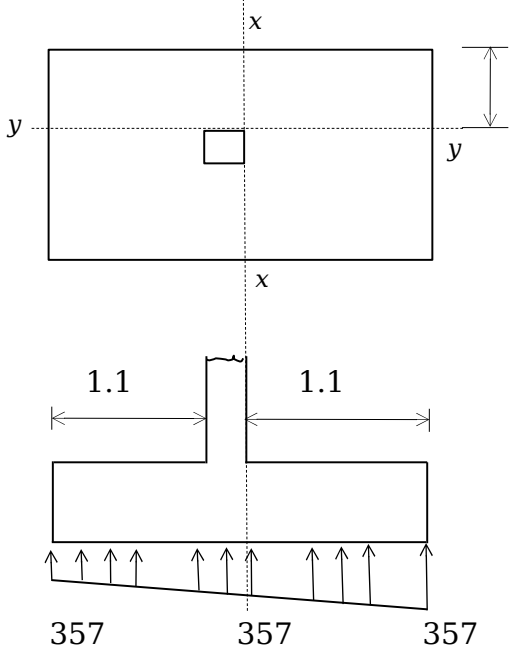
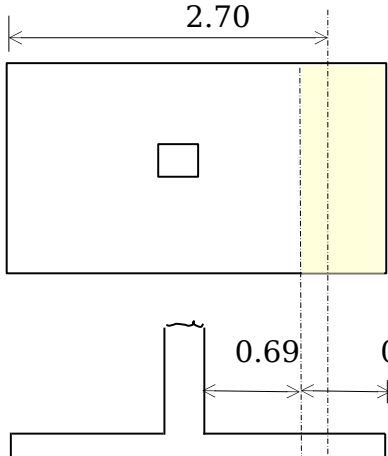


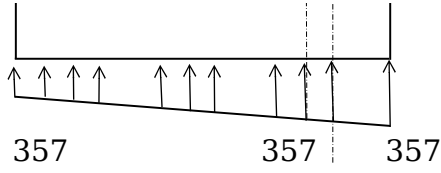
Ref.	Calculations
Project	kkkkkkkkkkkkkkkkkkkkk kkkkkkkkkkkkkkkkkkkkk
Location	
	FOOTING : D/5 SPECIFICATION Ultimate action: Axial, N = Moment, M = Materials : Concrete, f_{ck} = Reinforcement, f_{yk} = Unit weight of RC = Soil bearing capacity = Nominal cover, c_{nom} = Column size : 400 x 700 mm Assumed ϕ_{bar} = 16 mm
	SIZE Service actions: Axial, N = 3000 / 1.30 = 2308 kN Moment, M = 0 / 1.30 = 0 kNm Assumed selfweight of footing 10% of service load. Area of footing required, = $1.1N$ / Bearing capacity = 1.1×2308 / 300 = 8.5 m² Try size : $B \times H \times h$ 2.90 x 2.90 x 0.75 m A = 8.41 m² $I_{xx} = BH^3/12$ = 5.894 m⁴ $y = H/2$ = 1.45 m Selfweight , = 25 x 8.4 x 0.75 = 158 kN Max. soil pressure, $P = (N+W)/A + My/I$ $= (158 + 2308) / 8.4$ $+ 0 \times 1.45 / 5.9$ $= 293 \text{ kN/m}^2$ $< 300 \text{ kN/m}^2$

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Ref.	Calculations
	<u>ANALYSIS</u>  Ultimate soil pressure, $P = N/A \pm My/I$ $= (3000 / 8.4)$ $+ (0 \times 1.45 / 5.9)$ $= 357 \pm 0.0$ $P_{\max} = 357 \text{ kN/m}^2$ $P_{\min} = 357 \text{ kN/m}^2$ Bending moment at column face, $M_{xx} = (357 \times 1.100^2 / 2) + [(357 - 357) \times 1.100^2 \times 2/3]$ $= 216 \text{ kNm/m} \times 2.90 = 626 \text{ kNm}$ $M_{yy} = (357 \times 1.250^2 / 2)$ $= 279 \text{ kNm/m} \times 2.90 = 808 \text{ kNm}$
6.1	<u>MAIN REINFORCEMENT</u> Effective depth, $d_x = h - c_{\text{nom}} - 0.5\phi_{\text{bar}} = 750 - 50 - 0.5 \times 16 = 692 \text{ mm}$ $d_y = h - c_{\text{nom}} - 1.5\phi_{\text{bar}} = 750 - 50 - 1.5 \times 16 = 676 \text{ mm}$ Longitudinal bar Bending Moment, $M = 626 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 626 \times 10^6 / (2900 \times 692^2 \times 25)$ $= 0.018 < K_{\text{bal}} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.98 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$

