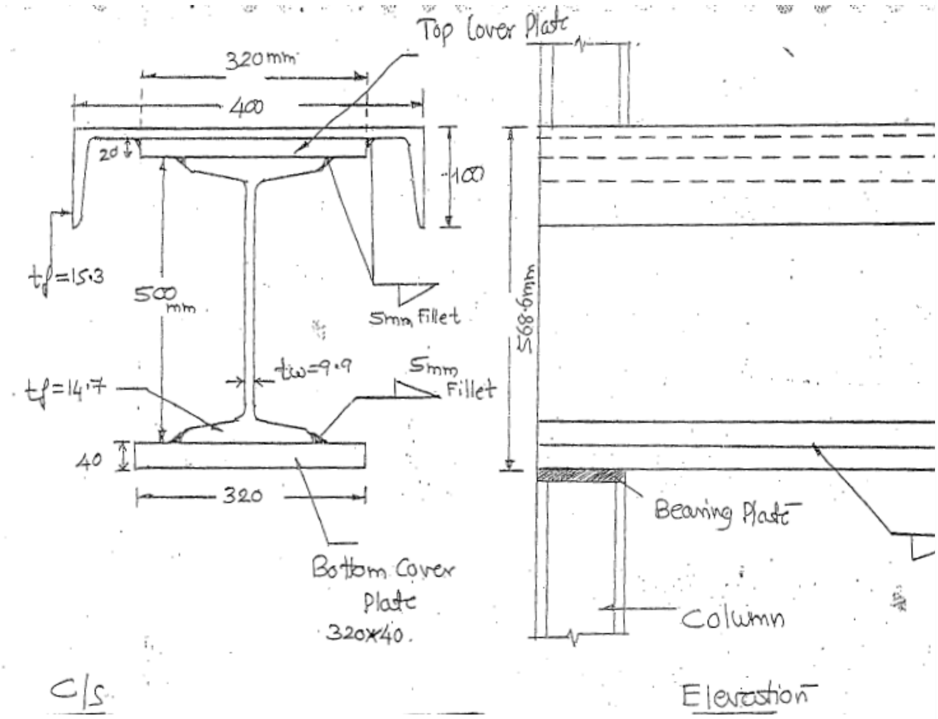
=0.745 N/mm …. (1)

(Formula for weld)

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

OR

2 (a) A R.C.C. retaining wall with counterforts is required to support earth to a height of 7m above the ground level. The trial pit taken at the site indicates that soil of bearing capacity 210kN/m^2 is available at a depth of 1.25m below the ground level. The weight of earth is 18kN/m^3 and angle of repose is 30°. The coefficient of friction between concrete and soil is 0.58. Use concrete M20 and steel grade Fe415. Design the retaining wall. [50] CO1 L2

Given Data:
fck= 20 N/mm^2, fy = 415N/mm^2, H = 7 m above G.L, Depth of footing below G.L. = 1.25 m, γ = 18 kN/m^3, μ = 0.58, SBC= 220 kN/m^2

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

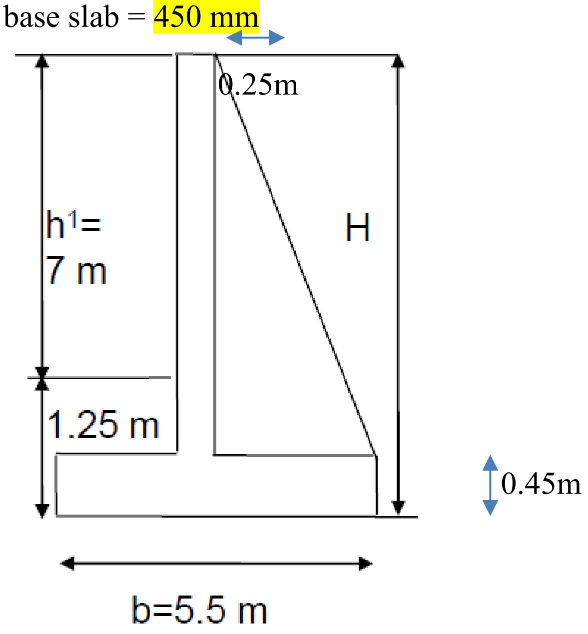
Coefficient of passive pressure = $k_p = \frac{1}{ka} = 3$

Taking depth of foundation as 1.25 m

The height of the wall above the base or Total height of retaining wall, H in metres = H = 7 + 1.25 = 8.25 m.

- **Proportioning of Wall Components – Stem, Heel, Toe and Counterforts**

1. Base width of retaining wall, b = 0.6 H to 0.7 H = 0.6 x 8.25 or 0.7 x 8.25 = (4.95 m to 5.78 m), Say b = 5.5 m
2. Width of Toe or Toe projection = b/4 = 5.5/4 =1.375 say 1.2 m or 1.3m
3. Assume thickness of vertical wall or stem = 250 mm (We are assuming constant thickness for stem slab)
4. Assume thickness of base slab = 450 mm



