CHAPTER FIVE

RESULTS AND DISCUSSION

Chapter five
Results and discussion

This chapter therefore considers the design of retaining wall structures, in particular the calculations and practical considerations necessary for the control of cracking.

All these aspects of design are outlined and illustrated with a detailed design example of a cantilever type retaining wall.

5.1 Laboratory Results:

Liquid Limit = 69

Plastic Limit = 25.8

Plasticity Index =40

Referring to table (3.1) Relation between Plasticity Index and Potential Swell this soil is high Potential Swell They found that using the method of removal and replacement by non-swelling soils significantly reduced the swelling characteristics.

5.2 Design Procedure:

STEP (1):

Determination of dimension of the tank

Quantity =per demand x Population

Every personal uses approximately 140-150 liters of water a day

Number of residents = 600

Daily water requirement = 150*600= 90000 liter per day =90 m^3

Assuming length is equal to the three times of breadth.

Area of the tank = $\frac{Q}{H}$

Assuming H = 3m

Area of the tank =$\frac{90}{3}$=30 m^2

B =√ (area of tank / 3)

B =√ (30/3) =3.16 =~ 3.20 m

L=3B, L =3*3.20= 9.60m

Assuming Free board= 500mm

Density γ=1800 kg/m3=17.6 kN/m^3

Angle of internal friction ϕ =30°

Angle of internal friction of foundation soil ϕ =33°

The assumed surcharge load = 50 KN/m^2

STEP (2):

Estimation of primary dimensions of the wall,

The dimensions of the retaining wall will be assumed as follow refer to figure4.3

a. The width of the wall base

B= 0.4H to 0.7H = 0.4 * 4 to 0.7 * 4

B = 1.6m to 2.569m, the width of the base will be assumed = 3.20m

b. The thickness of the stem at the top

t=H/12 to H/10

t=0.33 to 0.4m take t= 0.4m

<u>Check of soil stresses</u>

(In case of during maintenance)

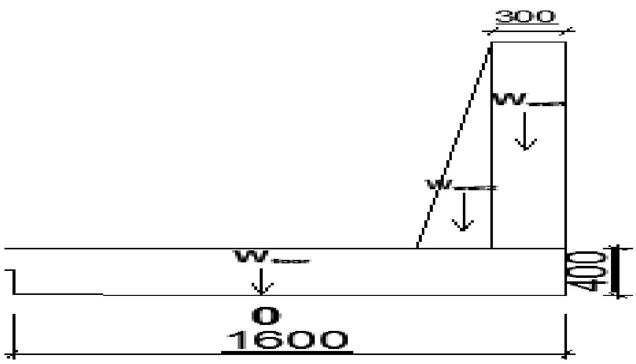

Fig. 5.1 loads on the wall

1. The loads:

Calculate total weight including walls

$W_{wall\ 1}$ = 25*(0.3*4) =27KN

$W_{wall\ 2}$= 25*(0.5*0.1*3. 6) =4.5 KN

W_{floor}= 25*(0.4*1.60) = 16KN

The sum of these weights (N):

N = 27+4.5+16=47.50 KN

M@0 = (27*0.55) + (4.5*0.434) + (16*0 .8) = 29.60 KN.m

A=1.6m^2

Z=0.43m^3

$$\mathrm{p_{max}} = \frac{\sum 47.50}{1.6} + \frac{\sum 29.60}{0.43}$$

$$= 98.52 \ < (\text{soil bearing capcity} = 180, \text{unsafe})$$

$$\mathrm{p_{min}} = \frac{\sum 47.50}{1.6} - \frac{\sum 29.60}{0.43} = -39.15(\text{tension, unsafe})$$

Use toe with length= 1.73 m

(In case of during maintenance)