	$= 626 \times 10^6 / (0.87 \times 500 \times 0.95 \times 692)$ $= 2189 \text{ mm}^2$

Ref.	Calculations
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (2.56 / 500) \times bd$ $= 0.0013 bd = 0.0013 \times 2900 \times 692 = 2677 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 2900 \times 750 = 87000 \text{ mm}^2$ Transverse bar Bending Moment, $M = 808 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 808 \times 10^6 / (2900 \times 676^2 \times 25)$ $= 0.024 < K_{bal} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.98 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$ $= 808 \times 10^6 / (0.87 \times 500 \times 0.95 \times 676)$ $= 2893 \text{ mm}^2$
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (2.56 / 500) \times bd$ $= 0.0013 bd = 0.0013 \times 2900 \times 676 = 2615 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 2900 \times 750 = 87000 \text{ mm}^2$
	<u>SHEAR</u> (i). Vertical shear : at $1.0d$ from column face  Average pressure at critical section $= 357 + (2.70 / 2.90) \times (0.0)$ 2.9 m = 357 kN/m² Vertical shear force $V_{Ed} = 357 \times 0.41 \times 2.9$ $= 422 \text{ kN}$

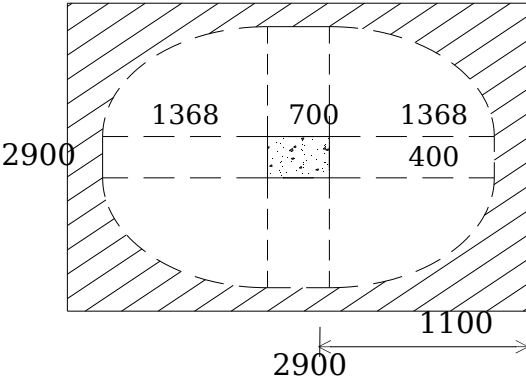
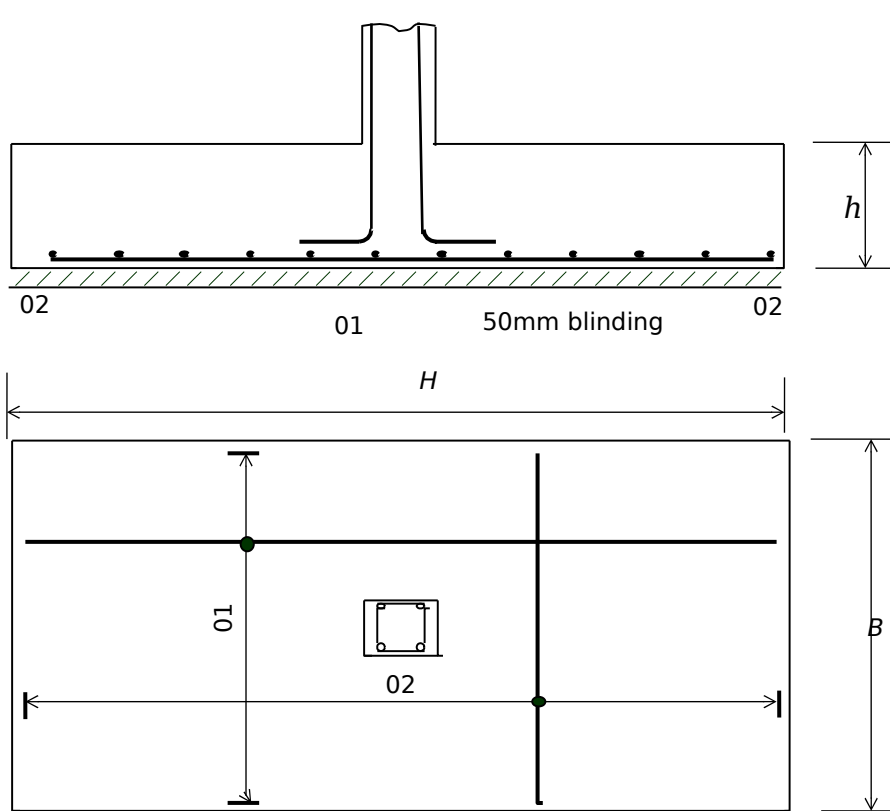
Ref.	Calculations
6.2.2	Design shear resistance, $V_{Rd,c} = [0.12 k (100 \rho_1 f_{ck})^{1/3}] bd$ $k = 1 + (200/d)^{1/2} \leq 2.0$ $= 1 + (200/692)^{1/2} = 1.54 \leq 2.0$ $\rho_1 = A_{sl}/bd \leq 0.02$ $= 4022 / (2900 \times 692) = 0.0058 \leq 0.02$ $V_{Rd,c} = 0.12 \times 1.54 \times (100 \times 0.006 \times 25)^{1/3} \times 2900 \times 692$ $= 903538 \text{ N} = 904 \text{ kN}$ $V_{min} = [0.035 k^{3/2} f_{ck}^{1/2}] bd$ $= 0.035 \times 1.54^2 \times 25^{1/2} \times 2900 \times 692$ $= 669589 \text{ N} = 670 \text{ kN}$ So, $V_{Rd,c} = 904 \text{ kN} > V_{Ed} = 422 \text{ kN}$
6.4.3	(ii). Punching shear at perimeter $2.0d$ from column face $d = (692 + 676) / 2 = 684 \text{ mm}$ $2d = 1368 \text{ mm}$  Control perimeter, $u = 2(700 + 400) + (2 \times \pi \times 1368)$ $= 10797 \text{ mm}$ Area within perimeter $A = (0.70 \times 0.40) + (2 \times 0.70 \times 1.368) + (2 \times 0.40 \times 1.368) + (\pi \times 1.368^2)$ $= 9.17 \text{ m}^2$ Average punching shear force at control perimeter $V_{Ed} = 357 [(2.90 \times 2.90) - 9.17] = -271 \text{ kN}$ Punching shear stress, $v_{Ed} = \beta (V_{Ed} / ud)$ where $\beta = 1 + k (M_{Ed}/V_{Ed})(u/W_1)$ $k = 0.65 \quad [c_1/c_2 = 700 / 400 = 1.8]$
6.4.3(3)	