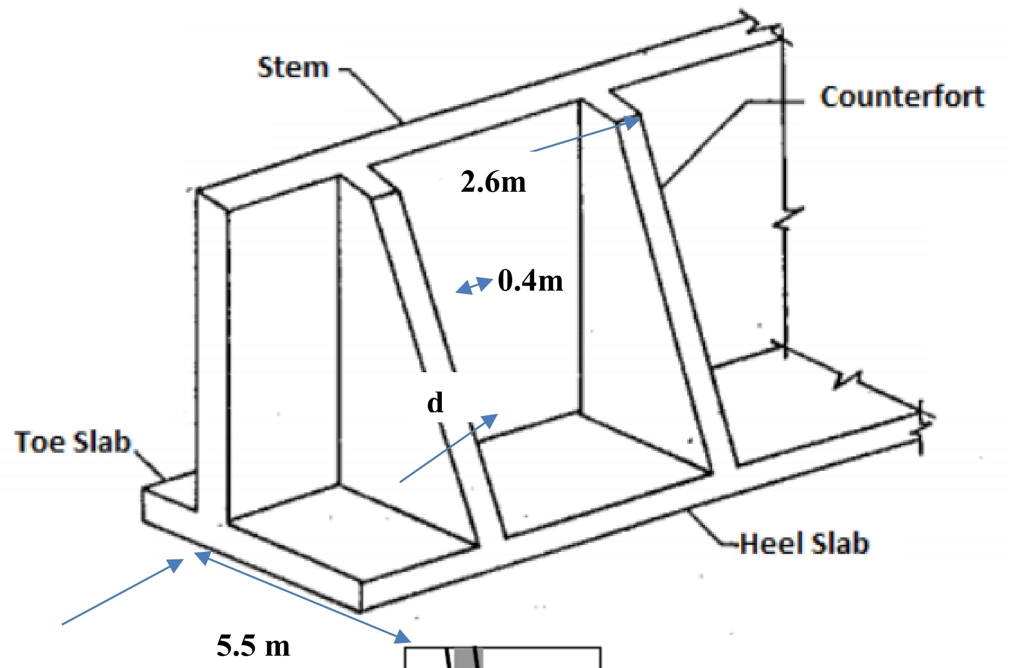
5. **Spacing of counterforts**

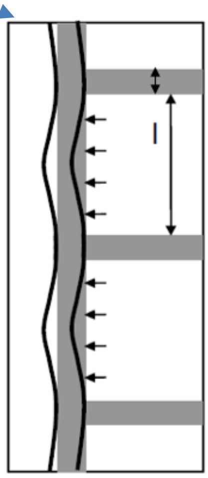
Clear spacing of counterforts, l = $3.5\left(\frac{H}{\gamma}\right)^{0.25} = 3.5\left(\frac{8.25}{18}\right)^{0.25}$ = 2.88 m

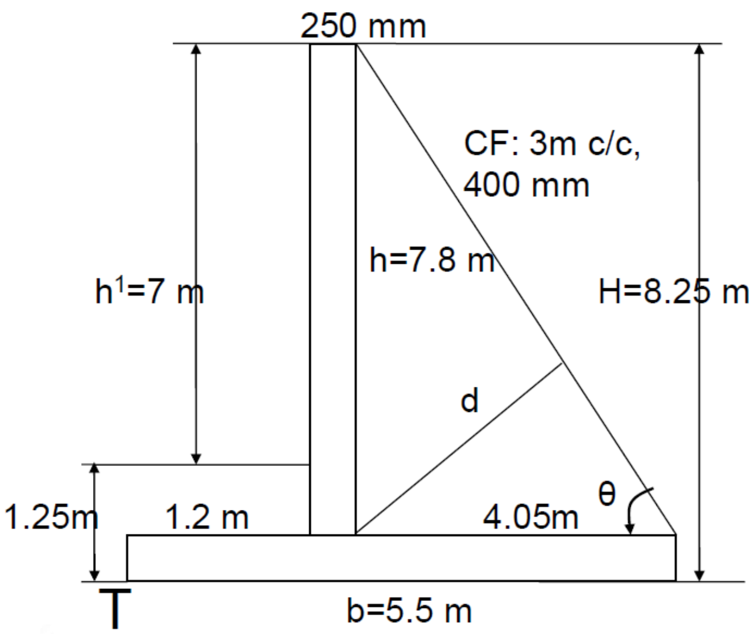
Assume width of counterfort = 400 mm,

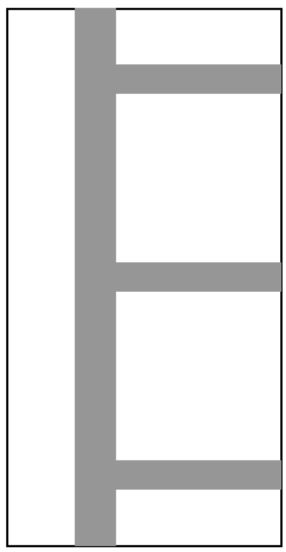
c/c spacing of counterforts = 2.88 + 0.40 = 3.28 m ≈ 3.00 m or 3.5m

So, clear spacing of counterforts becomes = 3.00 - 0.4 = 2.6 m

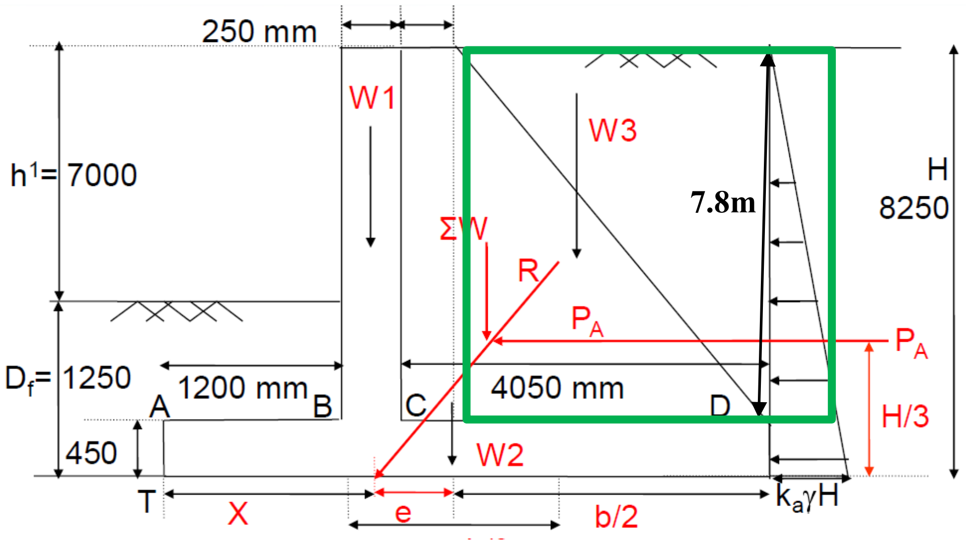








b. Check Stability of Wall



Calculations of Restoring moment – Weight of retaining wall and weight of earth fill retained on heel slab

