GEODOMISI Ltd. - Dr. Costas Sachpazis Civil & Geotechnical Engineering Consulting Company Tel:+30 210 5 238 127, Fax: 210 5711461, Mob: 0030 6936425722 Web:www.sachpazis.info, Email:costas©sachpazis.info	Project: **PROPOSED PIGS BUILDING FOR CHAGLANAS ABATTOIR**				Job Ref.	
	Section				Sheet no./rev. 1	
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Project: PROPOSED PIGS BUILDING FOR CHAGLANAS ABATTOIR

R.C. FOUNDATION ANALYSIS, DETAILS & DETAILING

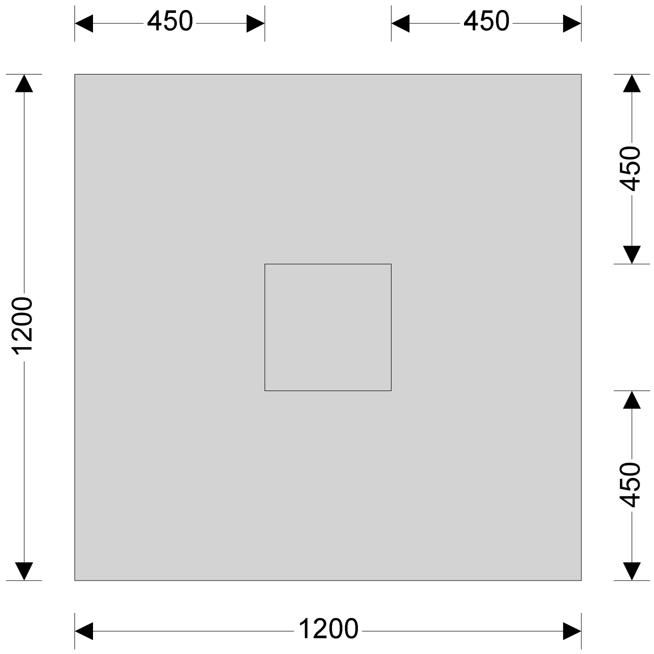
By

Kimberly Baptiste

02 Dec 2016

PAD FOOTING ANALYSIS & DESIGN (BS8110) FOR PROPOSED PIGS BUILDING FOR CHAGLANAS ABATTOIR

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)



Pad footing details

Length of pad footing; $L = \mathbf{1200}$ mm

Width of pad footing; $B = \mathbf{1200}$ mm

Area of pad footing; $A = L \times B = \mathbf{1.440}$ m^2

Depth of pad footing; $h = \mathbf{300}$ mm

Depth of soil over pad footing; $h_{soil} = \mathbf{850}$ mm

Density of concrete; $\rho_{conc} = \mathbf{24.0}$ kN/m^3

Column details

Column base length; $l_A = \mathbf{300}$ mm

Column base width; $b_A = \mathbf{300}$ mm

Column eccentricity in x; $e_{PxA} = \mathbf{0}$ mm

Column eccentricity in y; $e_{PyA} = \mathbf{0}$ mm

Soil details

Density of soil; $\rho_{soil} = \mathbf{19.0}$ kN/m^3

Design shear strength; $\phi' = \mathbf{20.0}$ deg

Design base friction; $\delta = \mathbf{15.0}$ deg

Allowable bearing pressure; $P_{bearing}$ = **110** kN/m^2

Axial loading on column

Dead axial load on column; P_{GA} = **37.0** kN

Imposed axial load on column; P_{QA} = **18.0** kN

Wind axial load on column; P_{WA} = **0.5** kN

Total axial load on column; P_A = **55.5** kN

Foundation loads

Dead surcharge load; F_{Gsur} = **0.000** kN/m^2

Imposed surcharge load; F_{Qsur} = **0.000** kN/m^2

Pad footing self weight; $F_{swt} = h \times \rho_{conc}$ = **7.200** kN/m^2

Soil self weight; $F_{soil} = h_{soil} \times \rho_{soil}$ = **16.150** kN/m^2

Total foundation load; $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil})$ = **33.6** kN