Table 6.1	$W_1 = (0.5c_1^2) + c_1c_2 + 4c_2d + 16d^2 + 2\pi dc_1$ $= (0.50 \times 700^2) + (700 \times 400)$ $4) + (4 \times 400 \times 684) + (16 \times 684^2)$ $+ (2\pi \times 684 \times 400) =$ $= 10.82 \times 10^6$ $= 1 + [0.65 \times (0 \times 10^6 / -271 \times 10^3)$ $\times (10797 / 10.82 \times 10^6)]$ $\beta = 1.00$

Ref.	Calculations
6.4.3(2)	$v_{Ed} = 1.00 - (271 \times 10^3 / 10797 \times 692)$ $= -0.04 \text{ N/mm}^2$ Shear resistance, $v_{Rd,c} = 670 \times 10^3 / (10797 \times 684)$ $= 0.09 \text{ N/mm}^2 > v_{Ed}$ (iii). Maximum Punching Shear at column perimeter. Punching shear force at column perimeter $V_{Ed} = 3000 \text{ kN}$ Column perimeter $u_o = (700 + 400) \times 2 = 2200 \text{ mm}$ Punching shear stress, $v_{Ed} = \beta (V_{Ed} / u_o d)$ where $\beta = 1 + k (M_{Ed}/V_{Ed})(u_o/W_1)$
Table 6.1	$W_1 = (0.5c_1^2) + c_1c_2$ $= (0.50 \times 700^2) + (700 \times 400)$ $= 0.53 \times 10^6$ $= 1 + [0.65 \times (0 \times 10^6 / 3000 \times 10^3)$ $\times (2200 / 0.53 \times 10^6)]$ $\beta = 1.00$ $v_{Ed} = 1.00 - (3000 \times 10^3 / 2200 \times 684)$ $= 1.99 \text{ N/mm}^2$ Maximum shear resistance, $V_{Rd,max} = 0.5 [0.6(1 - f_{ck}/250)] f_{ck}/1.5$ $= 0.5 [0.6 (1 - 25 / 250)] (25/1.5)$ $= 4.50 \text{ N/mm}^2 > v_{Ed}$
7.3.3	CRACKING $h = 750 \text{ mm} > 200 \text{ mm}$ Assume steel stress under quasi permanent loading ,

Table 7.3N	$= 0.6 (f_{yk}/1.15)(A_{s,req}/A_{s,prov})$ $= 0.6 (500 / 1.15) (2189 / 4022)$ $= 142 \text{ N/mm}^2$ For design crack width = 0.3 mm Max. allowable bar spacing = 250 mm Max. bar spacing 1 = $[2900 - 2(58) - 16] / 19$ $= 146 \text{ mm} < 250 \text{ mm}$ Max. bar spacing 2 = $[2900 - 2(58) - 16] / 19$ $= 146 \text{ mm} < 250 \text{ mm}$

