SIMPLY SUPPORT FLANGE BEAM DESIGN

Continuous Beam

Figure 5.3 shows part of the first floor plan of a reinforced concrete office building. During construction, slabs and beams are cast together. The overall thickness of slab is 100mm and the dimensions of the beams are as given in the diagram. The finishes, ceiling and services form a characteristic permanent action of 1.5kN/m^2. The characteristic variable action is 3.0kN/m^2. Three metre high brickwall weighing 2.6kN/m^2 is placed over the entire span of all beams. The construction materials consist of Grade C25 concrete and Grade 500 steel reinforcement. For durability consideration a nominal cover of 30mm is required. Based on the information provided:

a) Calculate the design action carried by beam 2/B-C.

b) Sketch the bending moment and shear force diagrams of beam 2/B-C.

c) Design the beam for ultimate and serviceability limit states.

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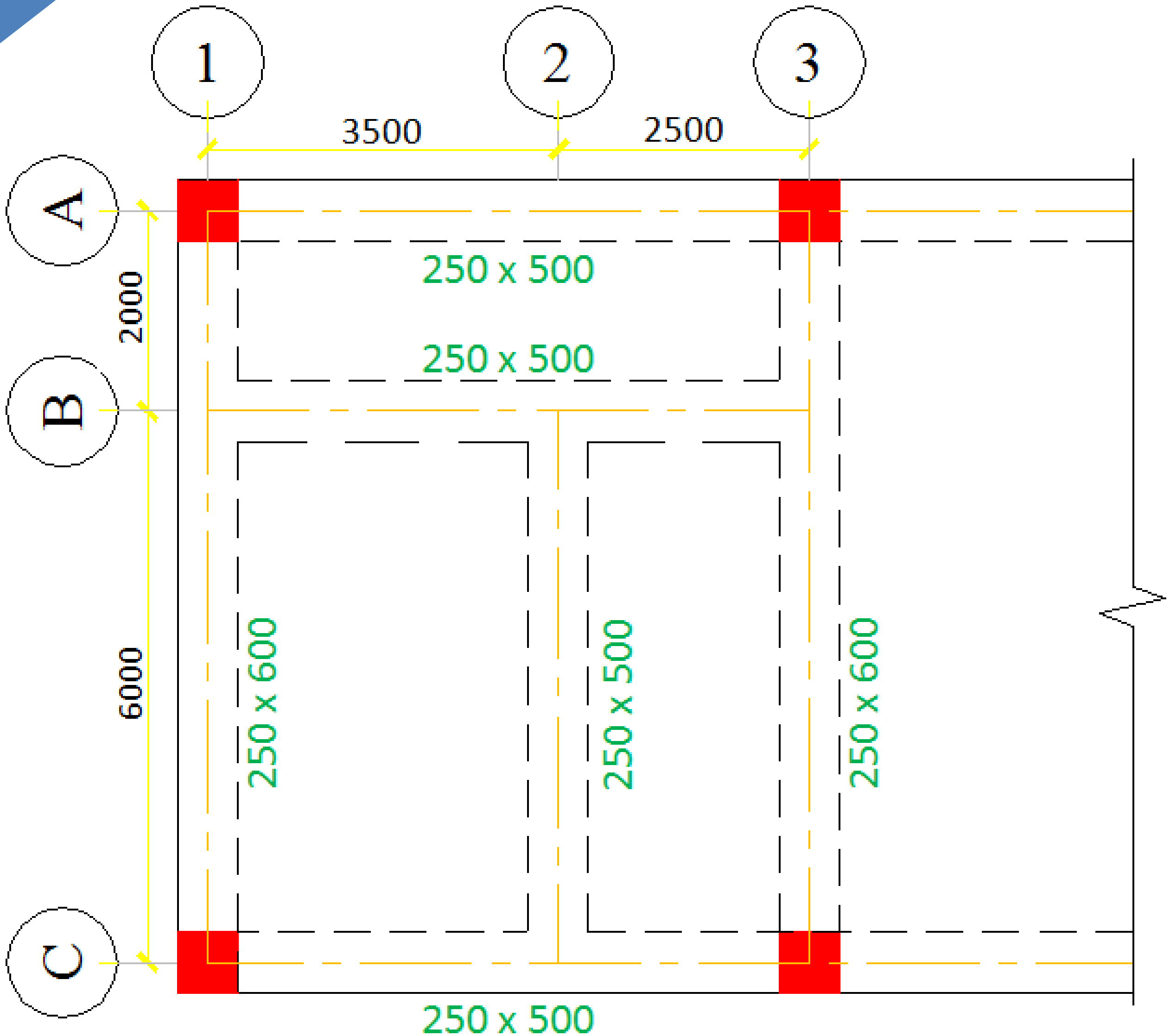