Horizontal loading on column base

Dead horizontal load in x direction; H_{GxA} = **0.0** kN

Imposed horizontal load in x direction; H_{QxA} = **0.0** kN

Wind horizontal load in x direction; H_{WxA} = **1.0** kN

Total horizontal load in x direction; H_{xA} = **1.0** kN

Dead horizontal load in y direction; H_{GyA} = **0.0** kN

Imposed horizontal load in y direction; H_{QyA} = **0.0** kN

Wind horizontal load in y direction; H_{WyA} = **1.0** kN

Total horizontal load in y direction; H_{yA} = **1.0** kN

Check stability against sliding

Resistance to sliding due to base friction

$H_{friction} = \max([P_{GA} + (F_{Gsur} + F_{swt} + F_{soil}) \times A], 0\ kN) \times \tan(\delta)$ = **18.9** kN

Passive pressure coefficient; $K_p = (1 + \sin(\phi')) / (1 - \sin(\phi'))$ = **2.040**

Stability against sliding in x direction

Passive resistance of soil in x direction; $H_{xpas} = 0.5 \times K_p \times (h^2 + 2 \times h \times h_{soil}) \times B \times \rho_{soil}$ = **14.0** kN

Total resistance to sliding in x direction; $H_{xres} = H_{friction} + H_{xpas}$ = **32.9** kN

PASS - Resistance to sliding is greater than horizontal load in x direction

Stability against sliding in y direction

Passive resistance of soil in y direction; $H_{ypas} = 0.5 \times K_p \times (h^2 + 2 \times h \times h_{soil}) \times L \times \rho_{soil}$ = **14.0** kN

Total resistance to sliding in y direction; $H_{yres} = H_{friction} + H_{ypas}$ = **32.9** kN

PASS - Resistance to sliding is greater than horizontal load in y direction

Check stability against overturning in x direction

Total overturning moment; $M_{xOT} = M_{xA} + H_{xA} \times h$ = **0.300** kNm

Restoring moment in x direction

Foundation loading; $M_{xsur} = A \times (F_{Gsur} + F_{swt} + F_{soil}) \times L / 2$ = **20.174** kNm

Axial loading on column; $M_{xaxial} = (P_{GA}) \times (L / 2 - e_{PxA})$ = **22.170** kNm

Total restoring moment; $M_{xres} = M_{xsur} + M_{xaxial}$ = **42.344** kNm

PASS - Restoring moment is greater than overturning moment in x direction

Check stability against overturning in y direction

Total overturning moment; $M_{yOT} = M_{yA} + H_{yA} \times h =$ **0.300** kNm

Restoring moment in y direction

Foundation loading; $M_{ysur} = A \times (F_{Gsur} + F_{swt} + F_{soil}) \times B / 2 =$ **20.174** kNm

Axial loading on column; $M_{yaxial} = (P_{GA}) \times (B / 2 - e_{PyA}) =$ **22.170** kNm

Total restoring moment; $M_{yres} = M_{ysur} + M_{yaxial} =$ **42.344** kNm

PASS - Restoring moment is greater than overturning moment in y direction

Calculate pad base reaction

Total base reaction; $T = F + P_A =$ **89.1** kN

Eccentricity of base reaction in x; $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T =$ **3** mm

Eccentricity of base reaction in y; $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T =$ **3** mm