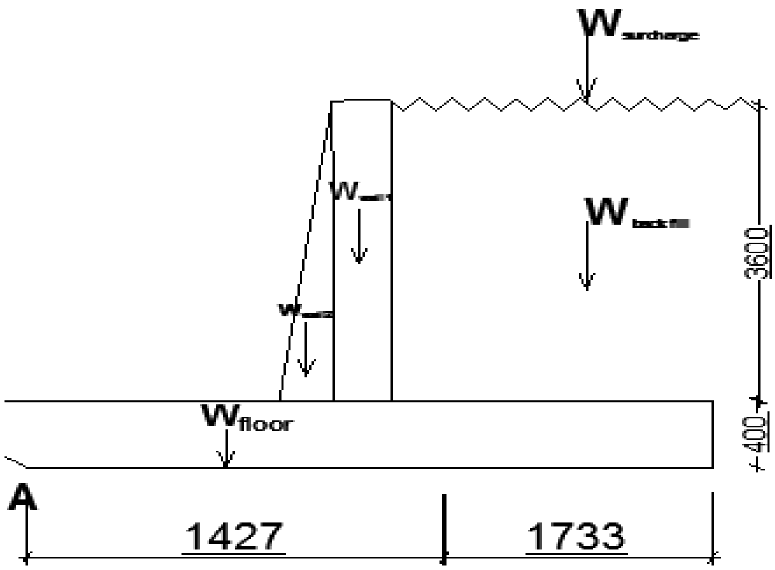

Fig. 5.2 loads on wall with toe

The loads and earth pressures acting on the wall:

II. The loads:

$W_{wall\ 1}$ = 25*(0.3*4) =27KN

$W_{wall\ 2}$= 25*(0.5*0.1*3. 6) =4.5 KN

W_{floor}= 25*(0.4*3.20) = 32KN

$W_{back\ fill}$= 17.6*3.6*1.73 = 109.61KN

$W_{surcharge}$= 50*1.73= 86.65KN

The sum of these weights:

$\sum W = \sum R_V$ = Σ 259.76 KN

2. The active earth pressures:

$$K_a = \frac{1-\sin \emptyset}{1+\sin \theta} = \frac{1-\sin 30}{1+\sin 30} = 0.33$$

The horizontal pressure at depth y from the top of the surcharge is:

$$17.6\mathrm{y}\frac{(1-0.5)}{(1+0.5)} = 5.87\mathrm{y}\ \mathrm{kN/m^2}$$

The horizontal pressure at the base is

5.87×4=23. 48kN/m^2

(i) Maximum soil pressure the base properties are:

R_V*x=M_R-M_0

$$\overline{\mathrm{X}} = \frac{\sum \mathrm{M_R} - \sum \mathrm{M_0}}{\sum \mathrm{RV}} = \frac{548.81-193.44}{259.76} = 1.37\mathrm{m}$$

$$e = \frac{b}{2} - \bar{x} = 1.28-1.37= 0.23\ \mathrm{m}$$

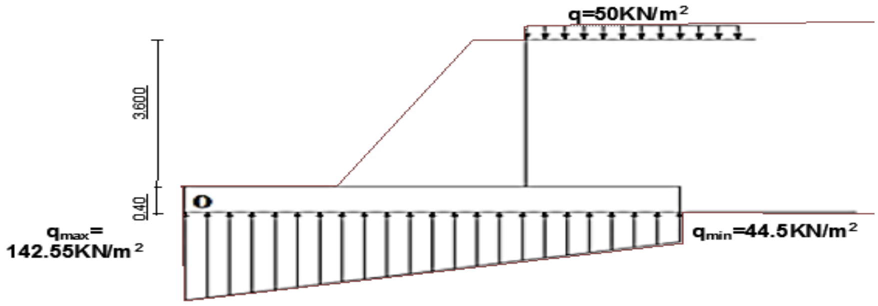

Fig. 5.3 the pressure distribution under the wall

$\bar{B} = B - 2e = 3.2 - 2 * 0.23 = 2.74$

Area A=2.74 m^2

The maximum soil pressure at a calculated for service load is:

$$= \frac{259.76}{2.74}\left(1 + \frac{6 * 0.23}{2.74}\right) = 142.55\mathrm{KN/m^2}$$

The minimum soil pressure at C calculated for service load is

$$\frac{259.76}{2.74}\left(1 - \frac{6 * 0.23}{2.74}\right) = 44.55\mathrm{kN/m^2}$$

∂_1=ka * q =.33*50 = 16.5

E_{a1} = 16.5*4= 66 kN

∂_2 =ka *(q+H* γ) =0.33*(50 +17.6*4) = 39.73

E_{a2} = 0.5*(∂_2 –∂_1) *4= 46.2 KN

(ii) Stability against overturning: The stabilizing moment about A of the wall for a partial safety factor γf=1.0 is

