LEMARG ENGINEERING	Project				Job Ref.	
	Section				Sheet no./rev. 1	
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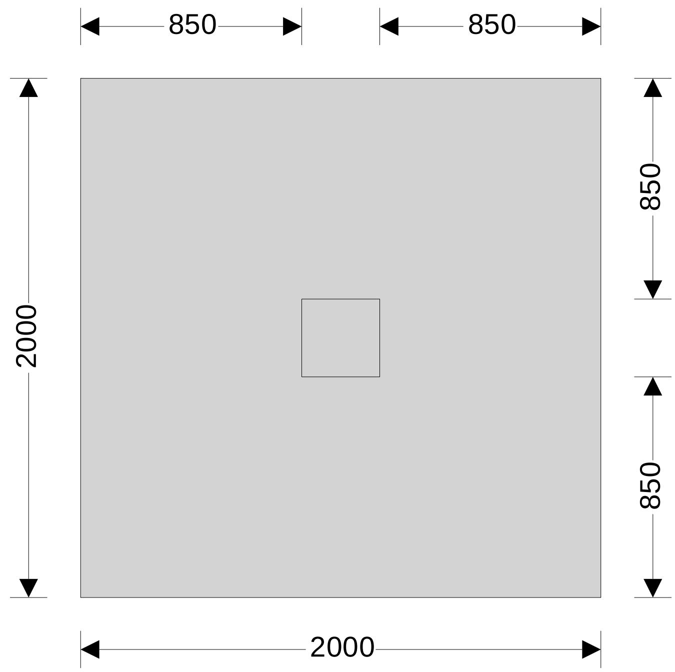
PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing: L = **2000** mm

Width of pad footing: B = **2000** mm

Area of pad footing: $A = L \times B = \mathbf{4.000}$ m^2

Depth of pad footing: h = **600** mm

Depth of soil over pad footing: h_{soil} = **750** mm

Density of concrete: ρ_{conc} = **24.0** kN/m^3

Column details

Column base length: l_A = **300** mm

Column base width: b_A = **300** mm

Column eccentricity in x: e_{PxA} = **0** mm

Column eccentricity in y: e_{PyA} = **0** mm

Soil details

Cohesive soil

Density of soil: ρ_{soil} = **18.0** kN/m^3

Design shear strength: ϕ' = **25.0** deg

Design base friction: δ = **19.3** deg

Allowable bearing pressure: $P_{bearing}$ = **250** kN/m^2

Axial loading on column

Dead axial load on column: P_{GA} = **500.0** kN

Imposed axial load on column: P_{QA} = **300.0** kN

Wind axial load on column P_{WA} = **0.0** kN

Total axial load on column P_A = **800.0** 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 $F_{swt} = h \times \rho_{conc}$ = **14.400** kN/m^2

Soil self weight $F_{soil} = h_{soil} \times \rho_{soil}$ = **13.500** kN/m^2

Total foundation load $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil})$ = **111.6** kN

Calculate pad base reaction

Total base reaction $T = F + P_A$ = **911.6** kN

Eccentricity of base reaction in x $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T$ = **0** mm

Eccentricity of base reaction in y $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T$ = **0** mm