Sr. No	Description of loads	Loads in kN	Dist. Of C G from T in m	Mome about T in k
1	Weight of stem W1	25x0.25x 1x7.8 = 48.75	1.2 + 0.25/2 =1.325	64.6
2	Weight of base slab W2	25x5.5x1 x0.45 = 61.88	5.5/2 =2.75	170.2
3	Weight of earth over heel slab W3	18x4.05x 1x7.8 = 568.62	1.20+0.25+4.05/2 = 3.475	1975.9
Total		ΣW = 679.25		ΣM =2

Calculations of overturning moment – Active earth pressure

Sr. No	Description of loads	Loads in kN	Dist. Of CG from T in m	Mome about T in kN
1	Horizontal earth pressure on stem slab	$\frac{1}{2}(k_a \times \gamma \times H)\, H =$ $= \frac{1}{2} \times \frac{1}{3} \times 18 \times 8.25 \times 8.25 =$ **204.19**	8.25/3	561.52

- **Check for overturning**

Factor of safety against overturning

FOS = $\frac{2210.71}{561.52}$ = 3.94 > **1.55**, Hence it is safe against overturning.

- **Check for sliding FOS =** μ∑W/P_H ≥ 1.55

Total horizontal force tending to slide the wall =Ph=**204.19kN**
Resisting force =μ ΣW = 0.58 x 679.25 = 393.97kN

Factor of safety against sliding = $\frac{\Sigma W\ \mu}{P_h} = \frac{393.97}{204.19}$ = 1.93>1.55 Hence it is safe against sliding.

- **Check for pressure distribution at base**

Let **X** be the distance of Resultant R from toe(T), $= \frac{\text{Net Moment}}{\Sigma W} = \frac{2210.71 - 561.52}{679.25}$ =2.43m

Eccentricity= **e = b/2 - X** = 5.5/2 - 2.43 = 0.32 < b/6 (0.91m)

Whole base is under compression.

Maximum pressure at toe

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

$= 166.61 kN/m^2 < SBC = 220 kN/m^2$

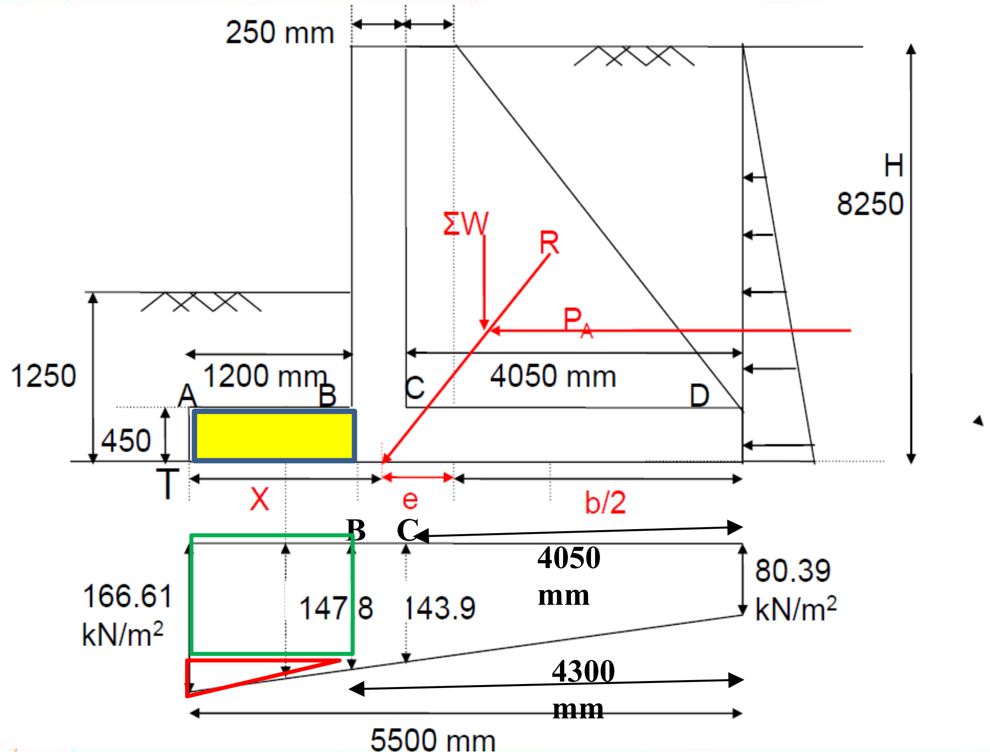
Minimum pressure at heel

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

$= 80.39 kN/m^2 < SBC = 220 kN/m^2$

By interpolation, Intensity of pressure at junction of stem with toe i.e. under B $= p_B = 80.39 + (166.61 - 80.39) \times 4.3/5.5 =$ **147.8kN**/m^2

By interpolation, Intensity of pressure at junction of stem with heel i.e. under C $= p_C = 80.39 + (166.61 - 80.39) \times 4.05/5.5 =$ **143.9 kN**/m^2



b) Design of Toe slab

Sr. No	**Description of loads**	**Loads in kN**	**Dist. Of C G. from B in m**	**Moment B in kN-**
1	Weight of Toe slab	25x 1.2 x 0.45 =	1.2/2	8.1
2	Weight due to upward soil pressure	-147.8 x 1.2 = -177.3	1.2/2	-106.4
3	Weight due to upward pressure	- ½ x (166.61- 147.8) x 1.2	2/3 x 1.2	-9.02

Total				ΣM = - 1(
Factored Moment Mu				= -160.9 l

- **To find steel**
  b = 1000mm, d = 400 mm, fck = 20N/mm², fy = 415N/mm², Mu = 160.9 x 10^6 Nmm

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

Ast = 1188.22mm²

Take 16 mm diameter bars as main bars, Spacing s $= \frac{1000\ \times\frac{\pi}{4}\times 16^2}{1188.22} = 170$ mm < 300 mm and 3 d.