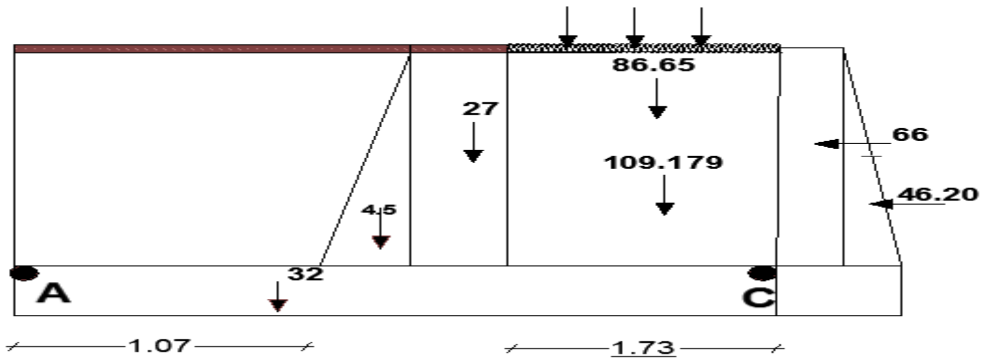

Fig. 5.4 Loads and earth pressures acting on the wall

Table 5.1 The resisting moments

load (KN)	Distance from A (m)	Moment about A(KN.m)
$W_{wall\ 1}$ =27	1.317	35.559
$W_{wall\ 2}$ =4.5	1.13	5.085
W_{floor} =32	1.6	51.2
$W_{back\ fill}$ = 109.179	2.33	254.78
$W_{surcharge}$ = 86.65	2.33	202.19
Total = 259.76		548.81

Table 5. 2 The overturning moments

Horizontal load (kN)	Distance from A(m)	Moment about A (kN. m)
Active pressure E_{a1}=66	2	132
E_{a2}=46.2	1.33	61.446
Total = 112.20		193.44

30.26+ (211.97*1.28) = 301.85 KN. m

The overturning moment for a partial safety factor γf=1.5 is:

$$\frac{M_R}{M_0} > 1.5$$

$\frac{548.81}{193.44} = 2.8 > 1.5$ The wall is safe against overturning.

(iii) Resistance to sliding: The forces resisting sliding are the friction under the base and the passive resistance for the top of the base:

$$\boldsymbol{F}_{\boldsymbol{S_0}} = \frac{\sum R_V}{\sum R_H}\mu$$

$$\mu = \tan \emptyset = \tan 33 = 0.65$$

$=\frac{259.76*0.65}{112.20}=1.5$

The resistance to sliding is satisfactory.

(iv) Over all comment the wall section is satisfactory.

The maximum soil pressure under the base controls the design.

Structural design

5.3 Design retaining wall:

Reference Bs-8110-1997	Calculation	Out put
	The structural design is made for ultimate loads. The partial safety factor for each pressure and surcharge is γf=1.4. The pressure at the top of the wall is ∂_1=1.4×16.50=23.10 kN/m^2 ∂_2=ka *(q+H* γ) =0.333*(50 +17.6*3.60) = 39.73 ∂_2-∂_1= 39.73- 23.10 = 16.63 16.63*1.4 = 23. 30 Shear= (23.10×3.6) + (0.5×3.6×23.30) =83.16+41.94=125.10 KN Moment= (83.16×0.5×3.6) + (41.94×$\frac{3.60}{3}$) =200 KN. m The cover is 50 mm; assume 16 mm diameter bars. Then d= 400-50-8 = 342 mm $\frac{M}{(b*d^2)} = \frac{200*10^6}{(1000*342^2)} = 1.70$ $\frac{100A_s}{b*d} = 0.40$ (figure 4: 13) 100 A_s / (342*1000) =0.40 =0.40*342*1000/100 As =1368 mm^2/m	ϕ 16 mm @ 140 C/C $\approx 1407 mm^2/m$