Ref.	Calculations
	<u>DETAILING</u>  Dimension: $B \times H \times h = 2900 \times 2900 \times 750$ Cover, $c_{nom} = 50 \text{ mm}$ Reinforcement: Bar mark 01: 20H16 @ 147

Bar mark 02: 20H 16 @ 147

Output	
Designed by :	1
EZIL	2
Checked by:	3
	4
Date:	5
	6
	7
	8
	9
	10
3000 kN	11
0 kNm	12
	13
25 N/mm ²	14
500 N/mm ²	15
25 kN/m ³	16
300 kN/m ²	17
50 mm	18
	19
	20
	21
	22
	23
	24
	25
	26
	27
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	34
	35
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	37
	38
	39
Size Ok!	40
	41

Use: 20 H16 (4022 mm ²)
msy '12

msy '12

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Output

Use: 20 H16
(4022 mm²)

1	
2	0.0013
3	
4	
5	
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	42
msy '12	43

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Output	
	1
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	3
	4
	5
	6
	7
	8
	9
	10
	11
Ok !	12
	13
	14
	15
	16
	17
	18
	19
	20
	21
	22
	23
	24
	25
	26
	27
	28
	29
	30
	31
	32

msy '12

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34
35
36
37
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39
40
41
42
43

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Output
Ok !
Ok !
Need spesific measure !

Ok !

Ok !

msy '12

Ok !

Ok !

msy '12

Ok !

Ok !

msy '12

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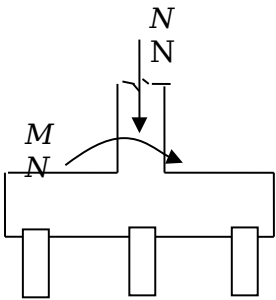
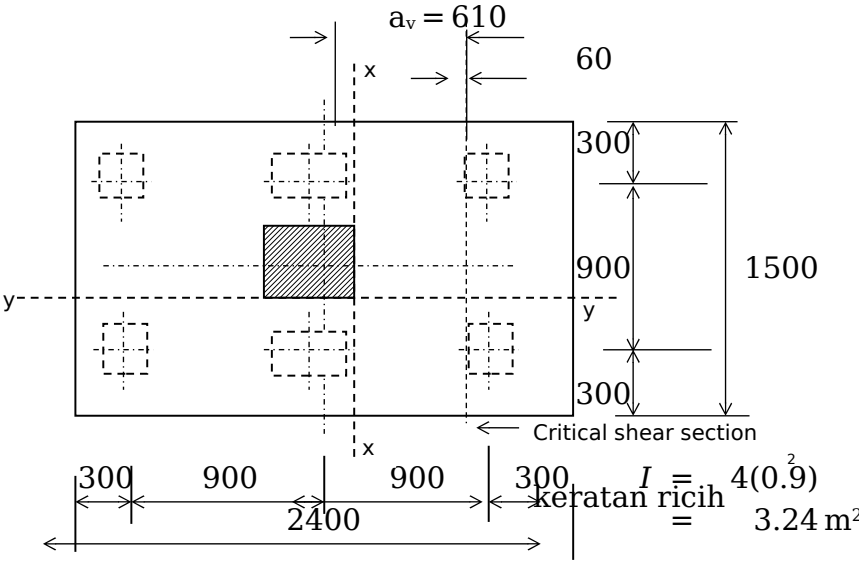
Output

mm

mm



Example 11.6 : Design of pile cap - Beam theory

Ref.	Calculations
	SPECIFICATION  Axial Load, $N_{ultimate}$ = Moment, $M_{ultimate}$ = Chac. Strength of concrete, f_{ck} = Chac. Strength of steel, f_{yk} = Cover, c = 75 mm ϕ_{bar} = Column size = 400 Pile : Precast RC pile 300 Working load = Safety factor =
	SIZE OF PILE CAP Service load = $(4200/1.40)$ = 3000 kN Assume selfweight of pilecap, say W = 200 kN No. of pile required = $(N + W) / \text{pile capacity}$ = $(3000 + 200) / 600$ = 5.3 Use : 6 Pile spacing = $k\phi_p$ = $3\phi_p$ Width, $B = (k + 1) \times \phi_p + 300$ = $(3 + 1)(300) + 300$ = 1500 mm Length, $L = (2k + 1) \times \phi_p + 300$ = $[2(3) + 1](300) + 300$ = 2400 mm Depth, $h = 2\phi_p + 400 = 2(300) + 400 = 1000$ mm Selfweight = $25(1.5 \times 2.4 \times 1.0)$ = 90 kN  Max. service load per pile, $F = (N + W)/n + Mx/I$ = $\{(3000 + 90.0) / 6\} + 53.6(0.90) / 3.24$ = 530 kN < 600 kN