Main bars - Provide **16 mm** Φ dia @ 170 mm c/c .

Distribution steel =

0.12 % x b x D = 0.12 x 1000 x 450/100 = 540 mm²

Let's provide 12 **mm** Φ diameter bars, Spacing s $= \frac{1000\ \times\frac{\pi}{4}\times 12^2}{540} = 210$ mm < 450mm and 5 d

Distribution bars - Provide **12 mm** Φ dia @ 210 mm c/c .

- **Development length = 47 x diameter of main bar = 47 x 16 = 750 mm**

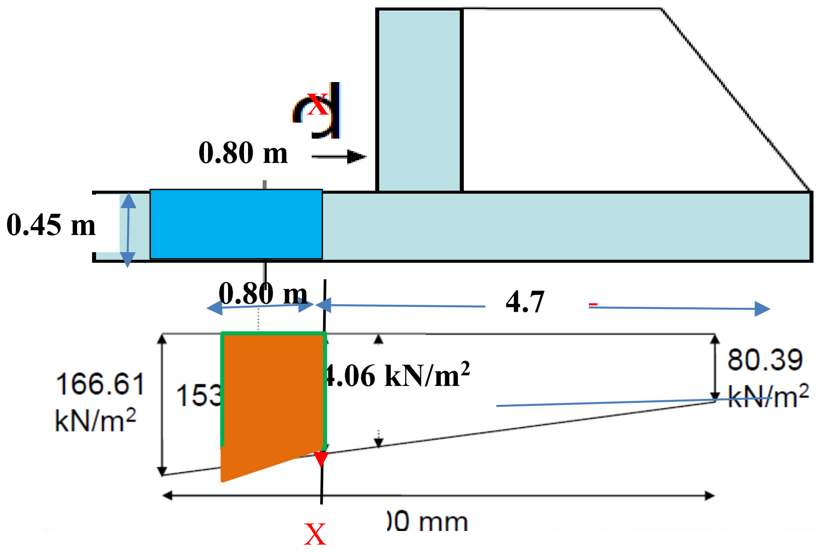
- **Check for Shear**

  Locate a critical section XX at a distance 'd' from junction of toe slab

  Critical section for shear: At distance d (= 400 mm) from the **junction** of the toe

  $\frac{166.61-80.39}{5.5} = \frac{y}{4.7}$, y = 73.67

  Pressure at section XX = 73.67 + 80.39 = 154.06 kN/m²



Net vertical shear = - (166.61 + 154.06) x 0.80/2 + (25 x 0.45 x 0.80) = 119.28kN

Net ultimate shear = $V_{u.max}$ = 1.5 x 119.28 = 178.9 kN
ζv = 178.9 x 1000/1000 x 400 = 0.447 MPa
pt = 100 x 1188.22/ (1000 x 400) = 0.29 %
ζc = 0.39 N/mm^2 ζc < ζv
Hence it is not safe in shear. Provide stirrups or increase percentage of steel, pt = 0.5%.

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_c bd$ The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Shear carried by steel, Vus = $V_u - \tau_c bd$ = 178.9 x 1000 – 0.39 x 1000 x 400 = 22.9kN
Using #8 mm 2-legged stirrups, Asv = 2 x π x 8^2 / 4 = 100.53 mm^2

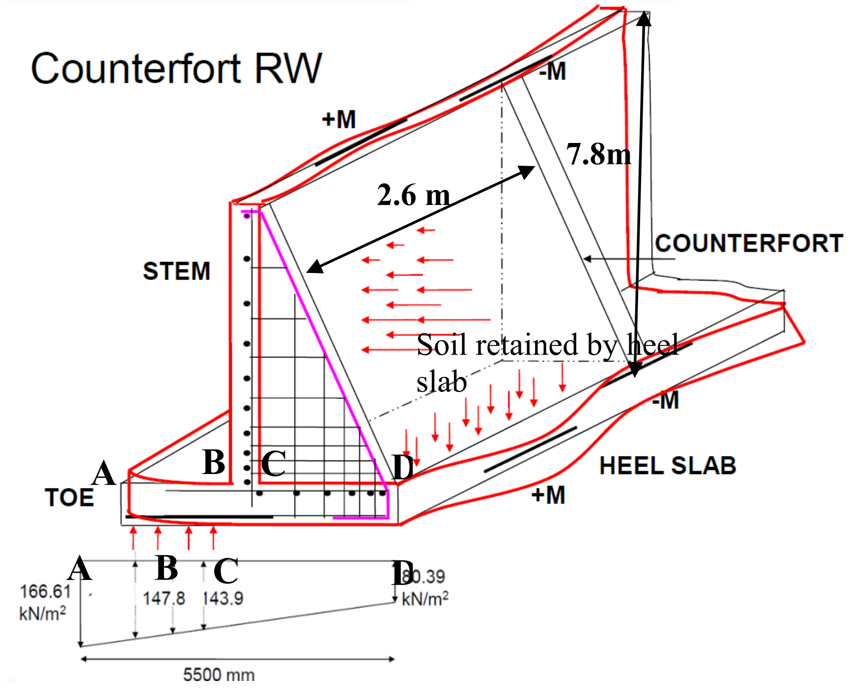
$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Spacing of vertical stirrups, s_v = 633.99 mm < 0.75 x 400 < 300 mm
Provide #8 mm 2-legged stirrups at 290 mm c/c.