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Reference Bs-8110-1997	**Calculation**	**Out put**
                  clause 3:4:5:2 clause 3.4.5.6	The depth y1 from the top where the 16 mm diameter bars can be reduced to a diameter of 12 mm. As= 807 mm^2 =100×807/ (1000×342) =0.23 $\frac{\mathrm{M}}{\mathrm{b} * d^2} = 1.25$ M= $\frac{1.25*1000*342^2}{10^6}$= 146.205 KN.m Mu= M * 1.4 = 1.4*146.205=204.687 kN.m The depth y1 is given by the equation $204.687 = \frac{23.1{y_1}^2}{2} + \frac{1.4 * 5.87\mathrm{y}^3{}_1}{6}$ Or y_1^3+8.43y_1^2-149.5 =0 Solve to give y_1=3.53m. The shear stress at the base of the wall is $\mathrm{V} = \frac{v}{\mathrm{b} * \mathrm{d}} = \frac{125.1 * 1000}{1000 * 342} = 0.36\mathrm{N/mm^2}$ The design shear stress is $\mathrm{V}_C = \frac{0.79\left(\frac{100\mathrm{A_S}}{\mathrm{b*d}}\right)^{\frac{1}{3}}\left(\frac{400}{342}\right)^{\frac{1}{4}}\left(\frac{\mathrm{F_{cu}}}{25}\right)^{\frac{1}{3}}}{1.25}$ V_c = 0.673 N/mm^2 The shear stress is satisfactory	

Continue

Reference Bs-8110-1997	Calculation	Out put
clause 3:4:4:4	For control of cracking the bar spacing must not exceed 3 times the effective depth. The spacing at the bars in the wall is 140 mm. This is less than the 160 mm clear spacing given in Table 3.30 of the code for crack control. For distribution steel provide the minimum area of 0.13% from Table 3.27 of the code A=0.0013×1000×400=$\underline{520\ mm^2/m}$ Wall reinforcement (outer side) the pressure at the base of the wall is the pressure at the top of the wall is Pw = γ_W*H =10*3.60=36.00 kN/m^2 Pw =1.4*36= 50.40 Shear=0.5×3.6×50.40=90.72kN Moment= (90.72×3.6/3) = 108.86kN. M $\frac{M}{b*d^2} = \frac{108.86*10^6}{1000*342^2} = 0.93$ $\frac{100A_s}{1000*342} = 0.4$ $\frac{0.4*342*1000}{100}$ $\underline{\underline{As = 1368\ mm^2/m}}$	ϕ 12 @ 200mm C/C $\approx 565 mm^2/m$ horizontally on the outer face ϕ 16 mm @ 140 C/C $\approx 1407 mm^2/m$

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Reference Bs-8110-1997	Calculation	Out put
	iii) Inner footing Referring to Fig. 5.4 the shear and moment at the face of the wall are as follows: Shear = 1.4*{(139.21*1.07) + (3.02*1.07/2) _ (32*1.07/3.20)} = 1.4(148.95+1.6-10.7) = 195.79 KN $\text{Moment} = 1.4 * [(148.95 - 10.7) * 0.535$ $-(6 * 1.07 * 2/3)$ =1.4(73.96-4.28) =97.55 kN.m $\frac{\text{M}}{\text{b}*d^2} = \frac{97.55*10^6}{1000*342^2} = 0834.$ $\frac{100\text{A}_\text{s}}{\text{b} * \text{d}} = 0.25$ $A_{S=}\frac{0.25*1000*342}{100}$ =805 mm²/m Shear stress $=\frac{195.79*10^3}{1000*342}$ = 0.57N/mm² This is satisfactory. (iv) Outer footing Referring to Fig. 5.4 the shear and moment at the face of the wall are as follows: Shear= 1.4*{(109.6+86.65) + (32*1.73)/3.2 _ (8.25*1.73)_ (0.5*31.38*1.73} = 1.4(109.6+86.65+17.3-14.72-17.143) = 199.855 KN Moment = 1.4 *{(0.865*198.83) - (17.143*0.4)} =231.183 kN.M	ϕ 12 @ 125 mm C/C $\approx 904\text{mm}^2/\text{m}$