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B =$ **0.006**

Base reaction acts within combined middle third of base

Calculate pad base pressures

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) =$ **59.774** kN/m^2

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) =$ **61.857** kN/m^2

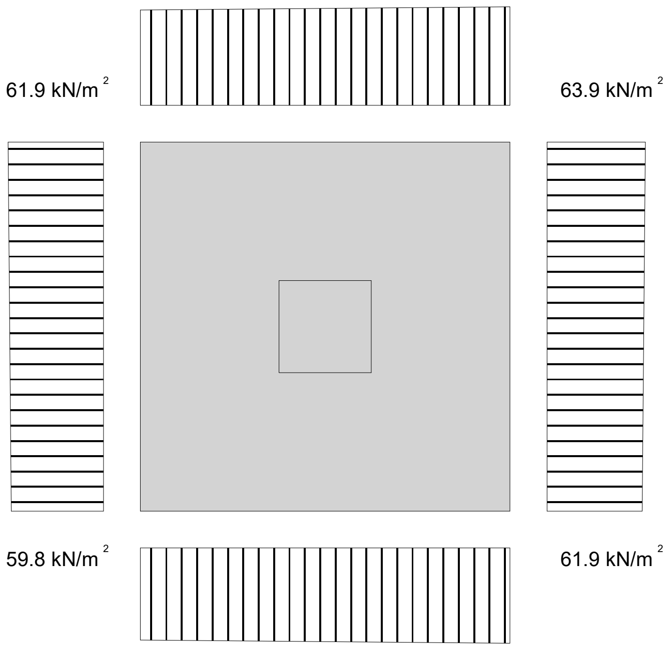
$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) =$ **61.857** kN/m^2

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) =$ **63.940** kN/m^2

Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) =$ **59.774** kN/m^2

Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) =$ **63.940** kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure



Partial safety factors for loads

Partial safety factor for dead loads; γ_{fG} = **1.40**

Partial safety factor for imposed loads; γ_{fQ} = **1.60**

Partial safety factor for wind loads; γ_{fW} = **0.00**

Ultimate axial loading on column

Ultimate axial load on column; $P_{uA} = P_{GA} \times \gamma_{fG} + P_{QA} \times \gamma_{fQ} + P_{WA} \times \gamma_{fW}$ = **80.5** kN

Ultimate foundation loads

Ultimate foundation load; $F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_{fG} + F_{Qsur} \times \gamma_{fQ}]$ = **47.1** kN

Ultimate horizontal loading on column

Ultimate horizontal load in x direction; $H_{xuA} = H_{GxA} \times \gamma_{fG} + H_{QxA} \times \gamma_{fQ} + H_{WxA} \times \gamma_{fW}$ = **0.0** kN

Ultimate horizontal load in y direction; $H_{yuA} = H_{GyA} \times \gamma_{fG} + H_{QyA} \times \gamma_{fQ} + H_{WyA} \times \gamma_{fW}$ = **0.0** kN

Ultimate moment on column

Ultimate moment on column in x direction; $M_{xuA} = M_{GxA} \times \gamma_{fG} + M_{QxA} \times \gamma_{fQ} + M_{WxA} \times \gamma_{fW}$ = **0.000** kNm

Ultimate moment on column in y direction; $M_{yuA} = M_{GyA} \times \gamma_{fG} + M_{QyA} \times \gamma_{fQ} + M_{WyA} \times \gamma_{fW}$ = **0.000** kNm

Calculate ultimate pad base reaction

Ultimate base reaction; $T_u = F_u + P_{uA}$ = **127.6** kN

Eccentricity of ultimate base reaction in x; $e_{Txu} = (P_{uA} \times e_{PxA} + M_{xuA} + H_{xuA} \times h) / T_u$ = **0** mm

Eccentricity of ultimate base reaction in y; $e_{Tyu} = (P_{uA} \times e_{PyA} + M_{yuA} + H_{yuA} \times h) / T_u$ = **0** mm

Calculate ultimate pad base pressures

$q_{1u} = T_u/A - 6 \times T_u \times e_{Txu}/(L \times A) - 6 \times T_u \times e_{Tyu}/(B \times A)$ = **88.614** kN/m^2