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B$ = **0.000**

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **227.900** kN/m^2

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **227.900** kN/m^2

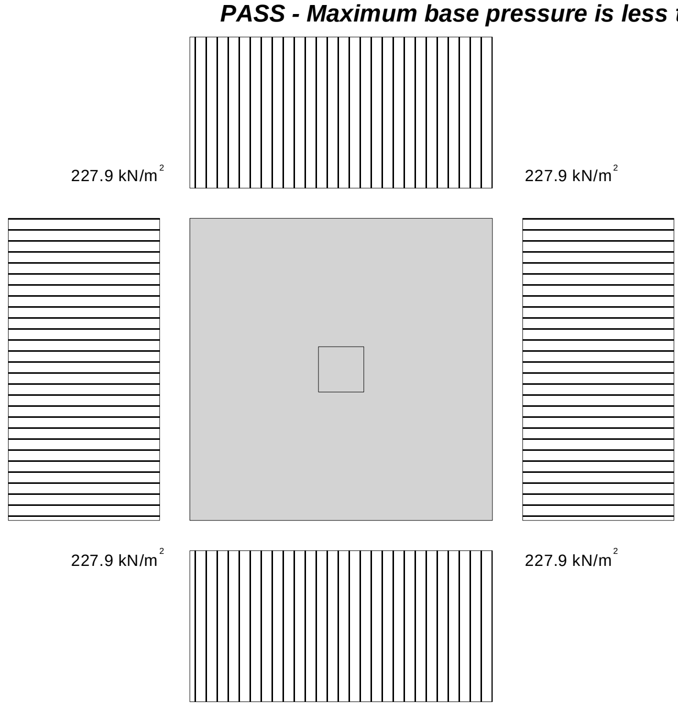
$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **227.900** kN/m^2

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **227.900** kN/m^2

Minimum base pressure $q_{min} = min(q_1, q_2, q_3, q_4)$ = **227.900** kN/m^2

Maximum base pressure $q_{max} = max(q_1, q_2, q_3, q_4)$ = **227.900** 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 — $P_{uA} = P_{GA} \times \gamma_{fG} + P_{QA} \times \gamma_{fQ} + P_{WA} \times \gamma_{fW}$ = **1180.0** kN

Ultimate foundation loads

Ultimate foundation load — $F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_{fG} + F_{Qsur} \times \gamma_{fQ}]$ = **156.2** 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 — $T_u = F_u + P_{uA}$ = **1336.2** 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

$q_{1u} = T_u/A - 6 \times T_u \times e_{Txu}/(L \times A) - 6 \times T_u \times e_{Tyu}/(B \times A)$ = **334.060** kN/m^2

$q_{2u} = T_u/A - 6 \times T_u \times e_{Txu}/(L \times A) + 6 \times T_u \times e_{Tyu}/(B \times A)$ = **334.060** kN/m^2

$q_{3u} = T_u/A + 6 \times T_u \times e_{Txu}/(L \times A) - 6 \times T_u \times e_{Tyu}/(B \times A)$ = **334.060** kN/m^2

$q_{4u} = T_u/A + 6 \times T_u \times e_{Txu}/(L \times A) + 6 \times T_u \times e_{Tyu}/(B \times A)$ = **334.060** kN/m^2

Minimum ultimate base pressure — $q_{minu} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **334.060** kN/m^2

Maximum ultimate base pressure — $q_{maxu} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **334.060** kN/m^2

Calculate rate of change of base pressure in x direction

Left hand base reaction — $f_{uL} = (q_{1u} + q_{2u}) \times B / 2$ = **668.120** kN/m

Right hand base reaction — $f_{uR} = (q_{3u} + q_{4u}) \times B / 2$ = **668.120** kN/m

Length of base reaction — $L_x = L$ = **2000** 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 — $L_L = L / 2 + e_{PxA}$ = **1000** mm

Right hand length — $L_R = L / 2 - e_{PxA}$ = **1000** mm

Calculate ultimate moments in x direction

Ultimate moment in x direction — $M_x = f_{uL} \times L_L^2 / 2 + C_x \times L_L^3 / 6 - F_u \times L_L^2 / (2 \times L)$ = **295.000** kNm

Calculate rate of change of base pressure in y direction

Top edge base reaction — $f_{uT} = (q_{2u} + q_{4u}) \times L / 2$ = **668.120** kN/m

Bottom edge base reaction — $f_{uB} = (q_{1u} + q_{3u}) \times L / 2$ = **668.120** kN/m

Length of base reaction — $L_y = B$ = **2000** 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 $L_T = B / 2 - e_{PyA} =$ **1000** mm

Bottom length $L_B = B / 2 + e_{PyA} =$ **1000** mm

Calculate ultimate moments in y direction

Ultimate moment in y direction $M_y = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) =$ **295.000** kNm