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Reference **Bs-8110-1997**	**Calculation**	**Out put**
	$\frac{\mathrm{M}}{\mathrm{b} * d^2} = \frac{231.18 * 10^6}{1000 * 342^2} = 2.74$ $\frac{100\mathrm{A_s}}{\mathrm{b} * \mathrm{d}} = 0.55$ $A_{S}=\frac{0.55*1000*342}{100}$ =<u>1881 mm^2/m</u> Shear stress $=\frac{199.85*10^3}{1000*342}$=0.58 N/mm^2 This is satisfactory. **Crack width** $\mathrm{a_c}=\frac{E_S}{E_C} = \frac{200}{28} = 7.14$ $a_{ce} = a_c * \frac{A_s}{b*d} = 7.14 * \frac{1407}{1000*342} = 0.029$ $\frac{a_c * A_s}{b * d} = \frac{0.029 * 1407}{1000 * 342} = 0.00012$ $\frac{\mathrm{x}}{\mathrm{d}} = 0.33$ from fig B. 3 X=0.33*342= 112.86 mm $\frac{I_c}{\mathrm{b} * \mathrm{d}^3} = 0.05$ from fig B. 4 $I_c=0.05*1000*(342)^3= 2000*10^6\ mm^4$ $\varepsilon_s = \frac{\mathrm{a_c} M}{E_s * \mathrm{I}_s} * (\mathrm{d} - \mathrm{x})$ $\varepsilon_s = \frac{7.14*204.68*10^6}{200*10^3*2000*10^6} * (342 - 112.86)$ =0.00083	ϕ 16 @ 100 mm C/C $\approx 2010mm^2$ /m

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Reference Bs-8110-1997	Calculation	Out put
Appendix B eq.1	$$\varepsilon_h = \frac{400-112.86}{342-112.86} * 0.00083 = 0.001$$ $$\varepsilon_m = \varepsilon_h - \frac{b*(h-x)(a-x)}{3E_S A_S(d-x)} = 0.0004$$ Design surface crack width= $$\frac{3a_{cr}\varepsilon_m}{1+2\left(\frac{a_{cr}-c_{min}}{h-x}\right)}$$ = $$\frac{3*76.5*0.0004}{1+2\left(\frac{76.5-50}{400-342}\right)} = 0.081 \text{ mm}$$ 0.081<0.2 mm ok	