Figure 5.3

1.0 SPECIFICATION

Slab thickness	= 100mm
Slab Characteristic actions:	
Finishes etc.	= 1.5kN/m
Variable, q_k	= 3.0kN/m
Unit weight brick-wall	= 2.6kN/m^2
Materials:	
Characteristic strength of concrete, f_{ck}	= 25N/mm^2
Characteristic strength of steel, f_{yk}	= 500N/mm^2
Characteristic strength of link, f_{yk}	= 500N/mm^2
Unit weight of reinforced concrete	= 25kN/m^3
Nominal cover	= 30mm
Assumed:	
$Ø_{bar1}$	= 25mm
$Ø_{bar2}$	= 12mm
$Ø_{link}$	= 6mm

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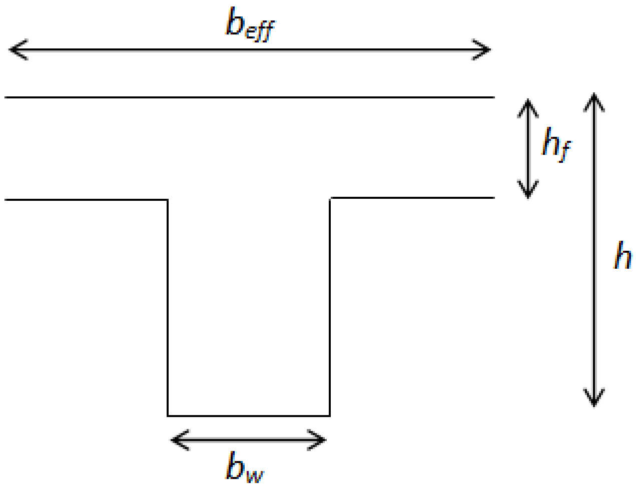
2.0 BEAM SIZE



$b_w = 250mm$
$h_f = 100mm$
$h = 500mm$
$b_1 = 1625mm$
$b_2 = 1125mm$
$l_o = 6000mm$

$b = b_1 + b_2 + b_w$
$= 2950mm$

$b_{eff,i} = 0.2b_i + 0.1l_o \leq 0.2l_o$

$= 0.2b1 + 0.1lo$
$= 915mm$

$0.2l_o = 1200mm$
$b_{eff,1} = 915mm$, $b_{eff,2} = 825mm$

Effective flange width,
$b_{eff} = \sum b_{eff,i} + b_w \leq b$

$= 915 + 825 + 250$
$= 1990mm < b$

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3.0 LOADING AND ANALYSIS

Loads on slab:

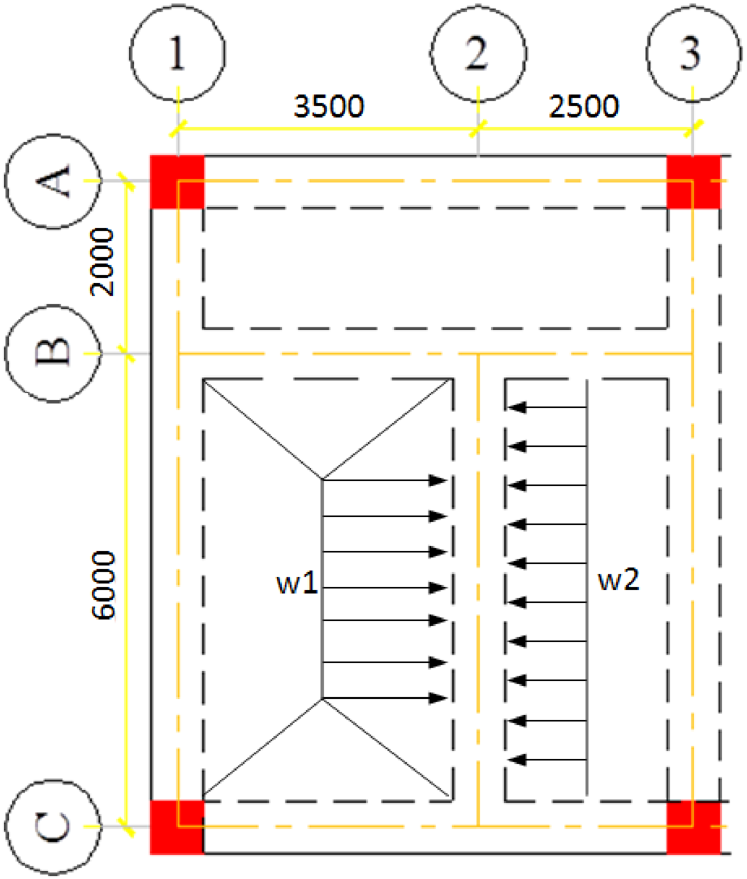
Self-weight	= 25 x 0.1
	= $2.5kN/m^2$
Finishes etc.	= $1.5kN/m^2$
Characteristic permanent action, g_k	= $4.0kN/m^2$
Characteristic variable action, q_k	= $3.0kN/m^2$

Distribution of load from slabs to beam areas follows:

Panel B-C/2-3: L_y/L_x = 6/2.5
= 2.4 > 2.0 (One-way slab)
Shear coefficient, β_v= 0.50

Panel B-C/1-2: L_y/L_x = 6/3.5
= 1.71 < 2.0 (Two-way slab)
Shear coefficient, β_v= 0.57

Loads on beam 2/B-C



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Characteristic permanent load

$w_o = 0.25\ (0.50-0.10) \times 25$		$= 2.50 kN/m$
$w_1 = \beta_v n L_x = 0.57 \times 4.00 \times 3.50$		$= 7.98 kN/m$
$w_2 = \beta_v n L_x = 0.50 \times 4.00 \times 2.50$		$= 5.00 kN/m$
$w_3 = 2.6 \times 3.0$ (Brickwall)		$= 7.8 kN/m$
	G_k	$= 23.28 kN/m$

Characteristic variable load

$w_1 = \beta_v n L_x = 0.57 \times 3.00 \times 3.50$		$= 5.99 kN/m$
$w_2 = \beta_v n L_x = 0.50 \times 3.00 \times 2.50$		$= 3.75 kN/m$
	Q_k	$= 9.74 kN/m$

