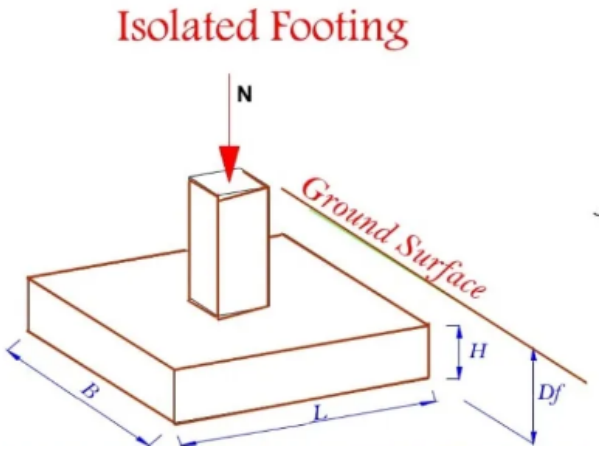
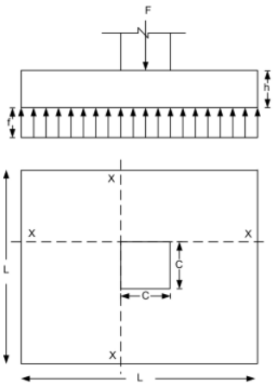
ISOLATED FOOTING DESIGN REPORT

Element: Isolated Footing 001

Design Code: BS EN 1992-1-1:2004 | Date: 28 October 2026

REFERENCE	CALCULATIONS	OUTPUT
§6.4.2 Loading Conditions	**SERVICE LOAD**  Column dimensions bc = 300.0 mm hc = 300.0 mm Unfactored dead load $G_k = 500.0kN$ Unfactored live load $Q_k = 150.0kN$ Service load $N_s = 1.0G_k + 1.0Q_k$ $N_s = 1.0 \times 500.0 + 1.0 \times 150.0 = 650.0kN$	**N_s = 650.0 kN** **G_k = 500.0 kN** **Q_k = 150.0 kN**
§6.5 Footing Size – SLS	**SIZING AT SLS** Soil bearing pressure q_s = 200.0 kN/m² Unit weight of concrete γ = 25.0 kN/m³ Assumed footing length L = 1000.0 mm Assumed footing width B = 1000.0 mm Assumed footing depth h = 300.0 mm Weight of footing: W_b = L × B × h × γ $W_b = \frac{1000.0}{1000} \times \frac{1000.0}{1000} \times \frac{300.0}{1000} \times 25.0 = 7.5kN$ Total service load $N_{s,total} = 650.0 + 7.5 = 657.5kN$ Footing area required $A_{req} = \frac{N_{s,total}}{q_s} = \frac{657.5}{200.0} = 3.29m^2$ For square footings: A = L² $L = \sqrt{A} = \sqrt{3.29} = 1.81m$ Required width B = required length L = 1.81 m Adopted L = 1000.0 mm, B = 1000.0 mm Area provided A_{prov} = L × B = 1 × 1 = 1 m² Area check: INCREASE SIZE	**A_{req} = 3.29 m²** **L_{req} = 1.81 m** **B_{req} = 1.81 m** **A_{prov} = 1 m²** **Status: INCREASE SIZE**