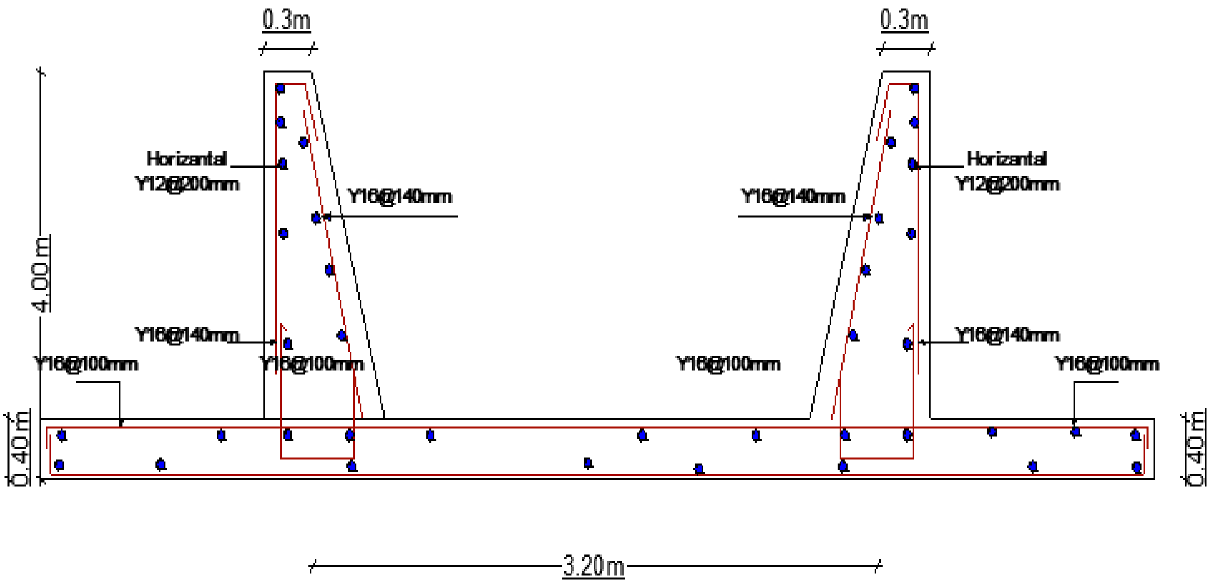

Fig. 5.5 Cross section of wall reinforcement

5.4 Design slab:

Reference Bs-8110-1997	**Calculation**	**Out put**
Table 2.1	$\frac{L_Y}{L_X} = \frac{9.60}{3.20}$= 9.60 /3.50 =2.7 > 2 (One way slab) self-weight = γ_c*H =25*0.15 = 3.75KN/m^2 **Design loads** design load=(1.4×5.25)+(1.6×2)=10.55 KN/m design load per span=10.55×3.5=33.76 KN Moments ==$\frac{W*L^2}{8}$=$\frac{33.76*3.20}{8}$ = 13.5KN.m Shear =$\frac{W}{2} = \frac{33.76}{2} = 16.88\text{KN}$	
clause3.4.4.4	**Design of moment steel** Assume 10 mm diameter bars with 25 mm cover. The effective depth is d=150−25−5=120 mm (i) Section at support	
clause 3:4:4:4 clause 3:4:4:1	$K = \frac{M}{d * b^2 f_{cu}} = \frac{13.5 * 10^6}{1000 * 120^2 * 30} = 0.3$ Z= 0.95d =114mm $A_S = \frac{M}{0.87F_YZ} = \frac{13.5*10^6}{0.87*460*114} = 286mm^2/\text{m}$	ϕ 8 mm @ 170 mm C/C 301 $mm^{2/}$m.
clause 3:4:4:4	The minimum area of reinforcement is $\frac{0.13B*H}{1000} = \frac{0.13*1000*150}{1000} = 195mm^2/m$	ϕ 8 mm @ 250 mm C/C

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Reference Bs-8110-1997	Calculation	Out put
clause 3:4:5:2 clause 3.4.5.6 equation 8 equation 7	**Shear resistance** $V = \frac{W}{2} = \frac{10.55}{2} = 5.27KN$ $V = \frac{V}{b*d} = \frac{5.27*10^3}{1000*120} = 0.44 N/mm^2$ $V_c = 0.79\left(\frac{100A_S}{b*d}\right)^{\frac{1}{3}}\left(\frac{400}{120}\right)^{\frac{1}{4}}\left(\frac{F_{cu}}{25}\right)^{\frac{1}{3}} \Big/ 1.25$ = 0.567 > V OK **Deflection** Actual = span/effective depth = 3200/120 = 26.67 mm $\frac{M}{d*b} = \frac{13.50*10^6}{1000*120} = 0.93$ $F_S = \frac{5}{8}\left(\frac{F_Y A_{req}}{A_{S\ Prov}}\right) = \frac{5}{8}\left(\frac{460*296}{301}\right) = 282.72 N/mm^2$ The modification factor is $= 0.55 + \left(\frac{477-F_S}{120}\right)\left(0.9 + \frac{M}{bd^2}\right) = 1.4$ allowable span/d ratio = 1.4*20 = 28mm Actual = span/effective depth = 26.67 mm The slab is satisfactory with respect to deflection.	

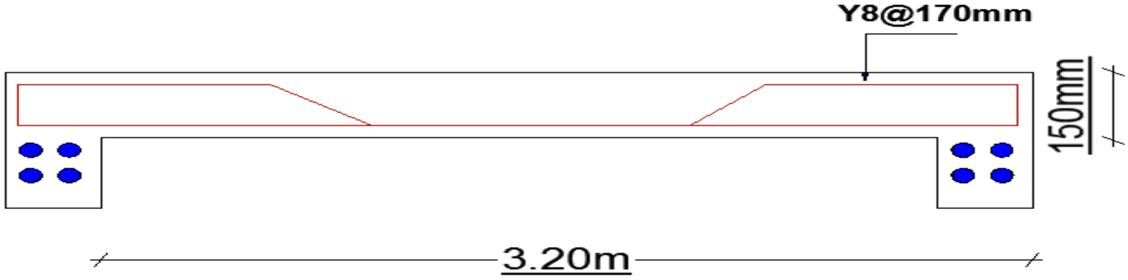

Fig. 5.6 Cross section slab reinforcement