Material details

Characteristic strength of concrete $f_{cu} =$ **30** N/mm^2

Characteristic strength of reinforcement $f_y =$ **500** N/mm^2

Characteristic strength of shear reinforcement $f_{yv} =$ **250** N/mm^2

Nominal cover to reinforcement $c_{nom} =$ **50** mm

Moment design in x direction

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

Depth of tension reinforcement $d_x = h - c_{nom} - \phi_{xB} / 2 =$ **542** mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_x = M_x / (B \times d_x^2 \times f_{cu}) =$ **0.017**

$K_x' = 0.156$

$K_x < K_x'$ compression reinforcement is not required

Lever arm $z_x = d_x \times \min([0.5 + \sqrt{(0.25 - K_x / 0.9)}], 0.95) =$ **515** mm

Area of tension reinforcement required $A_{s\_x\_req} = M_x / (0.87 \times f_y \times z_x) =$ **1317** mm^2

Minimum area of tension reinforcement $A_{s\_x\_min} = 0.0013 \times B \times h =$ **1560** mm^2

Tension reinforcement provided **12 No. 16 dia. bars bottom (175 centres)**

Area of tension reinforcement provided $A_{s\_xB\_prov} = N_{xB} \times \pi \times \phi_{xB}^2 / 4 =$ **2413** mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

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

Depth of tension reinforcement $d_y = h - c_{nom} - \phi_{xB} - \phi_{yB} / 2 =$ **526** mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_y = M_y / (L \times d_y^2 \times f_{cu}) =$ **0.018**

$K_y' = 0.156$

$K_y < K_y'$ compression reinforcement is not required

Lever arm $z_y = d_y \times \min([0.5 + \sqrt{(0.25 - K_y / 0.9)}], 0.95) =$ **500** mm

Area of tension reinforcement required $A_{s\_y\_req} = M_y / (0.87 \times f_y \times z_y) =$ **1357** mm^2

Minimum area of tension reinforcement $A_{s\_y\_min} = 0.0013 \times L \times h =$ **1560** mm^2

Tension reinforcement provided **10 No. 16 dia. bars bottom (200 centres)**

Area of tension reinforcement provided $A_{s\_yB\_prov} = N_{yB} \times \pi \times \phi_{yB}^2 / 4 =$ **2011** 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 $q_{su} = (q_{1u} - C_y \times (B / 2 + e_{PyA} + b_A / 2 + d_y) / L + q_{4u}) / 2$

$q_{su} =$ **334.060** kN/m^2

Area loaded for shear $A_s = L \times (B / 2 - e_{PyA} - b_A / 2 - d_y) =$ **0.648** m^2

Ultimate shear force $V_{su} = A_s \times (q_{su} - F_u / A) =$ **191.160** kN

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

Design shear stress — $v_{su} = V_{su} / (L \times d_y) = \mathbf{0.182}\ N/mm^2$

From BS 8110:Part 1:1997 - Table 3.8

Design concrete shear stress — $v_c = 0.79\ N/mm^2 \times \min(3, [100 \times A_{s\_yB\_prov} / (L \times d_y)]^{1/3}) \times \max((400\ mm / d_y)^{1/4}, 0.67) \times (\min(f_{cu} / 1\ N/mm^2, 40) / 25)^{1/3} / 1.25 = \mathbf{0.361}\ N/mm^2$

Allowable design shear stress — $v_{max} = \min(0.8\ N/mm^2 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5\ N/mm^2) = \mathbf{4.382}\ N/mm^2$

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

Calculate ultimate punching shear force at face of column

Ultimate pressure for punching shear — $q_{puA} = q_{1u}+[(L/2+e_{PxA}-l_A/2)+(l_A)/2]\times C_x/B-[(B/2+e_{PyA}-b_A/2)+(b_A)/2]\times C_y/L = \mathbf{334.060}\ kN/m^2$

Average effective depth of reinforcement — $d = (d_x + d_y) / 2 = \mathbf{534}\ mm$

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 — $V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = \mathbf{1153.450}\ kN$