REFERENCE	CALCULATIONS	OUTPUT
§6.4.3 Ultimate Analysis	**ULTIMATE ANALYSIS**  **Ultimate load N_{Ed}** $N_{Ed} = 1.35G_k + 1.5Q_k$ $N_{Ed} = 1.35 \times 500.0 + 1.5 \times 150.0 = 900.0kN$ **Bearing pressure q_u** $q_u = \frac{N_{Ed}}{A_{prov}} = \frac{900.0}{1} = 900kN/m^2$ **Design moments M_{Ed} (per metre width, at the column face)** Bending along the length L $y = \frac{L}{2} - \frac{b_c}{2} = \frac{1000.0}{2} - \frac{300.0}{2} = 350mm$ $M_{Ed,y} = \frac{q_u y^2}{2} = \frac{900 \times 350^2}{2 \times 10^6} = 55.12kNm$ Bending along the width B $z = \frac{B}{2} - \frac{h_c}{2} = \frac{1000.0}{2} - \frac{300.0}{2} = 350mm$ $M_{Ed,z} = \frac{q_u z^2}{2} = \frac{900 \times 350^2}{2 \times 10^6} = 55.12kNm$	**N_{Ed} = 900.0 kN** **q_u = 900 kN/m²** **$M_{Ed,y}$ = 55.12 kNm** **$M_{Ed,z}$ = 55.12 kNm**
§9.2.1.1 Reinforcement Design	**REINFORCEMENT (B1)** **Reinforcement design, bottom layer (B1)** Concrete cover c_c = 50.0 mm Bar diameter φ = 16.0 mm $d = h - c_c - \frac{\phi}{2} = 300.0 - 50.0 - \frac{16.0}{2} = 242mm$ f_{ck} = 25.0 N/mm², f_{yk} = 500.0 N/mm², design strip b = 1000 mm In footings there is no moment redistribution ($f_{ck} \leq$ C50/60) $k_{bal} = 0.167$ $k = \frac{M}{bd^2 f_{ck}} = \frac{55.12 \times 10^6}{1000 \times 242^2 \times 25.0} = 0.038$ Since k = 0.038 < k_{bal} = 0.167 → singly reinforced $\frac{z}{d} = 0.5 + \sqrt{0.25 - \frac{k}{1.134}} = 0.966$ $z = 0.966 \times 242 = 234mm$ $z_{lim} = 0.95 \times 242 = 230mm$ Lever arm z = min(z, z_{lim}) = 230 mm **Reinforcement area required** $A_{s,req} = \frac{M \times 10^6}{0.87 f_{yk} z} = \frac{55.12 \times 10^6}{0.87 \times 500.0 \times 230} = 551mm^2/m$ **Minimum reinforcement** $f_{ctm} = 0.3 f_{ck}^{2/3} = 2.56N/mm^2$ $A_{s,min,req} = 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \times \frac{2.56}{500.0} \times 1000 \times 242 = 323mm^2/m$	**k = 0.038 < k_bal ✓** **$A_{s,req}$ = 551 mm²/m** **$A_{s,prov}$ = 1005 mm²/m** **Status: OK** **f_s = 169.97 N/mm²** **S_{max} = 287 mm** **S_{prov} = 200.0 mm** **Crack control: OK**

REFERENCE	CALCULATIONS	OUTPUT

REINFORCEMENT (B1) *(continued)*

0.0013 b d = 0.0013 × 1000 × 242 = 315 mm²/m

$A_{s,min}$ = max(323, 315) = 323 mm²/m

$A_{s,req}$ = max(323, 551) = 551 mm²/m (limit 4% A_c = 9680 mm²/m)

Provide reinforcement

Spacing S = 200.0 mm

$$A_{s,prov} = \pi\left(\frac{\phi}{2}\right)^2 \times \frac{1000}{S} = \pi\left(\frac{16.0}{2}\right)^2 \times \frac{1000}{200.0} = 1005 mm^2/m$$

Number of bars across B = B/S = 1000.0/200.0 → 5 bars

$$A_{s,prov} = 7 \times \pi\left(\frac{16.0}{2}\right)^2 = 1407 mm^2$$

Provided: 7 H16.0 @ 200.0 mm (B1)

Check: 1005 ≥ 551 mm²/m → OK

SERVICEABILITY LIMIT STATE (SLS): CRACK WIDTH / SPACING VERIFICATION

Table 7.2N Maximum bar diameters ϕ^*_s for crack control[1]

Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 7.3N Maximum bar spacing for crack control[1]

Steel stress[2] [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Steel stress under service load