$q_{2u} = T_u/A - 6 \times T_u \times e_{Txu}/(L \times A) + 6 \times T_u \times e_{Tyu}/(B \times A)$ = **88.614** kN/m^2

$q_{3u} = T_u/A + 6 \times T_u \times e_{Txu}/(L \times A) - 6 \times T_u \times e_{Tyu}/(B \times A)$ = **88.614** kN/m^2

$q_{4u} = T_u/A + 6 \times T_u \times e_{Txu}/(L \times A) + 6 \times T_u \times e_{Tyu}/(B \times A)$ = **88.614** kN/m^2

Minimum ultimate base pressure; $q_{minu} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **88.614** kN/m^2

Maximum ultimate base pressure; $q_{maxu} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **88.614** kN/m^2

Calculate rate of change of base pressure in x direction

Left hand base reaction; $f_{uL} = (q_{1u} + q_{2u}) \times B / 2$ = **106.336** kN/m

Right hand base reaction; $f_{uR} = (q_{3u} + q_{4u}) \times B / 2$ = **106.336** kN/m

Length of base reaction; $L_x = L$ = **1200** mm

Rate of change of base pressure; $C_x = (f_{uR} - f_{uL}) / L_x$ = **0.000** kN/m/m

Calculate pad lengths in x direction

Left hand length; $L_L = L / 2 + e_{PxA}$ = **600** mm

Right hand length; $L_R = L / 2 - e_{PxA}$ = **600** mm

Calculate ultimate moments in x direction

Ultimate moment in x direction; $M_x = f_{uL} \times L_L^2 / 2 + C_x \times L_L^3 / 6 - F_u \times L_L^2 / (2 \times L)$ = **12.080** kNm

Calculate rate of change of base pressure in y direction

Top edge base reaction; $f_{uT} = (q_{2u} + q_{4u}) \times L / 2$ = **106.336** kN/m

Bottom edge base reaction; $f_{uB} = (q_{1u} + q_{3u}) \times L / 2$ = **106.336** kN/m

Length of base reaction; $L_y = B$ = **1200** mm

Rate of change of base pressure; $C_y = (f_{uB} - f_{uT}) / L_y$ = **0.000** kN/m/m

Calculate pad lengths in y direction

Top length; $L_T = B / 2 - e_{PyA} = \mathbf{600}$ mm

Bottom length; $L_B = B / 2 + e_{PyA} = \mathbf{600}$ mm

Calculate ultimate moments in y direction

Ultimate moment in y direction; $M_y = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) = \mathbf{12.080}$ kNm

Material details

Characteristic strength of concrete; $f_{cu} = \mathbf{25}$ N/mm^2

Characteristic strength of reinforcement; $f_y = \mathbf{500}$ N/mm^2

Characteristic strength of shear reinforcement; $f_{yv} = \mathbf{500}$ N/mm^2

Nominal cover to reinforcement; $c_{nom} = \mathbf{50}$ mm

Moment design in x direction

Diameter of tension reinforcement; $\phi_{xB} = \mathbf{16}$ mm

Depth of tension reinforcement; $d_x = h - c_{nom} - \phi_{xB} / 2 = \mathbf{242}$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_x = M_x / (B \times d_x^2 \times f_{cu}) = \mathbf{0.007}$

$K_x' = 0.156$

K_x < K_x' compression reinforcement is not required

Lever arm; $z_x = d_x \times \min([0.5 + \sqrt{(0.25 - K_x / 0.9)}], 0.95) = \mathbf{230}$ mm

Area of tension reinforcement required; $A_{s\_x\_req} = M_x / (0.87 \times f_y \times z_x) = \mathbf{121}$ mm^2

Minimum area of tension reinforcement; $A_{s\_x\_min} = 0.0013 \times B \times h = \mathbf{468}$ mm^2