Effective shear force at shear perimeter — $V_{puAeff} = V_{puA} = \mathbf{1153.450}\ kN$

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress — $v_{puA} = V_{puAeff} / (u_{pA} \times d) = \mathbf{1.800}\ N/mm^2$

Allowable design shear stress — $v_{max} = \min(0.8N/mm^2 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5\ N/mm^2) = \mathbf{4.382}\ N/mm^2$

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 — $q_{puA1.5d} = q_{1u}+[L/2]\times C_x/B-[(B/2+e_{PyA}-b_A/2-1.5\times d)+(b_A+2\times 1.5\times d)/2]\times C_y/L = \mathbf{334.060}\ kN/m^2$

Average effective depth of reinforcement — $d = (d_x + d_y) / 2 = \mathbf{534}\ mm$

Area loaded for punching shear at column — $A_{pA1.5d} = L\times(b_A+2\times 1.5\times d) = \mathbf{3.804}\ m^2$

Length of punching shear perimeter — $u_{pA1.5d} = 2\times L = \mathbf{4000}\ mm$

Ultimate shear force at shear perimeter — $V_{puA1.5d} = P_{uA} + (F_u / A - q_{puA1.5d}) \times A_{pA1.5d} = \mathbf{57.820}\ kN$

Effective shear force at shear perimeter — $V_{puA1.5deff} = V_{puA1.5d} \times 1.25 = \mathbf{72.275}\ kN$

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

Design shear stress — $v_{puA1.5d} = V_{puA1.5deff} / (u_{pA1.5d} \times d) = \mathbf{0.034}\ N/mm^2$

From BS 8110:Part 1:1997 - Table 3.8

Design concrete shear stress — $v_c = 0.79\ N/mm^2 \times \min(3, [100 \times (A_{s\_xB\_prov} / (B \times d_x) + A_{s\_yB\_prov} / (L \times d_y)) / 2]^{1/3}) \times \max((800\ mm / (d_x + d_y))^{1/4}, 0.67) \times (\min(f_{cu} / 1\ N/mm^2, 40) / 25)^{1/3} / 1.25 = \mathbf{0.370}\ N/mm^2$

Allowable design shear stress — $v_{max} = \min(0.8N/mm^2 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5\ N/mm^2) = \mathbf{4.382}\ N/mm^2$

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

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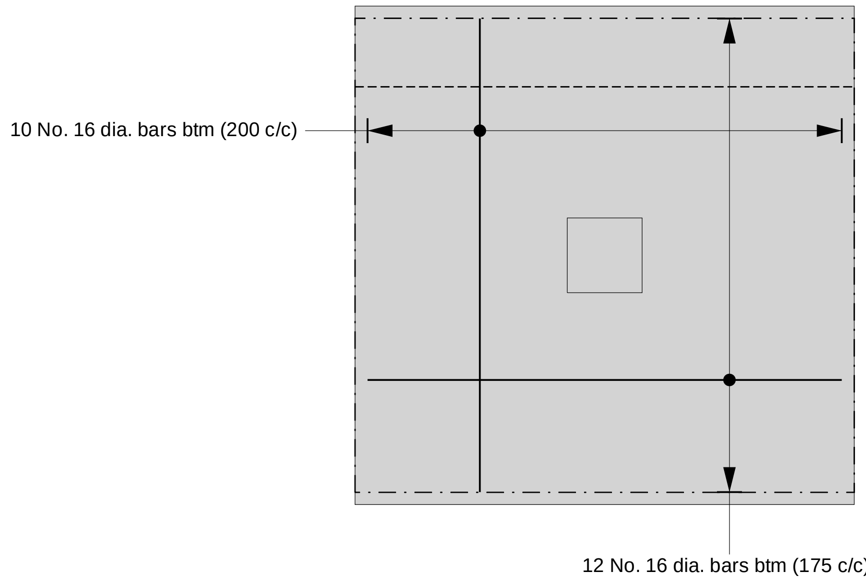

---- Shear at d from column face

— · — Punching shear perimeter at 1.5 × d from column face