Total design load on beam 2/B-C

$w = 1.35G_k + 1.5Q_k$	$= 46.03 kN/m$

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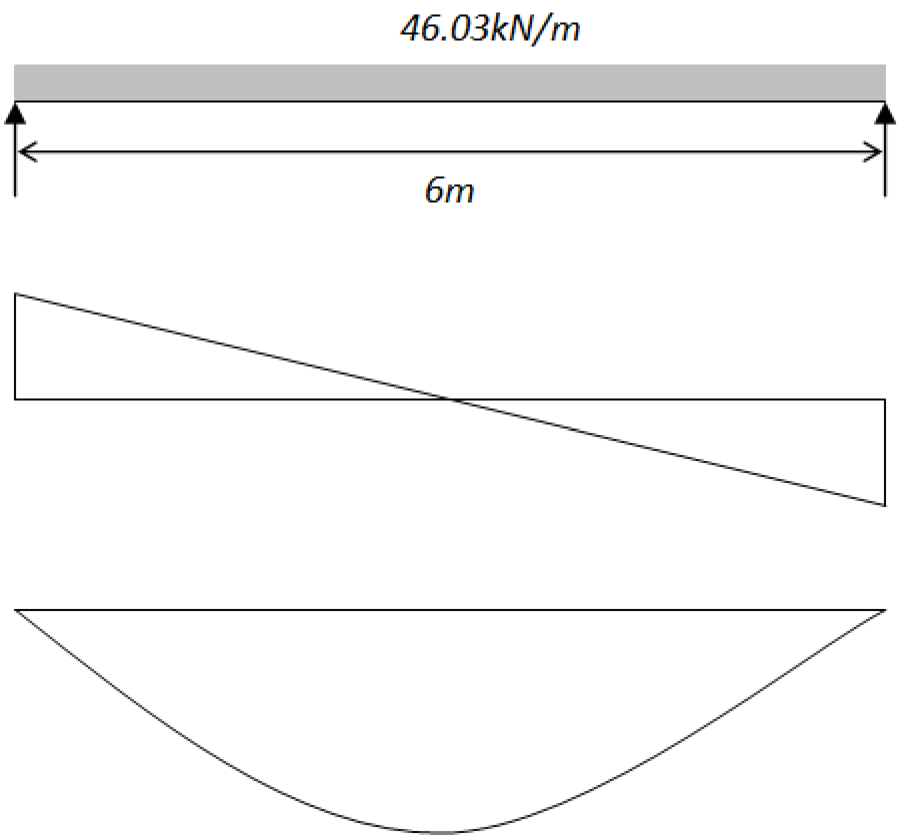

Shear Force,

$V = wL/2$

$= 138.1kN$

Bending Moment,

$M = wL^2/8$

$= 207.1kNm$

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4.0 MAIN REINFORCEMENT

Effective depth:
$d = h - C_{nom} - Ø_{link} - Ø_{bar}/2$ $= 451.5mm$

$M_{ed} = 207.1kNm$
$M_f = 0.567f_{ck}bh_f(d-0.5h_f)$ $= 0.567 \times 25 \times 1990 \times 100(451.5-50)$
$= 1133kNm$

$M_{ed} < M_f \implies$ Neutral axis within the flange $\therefore$ No compression reinforcement

$K = M/bd^2f_{ck}$ $= 207.1x10^6/(1990 \times 451.1^2 \times 25)$
$= 0.20$

$z = d[0.5+\sqrt{0.25 - K/1.134}$
$z = 0.98d$

Area of tension reinforcement
$A_s = M_{ed}/0.87f_{yk}z$ $= 207.1x10^6/(0.87 \times 25 \times 0.98 \times 451.5)$
$= 1110\ mm^2$

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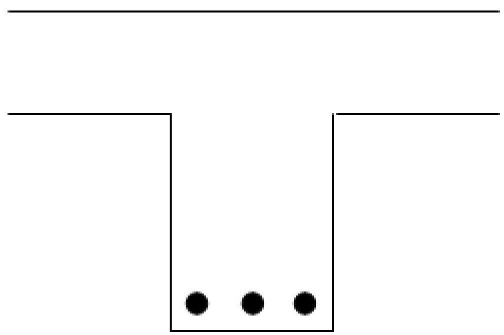

Use: 3H 25
($1473mm^2$)

Min. and max. reinforcement area,

$A_{s,min} = 0.26(f_{ctm}/f_{yk})bd$

$= 0.26(2.56/500)bd$
$= 0.0013bd$ (use $0.0013bd$)
$= 0.0013 \times 250 \times 451.5$
$= 151mm^2$

$A_{s,max} = 0.04A_c$

$= 0.04 \times 250 \times 500$
$= 5000mm^2$

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5.0 SHEAR REINFORCEMENT

V_{ed} = 138.1kN

Concrete strut capacity

$V_{Rd,max} = 0.36 b_w d f_{ck}(1-f_{ck}/250)/(\cot\theta+\tan\theta)$ = 318kN (θ=22, cotθ=2.5)
= 457kN (θ=45, cotθ=1.0)