- **Design of Heel Slab**

Heel slab is a continuous slab. Consider 1 m wide strip near the outer edge **D**
The forces acting near the edge **D** are

1. Downward wt. of soil retained on heel slab =18 x 7.8 x 1 = 140.4 kN/m
2. Downward self wt. of heel slab = 25 x 0.45 x 1= 11.25 kN/m



Upward soil pressure at D = -80.39 kN/m^2

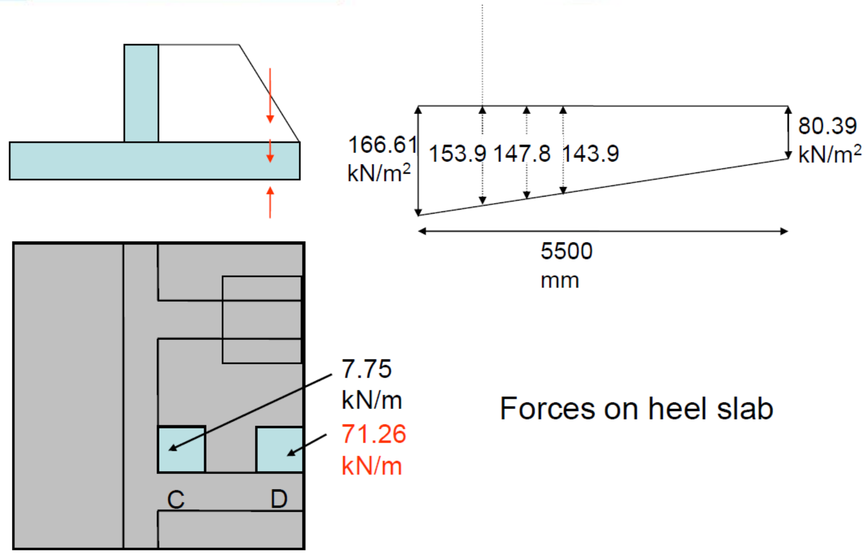
Upward wt due to soil pressure = - 80.39 x 1= - 80.39 kN/m

Net force at D , p = 140.4 + 11.25 - 80.39 = 71.26 kN/m

Also Net force at C, p = 140.4 + 11.25 - 143.9 = 7.75 kN/m

Factored Negative Bending Moment for heel at junction of counterfort (D)

Mu = 1.5 x pl^2/12 = 1.5 x 71.26 x 2.6^2/12 = 60.2 kN-m (At the junction of Counter Fort)



Forces on heel slab

- **To find steel**
  b = 1000mm, d = 400 mm, fck = 20N/mm^2, fy = 415N/mm^2, Mu = 60.2 x 10^6 Nmm

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

Find Ast = 426 mm^2 , Ast$_{min}$ = 0.12 x 1000 x 450/100 = 540 mm^2

426 mm^2 < 540 mm^2 Provide Ast = 540 mm^2

Provide # 12 mm @ 210 mm c/c < 300 mm

Check for shear (Heel slab)

Shear Force at D = 71.26 x 2.6/2 =

Factored shear = V_u= 1.5 x Shear Force = 139 kN

pt = 100 x 540 / (1000 x 400) = 0.13 and M20 concrete, ζ_c = 0.28 N/mm^2

$\zeta_v = V_{umax}/bd$ = 139 x 1000 /(1000 x 400) = 0.35 N/mm^2

$\zeta_c < \zeta_v$, Unsafe, hence shear steel is needed.

Using #8 mm 2-legged stirrups,

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_c bd$ The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Shear carried by steel, Vus = $V_u - \tau_c bd$ = 139 x 1000 – 0.28 x 1000 x 400 =27 kN

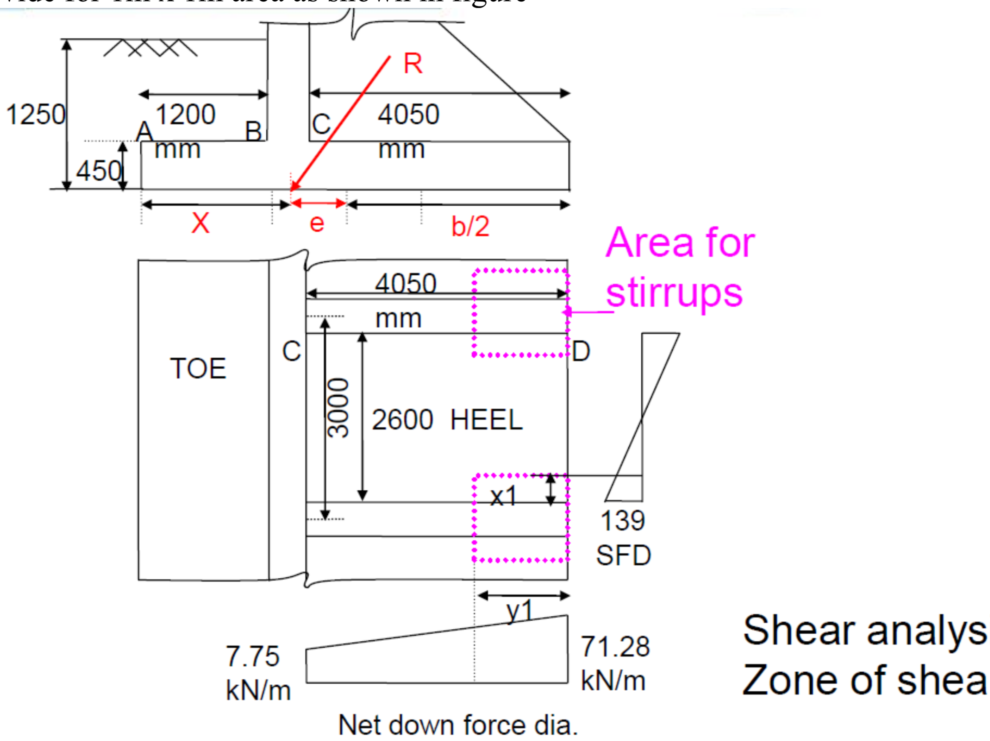
Using #8 mm 2-legged stirrups, Asv = 2 x π x 8^2 / 4 = 100.53 mm^2

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Spacing s_v = 538 mm < 0.75 x 400 = 300 mm

Provide #8 mm 2-legged stirrups at 290 mm c/c.