REINFORCEMENT (B1) *(continued)*

$$f_s = 310 \times \frac{500}{f_{yk}} \times \frac{A_{s,req}}{A_{s,prov}} = 310 \times \frac{500}{500.0} \times \frac{551}{1005} = 169.97 N/mm^2$$

Maximum allowable spacing (EC2 Table 7.3N)

Stress thresholds: [160, 200, 240, 280, 320, 360] MPa

Spacing limits: [300, 250, 200, 150, 100, 50] mm

Interpolated for σ_s = 169.97 N/mm² → $S_{max,table}$ = 287 mm

Detailing limit (EC2 9.3.1.1(3)): min(3h, 400) = min(3 × 300.0, 400) = 400 mm

S_{max} = min(287, 400) = 287 mm

Provided spacing S = 200.0 mm, which is ≤ S_{max} = 287 mm

PASS: Provided bar spacing is adequate for crack control limit (wk = 0.3mm)

REFERENCE	CALCULATIONS	OUTPUT
§9.2.1.1 Reinforcement Design	**REINFORCEMENT (B2)** **Reinforcement design, bottom layer (B2)** Concrete cover c_c = 50.0 mm Bar diameter φ = 12.0 mm $d = h - c_c - \phi_{main} - \frac{\phi}{2} = 300.0 - 50.0 - 16.0 - \frac{12.0}{2} = 228mm$ f_{ck} = 25.0 N/mm², f_{yk} = 500.0 N/mm², design strip b = 1000 mm In footings there is no moment redistribution (f_{ck} ≤ C50/60) $k_{bal} = 0.167$ $k = \frac{M}{bd^2 f_{ck}} = \frac{55.12 \times 10^6}{1000 \times 228^2 \times 25.0} = 0.042$ Since k = 0.042 < k_{bal} = 0.167 → singly reinforced $\frac{z}{d} = 0.5 + \sqrt{0.25 - \frac{k}{1.134}} = 0.961$ $z = 0.961 \times 228 = 219mm$ $z_{lim} = 0.95 \times 228 = 217mm$ Lever arm z = min(z, z_{lim}) = 217 mm **Reinforcement area required** $A_{s,req} = \frac{M \times 10^6}{0.87 f_{yk} z} = \frac{55.12 \times 10^6}{0.87 \times 500.0 \times 217} = 585mm^2/m$ **Minimum reinforcement** $f_{ctm} = 0.3 f_{ck}^{2/3} = 2.56N/mm^2$ $A_{s,min,req} = 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \times \frac{2.56}{500.0} \times 1000 \times 228 = 304mm^2/m$	**k = 0.042 < k_bal ✓** **$A_{s,req}$ = 585 mm²/m** **$A_{s,prov}$ = 565 mm²/m** **Status: INCREASE REINFORCEMENT** **f_s = 320.73 N/mm²** **S_{max} = 99 mm** **S_{prov} = 200.0 mm** **Crack control: FAIL**

REFERENCE	CALCULATIONS	OUTPUT

REINFORCEMENT (B2) *(continued)*

0.0013 b d = 0.0013 × 1000 × 228 = 296 mm²/m

$A_{s,min}$ = max(304, 296) = 304 mm²/m

$A_{s,req}$ = max(304, 585) = 585 mm²/m (limit 4% A_c = 9120 mm²/m)

Provide reinforcement

Spacing S = 200.0 mm

$$A_{s,prov} = \pi\left(\frac{\phi}{2}\right)^2 \times \frac{1000}{S} = \pi\left(\frac{12.0}{2}\right)^2 \times \frac{1000}{200.0} = 565 mm^2/m$$

Number of bars across L = L/S = 1000.0/200.0 → 5 bars

$$A_{s,prov} = 7 \times \pi\left(\frac{12.0}{2}\right)^2 = 792 mm^2$$

Provided: 7 H12.0 @ 200.0 mm (B2)

Check: 565 < 585 mm²/m → INCREASE REINFORCEMENT

SERVICEABILITY LIMIT STATE (SLS): CRACK WIDTH / SPACING VERIFICATION

Table 7.2N Maximum bar diameters ϕ^*_s for crack control[1]

Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 7.3N Maximum bar spacing for crack control[1]

Steel stress[2] [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Steel stress under service load

REINFORCEMENT (B2) *(continued)*

$$f_s = 310 \times \frac{500}{f_{yk}} \times \frac{A_{s,req}}{A_{s,prov}} = 310 \times \frac{500}{500.0} \times \frac{585}{565} = 320.73 N/mm^2$$

Maximum allowable spacing (EC2 Table 7.3N)

Stress thresholds: [160, 200, 240, 280, 320, 360] MPa

Spacing limits: [300, 250, 200, 150, 100, 50] mm

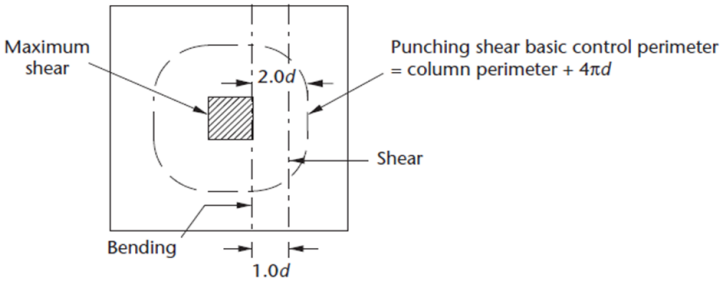
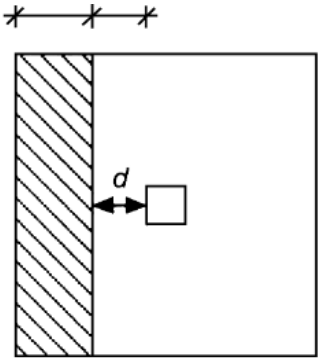
Interpolated for σ_s = 320.73 N/mm² → $S_{max,table}$ = 99 mm

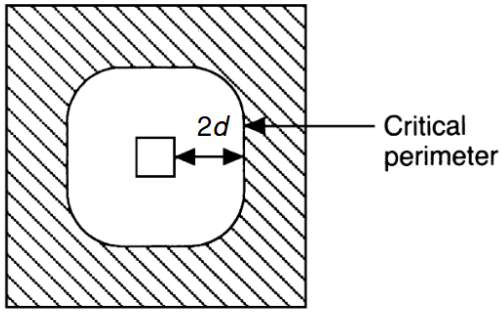
Detailing limit (EC2 9.3.1.1(3)): min(3h, 400) = min(3 × 300.0, 400) = 400 mm

S_{max} = min(99, 400) = 99 mm

Provided spacing S = 200.0 mm, which is > S_{max} = 99 mm

FAIL: Bar spacing exceeds the allowable crack control threshold. Reduce spacing or increase rebar size.