Example 11.6 : Design of pile cap - Beam theory

Ref.	Calculations
	<u>Analysis</u> Ultimate load per pile, $F = P/n + Mx/I$ $= (4200 / 6) + 75.0 (0.90) / 3.24$ $= 700.0 + 20.8$ $= 721 \text{ kN}$ Maximum moment at column face, $M_{xx} = 2 \{ 721 \times (0.90 - 0.20) \} = 1009 \text{ kNm}$ $M_{yy} = 3 \{ 700 \times (0.45 - 0.20) \} = 525 \text{ kNm}$
6.1	<u>MAIN REINFORCEMENT</u> Effective depth, $d_x = h - C_{nom} - 0.5\phi_{bar} = 1000 - 75 - 0.5 \times 16 = 917 \text{ mm}$ $d_y = h - C_{nom} - 1.5\phi_{bar} = 1000 - 75 - 1.5 \times 16 = 901 \text{ mm}$ Longitudinal bar Bending Moment, $M = 1009 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 1009 \times 10^6 / (1500 \times 917^2 \times 35)$ $= 0.023 < K_{bal} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.98 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$ $= 1009 \times 10^6 / (0.87 \times 500 \times 0.95 \times 917)$ $= 2663 \text{ mm}^2/\text{m}$
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (3.21 / 500) \times bd$ $= 0.0017 bd = 0.0017 \times 1500 \times 917 = 2296 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 1500 \times 1000 = 60000 \text{ mm}^2$ Transverse bar Bending Moment, $M = 525 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 525 \times 10^6 / (2400 \times 901^2 \times 35)$ $= 0.008 < K_{bal} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.99 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$ $= 525 \times 10^6 / (0.87 \times 500 \times 0.95 \times 901)$

	= 1410 mm ² /m

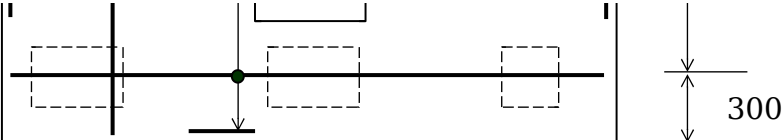
Example 11.6 : Design of pile cap - Beam theory

Ref.	Calculations
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (3.21 / 500) \times bd$ $= 0.0017 bd = 0.0017 \times 2400 \times 901 = 3609 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 2400 \times 1000 = 96000 \text{ mm}^2$
3.11.4.3	<u>SHEAR</u> (i). Vertical shear - at $\phi/5$ section inside pile 2 pile outside the critical section. Shear force, $V_{Ed} = 2(721) = 1442 \text{ kN}$ Pile spacing $< 3\phi_{pile}$ Consider shear enhancement over the whole width of pile cap Reduced shear force, $V_{Ed} = 1442 \times (a_v / 2d)$ $= 1442 \times 610 / (917 \times 2) = 480 \text{ kN}$
6.2.2	Design shear resistance, $V_{Rd,c} = [0.12 k(100\rho_1 f_{ck})^{1/3}] bd$ $k = 1 + (200/d)^{1/2} \leq 2.0$ $= 1 + (200 / 917)^{1/2} = 1.47 \leq 2.0$ $\rho_1 = A_{sl}/bd \leq 0.02$ $= 2815 / 1500 \times 917 = 0.0031 \leq 0.02$ $V_{Rd,c} = 0.12 \times 1.47 \times (100 \times 0.0031 \times 35)^{1/3} \times 1500 \times 917$ $= 534 \times 10^3 \text{ N} = 534 \text{ kN}$ $V_{min} = [0.035k^{3/2}f_{ck}^{1/2}] bd$ $= 0.035 \times 1.47^{3/2} \times 35^{1/2} \times 1500 \times 917$ $= 506 \times 10^3 \text{ N} = 506 \text{ kN}$ So, $V_{Rd,c} = 534 \text{ kN} > V_{Ed}$
6.4	(ii). Punching Shear : Critical at perimeter $2.0d$ from column face. Pile spacing $< 3\phi_{pile}$ No punching shear check is necessary (iii). Maximum Punching Shear at column perimeter. Maximum shear resistance, $V_{Rd,max} = 0.5ud [0.6(1 - f_{ck}/250)] f_{ck}/1.5$

	$= 0.5(4 \times 400) \times 909 [0.6 (1 - 35 / 250)] (35/1.5)$ $= 8755 \text{ kN} > V_{Ed, \max} = 4200 \text{ kN}$