Provide for 1m x 1m area as shown in figure



- **Area of steel for +ve moment (Heel slab)**

Maximum +ve ultimate moment at mid span of heel slab = 1.5 x 71.26 x 2.6^2/16

= + 45.15 kN-m

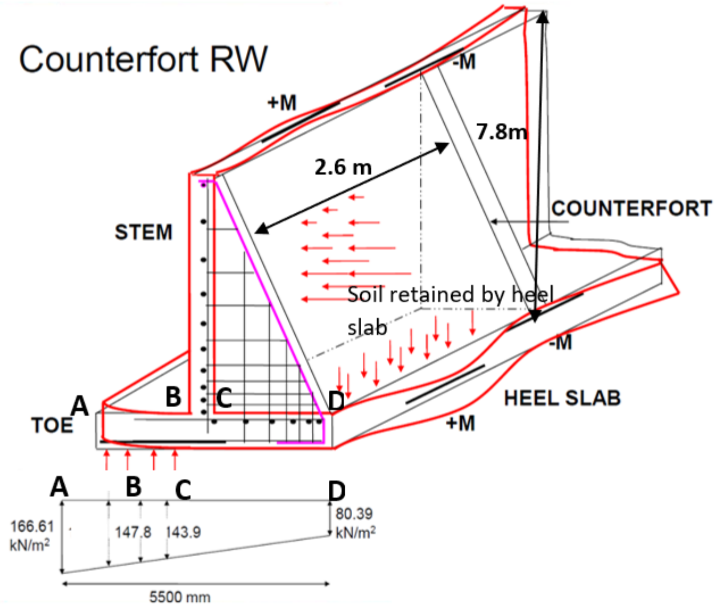
Since 45.15 kNm < 60. 2 kNm, Provide minimum steel.

Ast, min = 540 mm^2

Provide Main bars # 12 mm bars at 200 mm c/c < 300 mm

Also provide distribution steel # 12 mm at 200 mm c/c < 300 mm

- **Design of Stem (Vertical Slab)**



Consider stem slab as continuous slab spanning between the counterforts and subjected to earth pressure.

The intensity of earth pressure = p_a= ka x γ x h = $\frac{1}{3}$ x 18 x 7.8= 46.8 kN/m^2

For 1m , it will be 46.8 kN/m^2 x 1 m = 46.8 kN

Maximum -ve ultimate moment near ends of counterforts,

Mu= 1.5 x $p_a l^2/12$ = 1.5 x 46.8 x 2.6^2/12 = 39.54 kN.m.

Find the required effective depth or thickness of the stem slab

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

Mu, lim =39.54 x 10^6 N mm, xu, max/d = 0.48, b = 1000, fck =20N/mm^2

After calculations find 'd', d = 119.70 mm ≈ 120 mm

However, provide total depth or thickness, D = 250 mm. Hence safe.

- To find steel:
- Effective depth, d = 250 – 50 = 200 mm , (effective cover = 50 mm)
- **b = 1000mm, d = 200 mm, fck = 20N/mm^2, fy = 415N/mm^2, Mu = 39.54x 10^6 Nmm**
- Ast = 582.1 mm^2 , Ast min = 0.0012 x 1000 x 250 =300 mm^2
- Ast provided > Ast,min . Hence safe
- Provide #12 mm @ 210 mm c/c

As the earth pressure decreases towards the top, the spacing of the bars is increased

.

Max. Ultimate shear = Vu_{max} = 1.5 x 46.8 x 2.6/2 = 91.26 kN

For pt = 100 x Ast / (1000 x 200) = 0.29 % and M20 concrete ζc = 0.38 N/mm^2

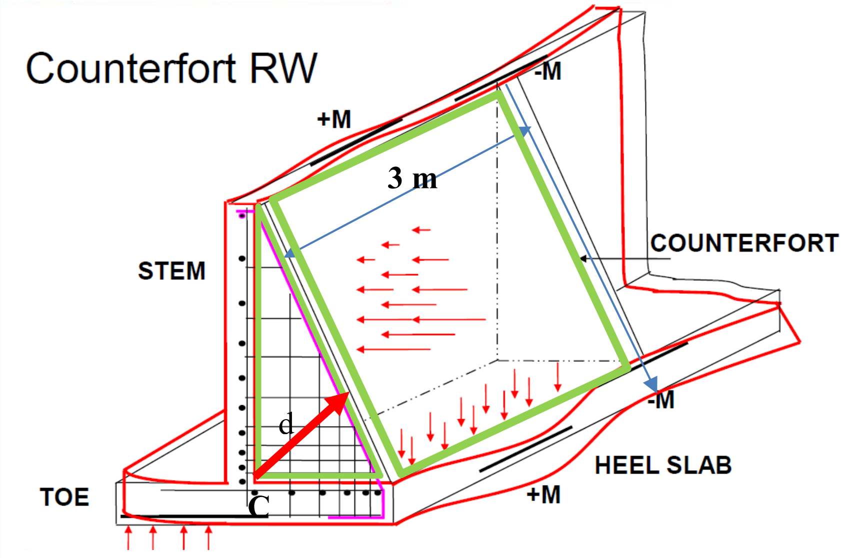
ζv = V_{umax}/bd = 91.26 x1000/(1000 x 200) = 0.45 N/mm^2

ζv > ζc , It is not safe in shear. Either increase the pt = 0.5%, so that ζc = 0.48 N/mm^2 or Provide shear reinforcement in the form of stirrups.

- **Design of Counterfort**

The total horizontal earth pressure acting on the counterfort = $\frac{1}{2}$ x k_a x γ x h^2 x c/c distance between counterfort

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 7.8^2 \times 3 = 547.56\text{kN}$



B.M. at the base at C= **547.56 x 7.8/3** = **1423.65kN.m**.