Tension reinforcement provided; **6 No. 16 dia. bars bottom (225 centres)**

Area of tension reinforcement provided; $A_{s\_xB\_prov} = N_{xB} \times \pi \times \phi_{xB}^2 / 4 = \mathbf{1206}$ mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

Diameter of tension reinforcement; $\phi_{yB} = \mathbf{16}$ mm

Depth of tension reinforcement; $d_y = h - c_{nom} - \phi_{xB} - \phi_{yB} / 2 = \mathbf{226}$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_y = M_y / (L \times d_y^2 \times f_{cu}) = \mathbf{0.008}$

$K_y' = 0.156$

K_y < K_y' compression reinforcement is not required

Lever arm; $z_y = d_y \times \min([0.5 + \sqrt{(0.25 - K_y / 0.9)}], 0.95) = \mathbf{215}$ mm

Area of tension reinforcement required; $A_{s\_y\_req} = M_y / (0.87 \times f_y \times z_y) = \mathbf{129}$ mm^2

Minimum area of tension reinforcement; $A_{s\_y\_min} = 0.0013 \times L \times h = \mathbf{468}$ mm^2

Tension reinforcement provided; **6 No. 16 dia. bars bottom (225 centres)**

Area of tension reinforcement provided; $A_{s\_yB\_prov} = N_{yB} \times \pi \times \phi_{yB}^2 / 4 = \mathbf{1206}$ mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from top face of column

Ultimate pressure for shear; $q_{su} = (q_{1u} - C_y \times (B / 2 + e_{PyA} + b_A / 2 + d_y) / L + q_{4u}) / 2$

$q_{su} = \mathbf{88.614}$ kN/m^2

Area loaded for shear; $A_s = L \times (B / 2 - e_{PyA} - b_A / 2 - d_y) = \mathbf{0.269}$ m^2

Ultimate shear force; $V_{su} = A_s \times (q_{su} - F_u / A) = \mathbf{15.032}$ kN

Shear stresses at d from top face of column (cl 3.5.5.2)

Design shear stress; $v_{su} = V_{su} / (L \times d_y) = \mathbf{0.055}$ N/mm^2

From BS 8110:Part 1:1997 - Table 3.8

Design concrete shear stress; $v_c = 0.79$ N/mm$^2 \times \min(3, [100 \times A_{s_yB_prov} / (L \times d_y)]^{1/3}) \times \max((400 \text{ mm} / d_y)^{1/4}, 0.67) \times (\min(f_{cu} / 1 \text{ N/mm}^2, 40) / 25)^{1/3} / 1.25 = \mathbf{0.556}$ N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8 \text{ N/mm}^2 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5 \text{ N/mm}^2) = \mathbf{4.000}$ N/mm^2

PASS - $v_{su} < v_c$ - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ultimate pressure for punching shear; $q_{puA} = q_{1u}+[(L/2+e_{PxA}-l_A/2)+(l_A)/2]\times C_x/B-[(B/2+e_{PyA}-b_A/2)+(b_A)/2]\times C_y/L = \mathbf{88.614}$ kN/m^2

Average effective depth of reinforcement; $d = (d_x + d_y) / 2 = \mathbf{234}$ mm

Area loaded for punching shear at column; $A_{pA} = (l_A)\times(b_A) = \mathbf{0.090}$ m^2

Length of punching shear perimeter; $u_{pA} = 2\times(l_A)+2\times(b_A) = \mathbf{1200}$ mm

Ultimate shear force at shear perimeter; $V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = \mathbf{75.497}$ kN

Effective shear force at shear perimeter; $V_{puAeff} = V_{puA} = \mathbf{75.497}$ kN

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress; $v_{puA} = V_{puAeff} / (u_{pA} \times d) = \mathbf{0.269}$ N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8\text{N/mm}^2 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5 \text{ N/mm}^2) = \mathbf{4.000}$ N/mm^2