Example 11.6 : Design of pile cap - Beam theory

Ref.	Calculations
7.3.3 Table 7.2N	CRACKING $h = 1000 \text{ mm} > 200 \text{ mm}$ Assume steel stress under quasi permanent loading , $= 0.55 (f_{yk}/1.15)(A_{s, \text{req}}/A_{s, \text{prov}})$ $= 0.55 (500 / 1.15) (2663 / 2815)$ $= 226 \text{ N/mm}^2$ For design crack width = 0.3 mm Max. allowable bar spacing = 200 mm Max. bar spacing 1 = $[1500 - 2(75) - 16] / 13$ $= 103 \text{ mm} < 200 \text{ mm}$ Max. bar spacing 2 = $[2400 - 2(75) - 16] / 15$ $= 149 \text{ mm} < 200 \text{ mm}$
	DETAILING 4H12 Binders 1000 300x300 precast RC pile 300 900 900 300 300 900 02

	
	Bar mark: 01 : 14H 16 @ 103 mm 02 : 16H 16 @ 149 mm Cover : 75 mm

Output
4200 kN 75 kNm 35 N/mm² 500 N/mm² 16 mm x 400 mm x 300 mm 600 kN 1.40
Ok

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Output
Use: 14 H16 (2815 mm²)
Use: 16 H16

(3217 mm ²)
msy '09

(3217 mm ²)
msy '09

page 3/4

Output
Ok !
Ok !

[illegible][illegible]

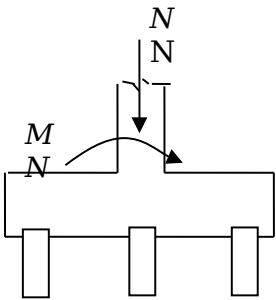
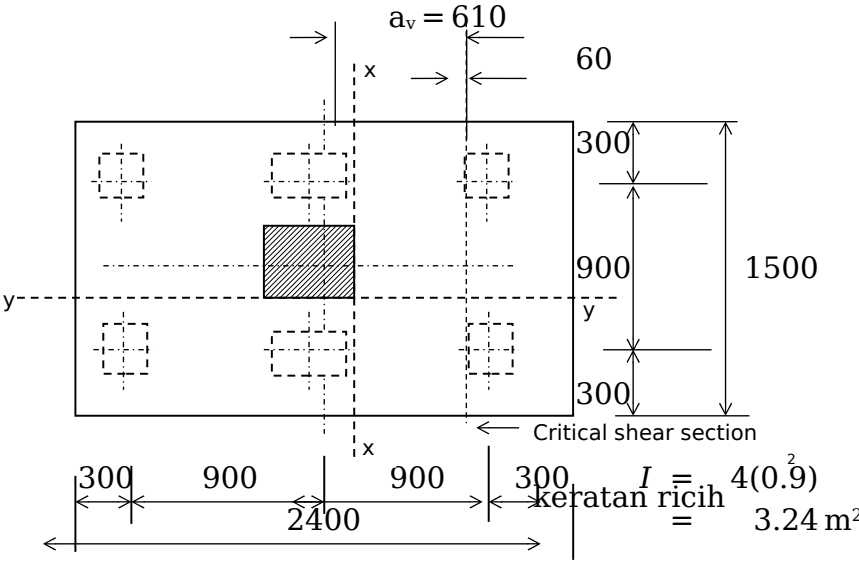
Ok !
msy '09

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Output
Need spesific measure !
Ok !
Ok !

msy '09

Example 9.6 : Design of pile cap - Beam theory

Ref.	Calculations
	SPECIFICATION  Axial Load, $N_{ultimate}$ = Moment, $M_{ultimate}$ = Chac. Strength of concrete, f_{ck} = Chac. Strength of steel, f_{yk} = Cover, c = 75 mm ϕ_{bar} = Column size = 400 Pile : Precast RC pile 300 Working load = Safety factor =
	SIZE OF PILE CAP Service load = $(4200/1.40)$ = 3000 kN Assume selfweight of pilecap, say W = 200 kN No. of pile required = $(N + W) / \text{pile capacity}$ = $(3000 + 200) / 600$ = 5.3 Use : 6 Pile spacing = $k\phi_p$ = $3\phi_p$ Width, $B = (k + 1) \times \phi_p + 300$ = $(3 + 1)(300) + 300$ = 1500 mm Length, $L = (2k + 1) \times \phi_p + 300$ = $[2(3) + 1](300) + 300$ = 2400 mm Depth, $h = 2\phi_p + 400 = 2(300) + 400 = 1000$ mm Selfweight = $25(1.5 \times 2.4 \times 1.0)$ = 90 kN  Max. service load per pile, $F = (N + W)/n + Mx/I$ = $\{(3000 + 90.0) / 6\} + 53.6(0.90) / 3.24$ = 530 kN < 600 kN