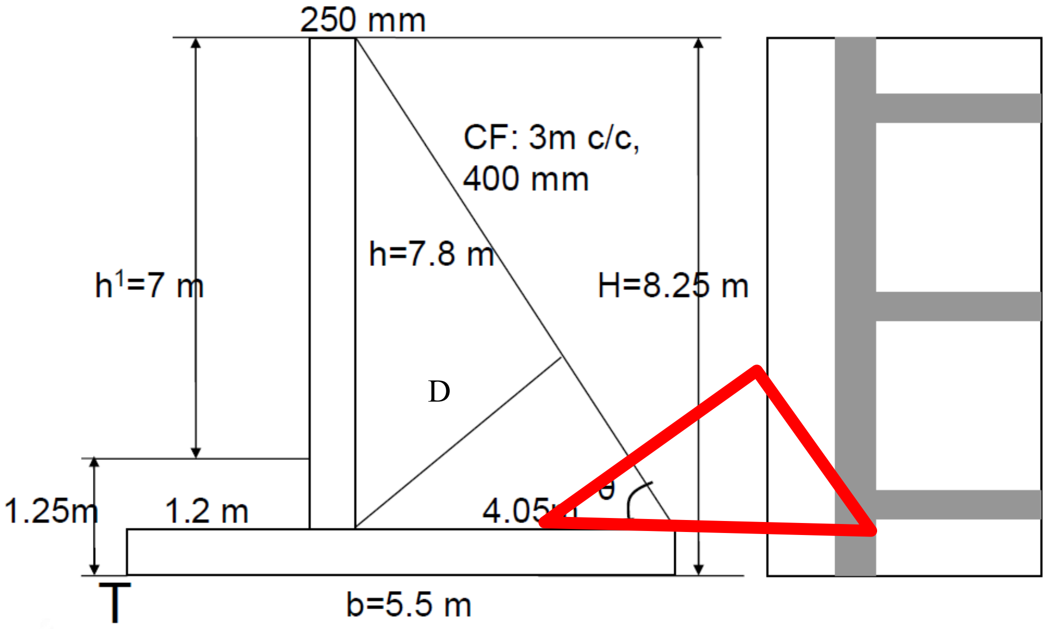
Ultimate moment = Mu= 1.5 x 1423.65 = 2135.48 kN.m.

Counterfort acts as a T-beam, lets find the effective depth

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

Mu,lim = 2135.48 x 10^6, xu, max/d = 0.48, b = 400, fck =20N/mm^2

Find 'd' = 1390 mm



The effective depth is taken at right angle to the sloping face of counterfort

tan θ = 7.8/4.05 = 1.93,

θ = $\tan^{-1}$(1.93) = 62.5° ,

From the geometry

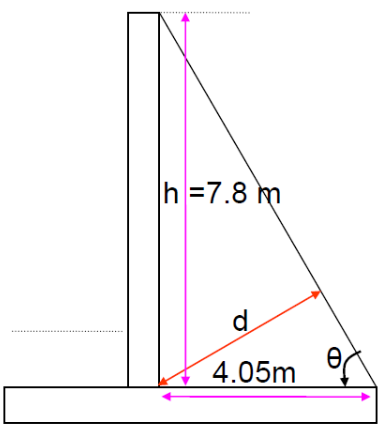
D /4.05 = sin 62.5 , D = 3.6 m = 3600 mm, d = 3600 – 50 = 3550 mm > 1390 mm .

Hence depth of counterfort provided is safe.

- **To find steel**
  b = 400mm, d = 3550 mm, fck = 20N/mm², fy = 415N/mm², Mu = 2135.48 **x 10^6 Nmm**

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

Ast = 1708 mm²



- **Check for minimum steel – IS 456 2000 CL 26.5.1.1**

26.5.1.1 ***Tension reinforcement***

a) ***Minimum reinforcement***—The minimum area of tension reinforcement shall be not less than that given by the following:

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam,

d = effective depth, and

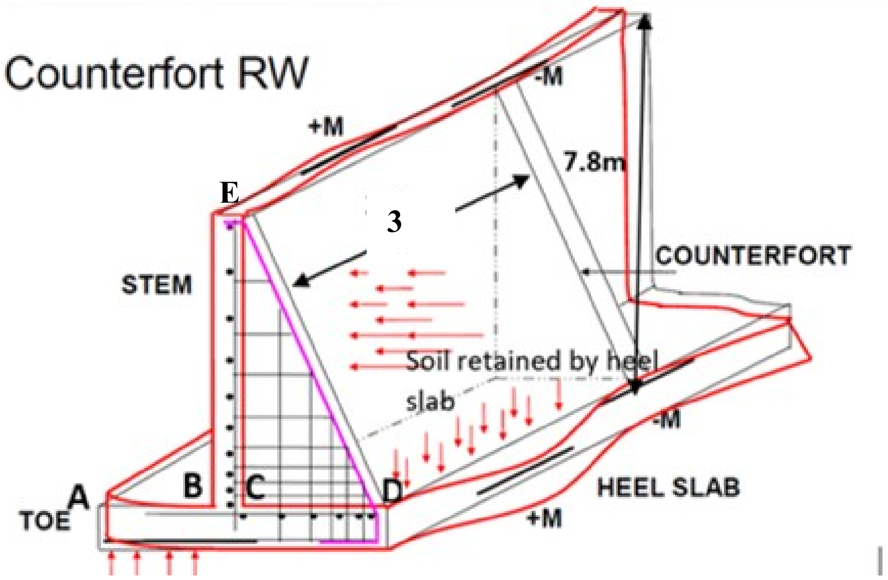
f_y = characteristic strength of reinforcement in N/mm².