PASS - Design shear stress is less than allowable design shear stress

Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ultimate pressure for punching shear; $q_{puA1.5d} = q_{1u}+[L/2]\times C_x/B-[(B/2+e_{PyA}-b_A/2-1.5\times d)+(b_A+2\times 1.5\times d)/2]\times C_y/L = \mathbf{88.614}$ kN/m^2

Average effective depth of reinforcement; $d = (d_x + d_y) / 2 = \mathbf{234}$ mm

Area loaded for punching shear at column; $A_{pA1.5d} = L\times(b_A+2\times 1.5\times d) = \mathbf{1.202}$ m^2

Length of punching shear perimeter; $u_{pA1.5d} = 2\times L = \mathbf{2400}$ mm

Ultimate shear force at shear perimeter; $V_{puA1.5d} = P_{uA} + (F_u / A - q_{puA1.5d}) \times A_{pA1.5d} = \mathbf{13.287}$ kN

Effective shear force at shear perimeter; $V_{puA1.5deff} = V_{puA1.5d} \times 1.25 = \mathbf{16.609}$ kN

Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

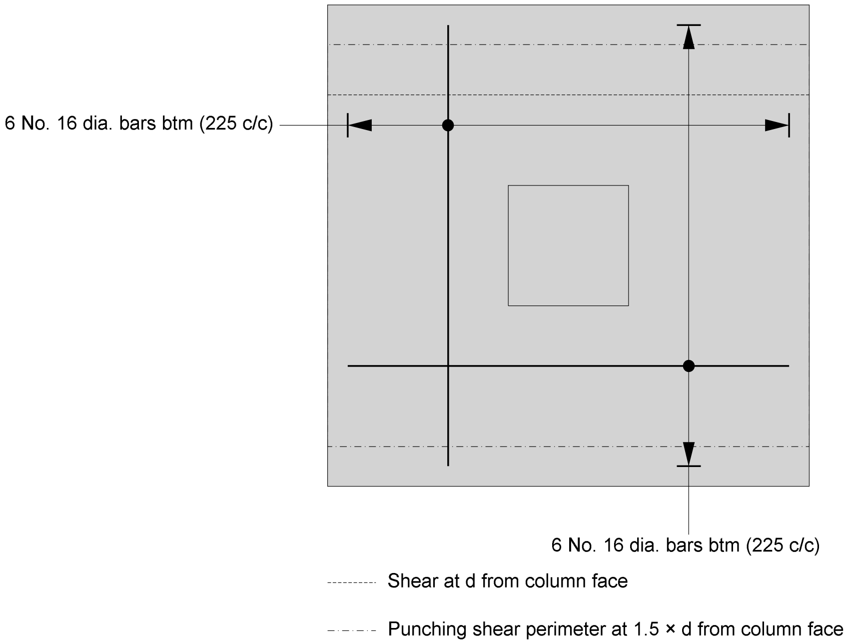
Design shear stress; $v_{puA1.5d} = V_{puA1.5deff} / (u_{pA1.5d} \times d) = \mathbf{0.030}$ N/mm^2

From BS 8110:Part 1:1997 - Table 3.8

Design concrete shear stress; $v_c = 0.79$ N/mm$^2 \times \min(3, [100 \times (A_{s_xB_prov} / (B \times d_x) + A_{s_yB_prov} / (L \times d_y)) / 2]^{1/3}) \times \max((800 \text{ mm} / (d_x + d_y))^{1/4}, 0.67) \times (\min(f_{cu} / 1 \text{ N/mm}^2, 40) / 25)^{1/3} / 1.25 = \mathbf{0.545}$ N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8\text{N/mm}^2 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5 \text{ N/mm}^2) = \mathbf{4.000}$ N/mm^2

PASS - $v_{puA1.5d} < v_c$ - No shear reinforcement required



6 No. 16 dia. bars btm (225 c/c)
6 No. 16 dia. bars btm (225 c/c)
Shear at d from column face
Punching shear perimeter at 1.5 × d from column face

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