Example 9.4 : Design of pile cap - Beam theory

Ref.	Calculations
	<u>Analysis</u> Ultimate load per pile, $F = P/n + Mx/I$ $= (4200 / 6) + 75.0 (0.90) / 3.24$ $= 700.0 + 20.8$ $= 721 \text{ kN}$ Maximum moment at column face, $M_{xx} = 2 \{ 721 \times (0.90 - 0.20) \} = 1009 \text{ kNm}$ $M_{yy} = 3 \{ 700 \times (0.45 - 0.20) \} = 525 \text{ kNm}$
6.1	<u>MAIN REINFORCEMENT</u> Effective depth, $d_x = h - C_{nom} - 0.5\phi_{bar} = 1000 - 75 - 0.5 \times 16 = 917 \text{ mm}$ $d_y = h - C_{nom} - 1.5\phi_{bar} = 1000 - 75 - 1.5 \times 16 = 901 \text{ mm}$ Longitudinal bar Bending Moment, $M = 1009 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 1009 \times 10^6 / (1500 \times 917^2 \times 35)$ $= 0.023 < K_{bal} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.98 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$ $= 1009 \times 10^6 / (0.87 \times 500 \times 0.95 \times 917)$ $= 2663 \text{ mm}^2/\text{m}$
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (3.21 / 500) \times bd$ $= 0.0017 bd = 0.0017 \times 1500 \times 917 = 2296 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 1500 \times 1000 = 60000 \text{ mm}^2$ Transverse bar Bending Moment, $M = 525 \text{ kNm}$ $K = M / bd^2 f_{ck}$ $= 525 \times 10^6 / (2400 \times 901^2 \times 35)$ $= 0.008 < K_{bal} = 0.167$ Compression reinforcement is not required $z = d [0.5 + \sqrt{0.25 - K/1.134}] = 0.99 d \leq 0.95d$ $A_s = M / 0.87 f_{yk} z$ $= 525 \times 10^6 / (0.87 \times 500 \times 0.95 \times 901)$

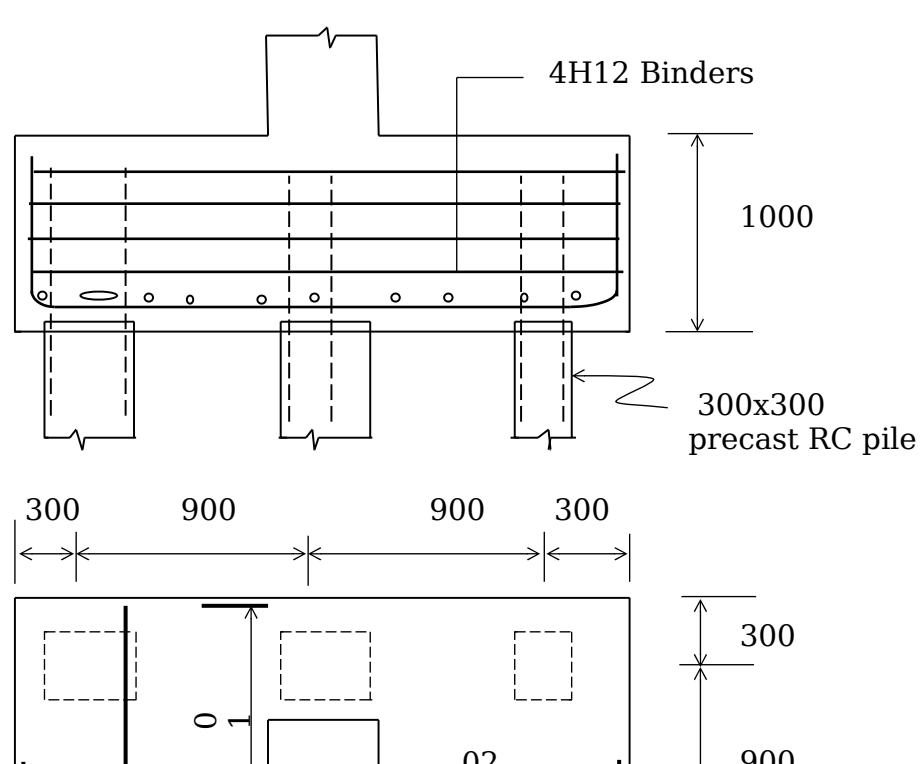
	= 1410 mm ² /m

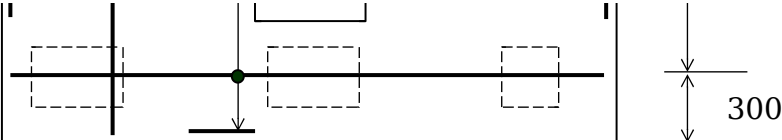
Example 9.4 : Design of pile cap - Beam theory