$V_{ed} < V_{Rd,max}$ cotθ=2.5
$V_{ed} < V_{Rd,max}$ cotθ=1.0

Use θ=22, cotθ=2.5

Shear links

$A_{sw}/s = V_{ed}/(0.78 f_{yk} d \cot\theta)$ = $138.1 \times 10^3/(0.78 \times 500 \times 452 \times 2.5)$
= 0.317

Try link H6, A_{sw}=57mm^2
Spacing, s=57/0.317=178 mm

Use H6-150

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Max. spacing, $S = 0.75d$

$= 0.7 \times 452$

$= 339$ mm

Minimum links:-

$A_{sw}/s = (0.08 f_{ck}^{1/2} b_w)/f_{yk}$

$= (0.08 \times 25^{1/2} \times 250)/500$

$= 0.200$

Try link H6, $A_{sw} = 57 \text{mm}^2$

Spacing, $s = 57/0.200 = 283\text{mm} < S$

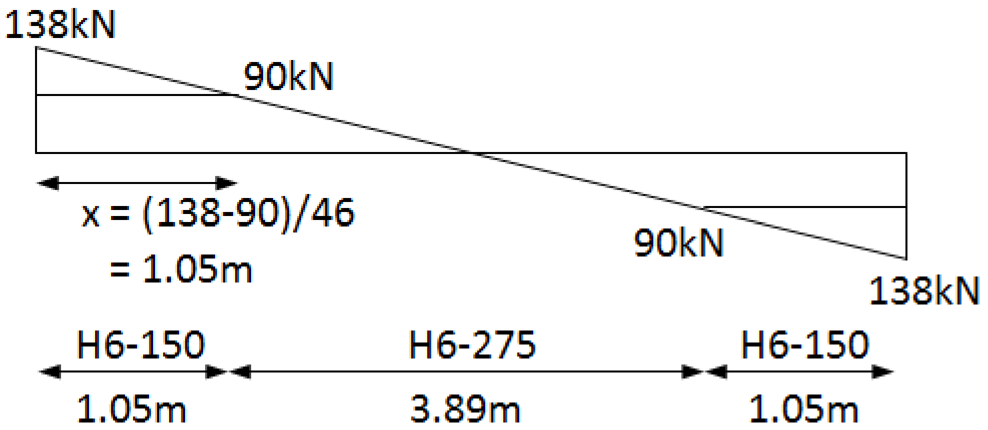
Use H6-275

Shear resistance of minimum links

$V_{min} = (A_{sw}/s)(0.78 d f_{yk} \cot\theta)$

$= (57/275)(0.78 \times 452 \times 500 \times 2.5)$

$= 90\text{kN}$



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Transverse steel in the flange

The longitudinal shear stresses are the greatest over a distance Δx measured from the point of zero moment,

$\Delta x = 0.5(L/2)$ $= 6000/4$

$= 1500mm$

The change in moment over distance Δx from zero moment

$\Delta M = (wL/2)(L/4) – (wL/4)(L/8) = 3wL^2/32$ $= (3 \times 46 \times 6.0^2)/32$

$= 155.35kNm$

The change in longitudinal force,

$\Delta F_d = [\Delta M/(d-0.5h_f] \times [(b-b_w)/2b]$ $= [155.35 \times 103/(452 - 50)] \times [(1990 - 250)/(2 \times 1990)]$

$= 169kN$

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Longitudinal shear stress

$v_{ed} = \Delta F_d/(h_f \Delta x)$ $= 169x10^3/(100 \times 1500)$

$= 1.13N/mm^2$

$v_{ed} > 0.27f_{ctk} = 0.27 \times 1.80 = 0.48N/mm^2$

Transverse shear reinforcement is required

Concrete strut capacity in the flange

$v_{ed,max} = 0.4f_{ck}(1-f_{ck}/250)/(\cot\theta+\tan\theta)$ $= 3.59N/mm^2$ (θ=26.5, $\cot\theta$=2.0)