- As per IS 456, Ast.min = 0.85 bd/fy = 0.85 x 400 x 3550/415 = 2908.4 mm^2 (T Beam section)
- Use 22 mm diameter bars, calculate no of bars = 2908.4 / (π x 22^2 /4) = 7.65 ≈ 8
- Provide 2 layers of bars ie 4 # 22 mm, 4 # 22 mm

- Development length = L_d = 47 x 22 = 1030 mm = **1.03 m**
- The half of the reinforcement can be curtailed is equal to √H = √7.8 = 2.79 m - **1.03 m** = 1.7 m from top, Bars are curtailed.

- **Design of Horizontal Ties or Horizontal stirrups (H S)**

The counter forts are subjected to tensile stresses along the outer face ED of the counter forts



The tension exerted on counterfort for 1 m height at base due to horizontal earth pressure, T

$T = k_a \times \gamma \times h \times$ c/c distance between counterfort $\times 1\ m$

= 1/3 x 18 x 7.8 x 3 x 1 = 140.4 kN

Area of steel required to resist the tension = Ast = $\frac{1.5 \times T}{0.87 \times f_y}$

1.5 x 140.4 x 10^3/ (0.87 x 415) = 583 mm^2

Using # 8 mm 2-legged stirrups, Ast = 100 mm^2 , spacing , s =

spacing = 1000 x 100/583 = 170 mm c/c.

Provide horizontal stirrups (H S) 2-legged # 8 mm at 170 mm c/c near bottom.

Since the horizontal pressure decreases with height, the spacing of stirrups can be increased from 170 mm c/c to 450 mm c/c towards the top.

- **Design of Vertical Ties or Vertical stirrups (V S)**

The maximum vertical tension exerted at the end of heel slab due to net downward force at D = 71.26 kN/m.

Total tension at D = 71.26 x c/c distance between counterforts = 71.26 x 3 = 213.78 kN

Area of steel required to resist the vertical tension = Ast = $\frac{1.5 \times T}{0.87 \times f_y}$=

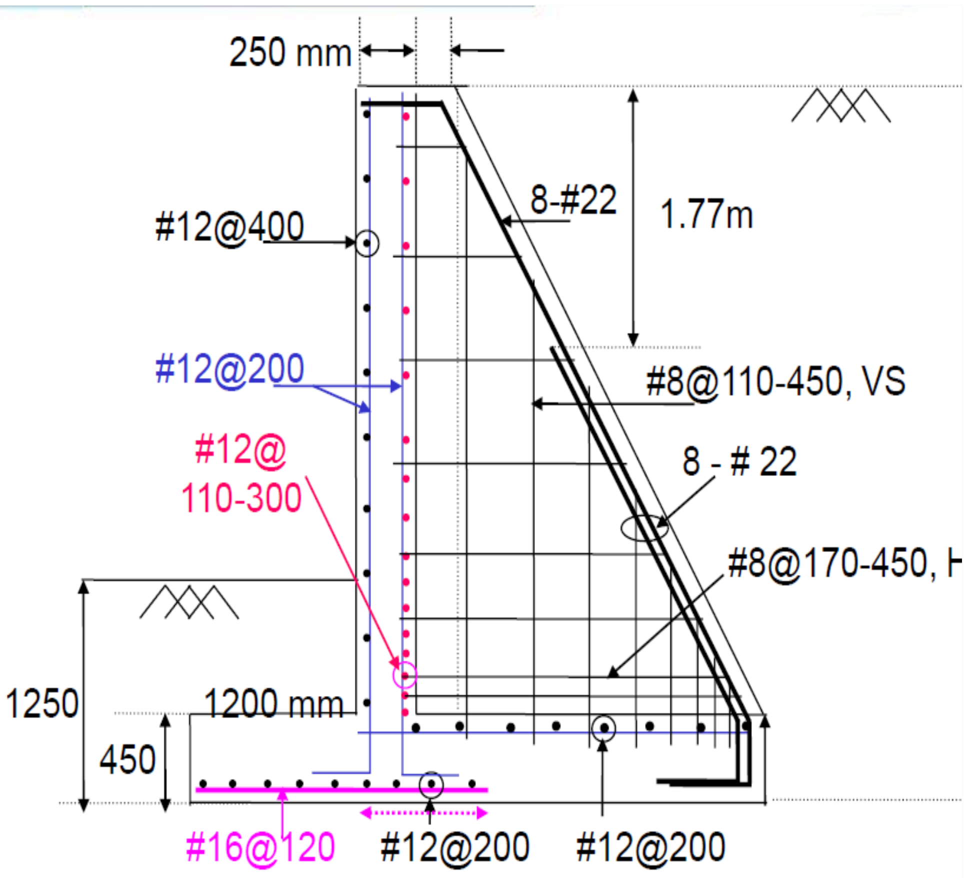
Required Ast = 1.5 x 213.78 x 10^3/ (0.87 x 415) = 888 mm^2

Using # 8 mm 2-legged stirrups, Ast = 100 mm^2

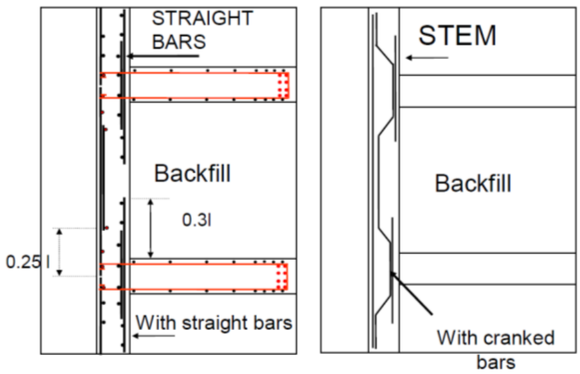
Spacing = 1000 x 100/888 = 110 mm c/c.

Provide vertical stirrups (V S) # 8 mm 2-legged stirrups at 110 mm c/c.

Increase the spacing of vertical stirrups from 110 mm c/c to 450 mm c/c towards the end C.



Cross section through counterforts



Section through stem at the junction of Base slab.

CI CCI