Ref.	Calculations
9.2.1.1	Minimum and maximum reinforcement area, $A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd = 0.26 \times (3.21 / 500) \times bd$ $= 0.0017 bd = 0.0017 \times 2400 \times 901 = 3609 \text{ mm}^2$ $A_{s,max} = 0.04A_c = 0.04 \times 2400 \times 1000 = 96000 \text{ mm}^2$
3.11.4.3	<u>SHEAR</u> (i). Vertical shear - at $\phi/5$ section inside pile 2 pile outside the critical section. Shear force, $V_{Ed} = 2(721) = 1442 \text{ kN}$ Pile spacing $< 3\phi_{pile}$ Consider shear enhancement over the whole width of pile cap Reduced shear force, $V_{Ed} = 1442 \times (a_v / 2d)$ $= 1442 \times 610 / (917 \times 2) = 480 \text{ kN}$
6.2.2	Design shear resistance, $V_{Rd,c} = [0.12 k(100\rho_1 f_{ck})^{1/3}] bd$ $k = 1 + (200/d)^{1/2} \leq 2.0$ $= 1 + (200 / 917)^{1/2} = 1.47 \leq 2.0$ $\rho_1 = A_{sl}/bd \leq 0.02$ $= 2815 / 1500 \times 917 = 0.0031 \leq 0.02$ $V_{Rd,c} = 0.12 \times 1.47 \times (100 \times 0.0031 \times 35)^{1/3} \times 1500 \times 917$ $= 534 \times 10^3 \text{ N} = 534 \text{ kN}$ $V_{min} = [0.035k^{3/2}f_{ck}^{1/2}] bd$ $= 0.035 \times 1.47^{3/2} \times 35^{1/2} \times 1500 \times 917$ $= 506 \times 10^3 \text{ N} = 506 \text{ kN}$
6.4	So, $V_{Rd,c} = 534 \text{ kN} > V_{Ed}$ (ii). Punching Shear : Critical at perimeter $2.0d$ from column face. Pile spacing $< 3\phi_{pile}$ No punching shear check is necessary (iii). Maximum Punching Shear at column perimeter. Maximum shear resistance, $V_{Rd,max} = 0.5ud [0.6(1 - f_{ck}/250)] f_{ck}/1.5$

	$= 0.5(4 \times 400) \times 909 [0.6 (1 - 35 / 250)] (35/1.5)$ $= 8755 \text{ kN} > V_{Ed, \max} = 4200 \text{ kN}$

Example 9.4 : Design of pile cap - Beam theory

Ref.	Calculations
7.3.3 Table 7.2N	CRACKING $h = 1000 \text{ mm} > 200 \text{ mm}$ Assume steel stress under quasi permanent loading , $= 0.55 (f_{yk}/1.15)(A_{s, \text{req}}/A_{s, \text{prov}})$ $= 0.55 (500 / 1.15) (2663 / 2815)$ $= 226 \text{ N/mm}^2$ For design crack width = 0.3 mm Max. allowable bar spacing = 200 mm Max. bar spacing 1 = $[1500 - 2(75) - 16] / 13$ $= 103 \text{ mm} < 200 \text{ mm}$ Max. bar spacing 2 = $[2400 - 2(75) - 16] / 15$ $= 149 \text{ mm} < 200 \text{ mm}$
	DETAILING  4H12 Binders 1000 300x300 precast RC pile 300 900 900 300 300 900 02

	 Bar mark: 01 : 14H 16 @ 103 mm 02 : 16H 16 @ 149 mm Cover : 75 mm

Output
4200 kN
75 kNm
35 N/mm ²
500 N/mm ²
16 mm
x 400 mm
x 300 mm
600 kN
1.40
Ok

Output

Use: 14 H16
(2815 mm²)

Use: 16 H16

(3217 mm ²)
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Output

Ok !
Ok !

Ok !

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Ok !

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Output
Need spesific measure ! Ok ! Ok !

Output
Need spesific measure ! Ok ! Ok !

Output
Need spesific measure ! Ok ! Ok !

Output
Need spesific measure ! Ok ! Ok !

msy '09