$= 4.50N/mm^2$ (θ=45, $\cot\theta$=1.0)

$v_{ed} < v_{Rd,max}$ $\cot\theta$=2.5

$v_{ed} < v_{Rd,max}$ $\cot\theta$=1.0

Use θ=26.5, $\cot\theta$=2.0

Transverse shear reinforcement

$A_{sf}/s_f = v_{ed}h_f/0.87f_{yk}\cot\theta$ $= 1.13 \times 100 / (0.87 \times 500 \times 2.0)$

$= 0.13$

Try H10, $A_{sf} = 79mm^2$

Spacing, $s_f = 79/0.13 = 608mm$

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Minimum transverse steel area,

$A_{s,min} = 0.26(f_{ctm}/f_{yk})bh_f$ $= 0.26(2.56/500)bh_f$

$= 0.0013\ bh_f$ (use $0.0013\ bh_f$)

$= 0.0013 \times 1000 \times 100$

$= 133mm^2/m$

Use H10 – 500 ($157mm^2/m$)

Additional longitudinal reinforcement

Additional tensile force,

$\Delta F_{td} = 0.5V_{ed}\cot\theta$ $= 0.5 \times 138 \times 2.48$

$= 171kN$

$M_{ed,max}/z$ $= 207.1\times10^6/428.9$

$= 483kN$

$A_{s,req} = \Delta F_{td}/0.87f_{yk}$ $= 171\times10^3/(0.87 \times 500)$

$= 393mm^2$

Use 1H25 ($491mm^2$)

6.0 DEFLECTION

Percentage of required tension reinforcement,

$\rho = A_{s,req}/bd$ = 1110/(250X452)
= 0.010

Reference reinforcement ratio,

$\rho_0 = (f_{yk})^{1/2} \times 10^{-3}$ = $(25)^{1/2} \times 10^{-3}$
= 0.005

Percentage of required compression reinforcement

$\rho' = A_{s',req}/bd$ = 0/(250x587)
= 0.000

Factor for structural system, K = 1.0

$\rho > \rho_0$ ∴use equation (2)

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right]$$

= 1.0(11+3.81)
= 14.8

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Modification factor for flange width, $b/b_w > 3$	= 0.80
Modification for span less than 7m	= 1.0
Modification factor for steel area provided	$=A_{s,prov}/A_{s,req}$ =1473/1110 =1.33 < 1.5
Therefore allowable span-effective depth ratio	$=(l/d)_{allowable}$ =14.81 x 0.80 x 1 x 1.33 =15.7
Actual span-effective depth	$=(l/d)_{actual}$ =6000/451.5 $=13.3 < (l/d)_{allowable}$

Deflection is OK

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7.0 CRACKING

Limiting crack width, w_{max} = 0.3mm

Steel stress,

$$f_s = \frac{f_{yk}}{1.15} \times \left[\frac{G_k + 0.3Q_k}{1.35G_k + 1.5Q_k}\right]\frac{1}{\delta} = \left(\frac{500}{1.15}\right) \times \left[\frac{23.3 + (0.3 \times 9.7)}{46.0}\right] \times 1.0$$

= 285N/mm²

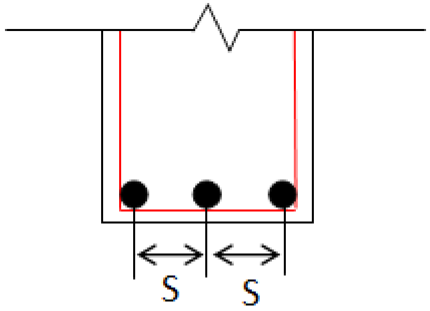
Max. allowable bar spacing =100mm

Bar spacing, S

= [250-2(30)-2(6)-20]/2

= 76.5mm < 100mm

(OK)



8.0 DETAILING