REFERENCE	CALCULATIONS	OUTPUT
§6.2.2 Shear Checks	**SHEAR CHECKS** For face and punching shear use the mean effective depth d = 235 mm **1. Face shear at the column face**  $u_0 = 2(b_c + h_c) = 2(300.0 + 300.0) = 1200mm$ $v_{Ed,face} = \beta\frac{N_{Ed}}{u_0 d} = 1.15 \times \frac{900.0 \times 10^3}{1200 \times 235} = 3.67N/mm^2$ $v_{Rd,max} = 0.5\left(0.6\left(1 - \frac{f_{ck}}{250}\right)\right)\frac{f_{ck}}{1.5} = 4.5N/mm^2$ Since $v_{Ed,face}$ = 3.67 N/mm² < $v_{Rd,max}$ = 4.5 N/mm² Therefore Footing Depth Adquate	**v_{Ed} = 3.67 N/mm²** **$v_{Rd,max}$ = 4.5 N/mm²** **Face shear: OK**
§6.2.2 Shear Checks	**SHEAR CHECKS** **2. Direct (one-way) shear at distance d from the column face**  **Shear failure across the length L** $X = \frac{L}{2} - \frac{b_c}{2} - d = \frac{1000.0}{2} - \frac{300.0}{2} - 242 = 108mm$ $p_E = \frac{N_{Ed}}{A} = \frac{900.0}{1} = 900kN/m^2$ $V_{Ed} = p_E \times B \times X = 900 \times 1 \times \frac{108}{1000} = 97.2kN$ $v_{Ed} = \frac{V_{Ed} \times 10^3}{B \times d} = \frac{97.2 \times 10^3}{1000.0 \times 242} = 0.402N/mm^2$ $k = 1 + \sqrt{\frac{200}{d}} = 1.91 \quad k = \min(1.91, 2.0) = 1.91$ $\rho_1 = \frac{A_{s,prov}}{bd} = \frac{1005}{1000 \times 242} = 0.00415 \quad \rho_1 = \min(0.00415, 0.02) = 0.00415$ $v_{Rd,c} = 0.12k(100\rho_1 f_{ck})^{1/3} = 0.12 \times 1.91 \times (100 \times 0.00415 \times 25.0)^{1/3} = 0.5N/mm^2$ $v_{Rd,c,min} = 0.035k^{3/2}\sqrt{f_{ck}} = 0.462N/mm^2$ **SHEAR CHECKS *(continued)*** $v_{Rd,c}$ = max(0.462, 0.5) = 0.5 N/mm² Since v_{Ed} = 0.402 N/mm² < $v_{Rd,c}$ = 0.5 N/mm² Therefore NO Shear reinforcement required	**$v_{Ed,L}$ = 0.402 N/mm²** **$v_{Rd,c,L}$ = 0.5 N/mm²** **Direct shear L: OK**

REFERENCE	CALCULATIONS	OUTPUT
§6.2.2 Shear Checks	**SHEAR CHECKS** **Shear failure across the width B** $X = \frac{B}{2} - \frac{h_c}{2} - d = \frac{1000.0}{2} - \frac{300.0}{2} - 228 = 122mm$ $p_E = \frac{N_{Ed}}{A} = \frac{900.0}{1} = 900kN/m^2$ $V_{Ed} = p_E \times L \times X = 900 \times 1 \times \frac{122}{1000} = 109.8kN$ $v_{Ed} = \frac{V_{Ed} \times 10^3}{L \times d} = \frac{109.8 \times 10^3}{1000.0 \times 228} = 0.482N/mm^2$ $k = 1 + \sqrt{\frac{200}{d}} = 1.94 \quad k = \min(1.94, 2.0) = 1.94$ $\rho_1 = \frac{A_{s,prov}}{bd} = \frac{565}{1000 \times 228} = 0.00248 \quad \rho_1 = \min(0.00248, 0.02) = 0.00248$ $v_{Rd,c} = 0.12k(100\rho_1 f_{ck})^{1/3} = 0.12 \times 1.94 \times (100 \times 0.00248 \times 25.0)^{1/3} = 0.427N/mm^2$ $v_{Rd,c,min} = 0.035k^{3/2}\sqrt{f_{ck}} = 0.472N/mm^2$ $v_{Rd,c}$ = max(0.472, 0.427) = 0.472 N/mm² Since v_{Ed} = 0.482 N/mm² > $v_{Rd,c}$ = 0.472 N/mm² Therefore Shear reinforcement is required, or increase footing depth	**$v_{Ed,B}$ = 0.482 N/mm²** **$v_{Rd,c,B}$ = 0.472 N/mm²** **Direct shear B: FAIL**
§6.4.4 Punching Shear	**PUNCHING SHEAR** **3. Punching shear at 2d from the column face**  $u_1 = u_0 + 4\pi d = 1200 + 4 \times \pi \times 235 = 4153mm$ $A_p = b_c h_c + 4d(b_c + h_c) + 4\pi d^2 = 1347978mm^2 = 1.35m^2$ Control perimeter fits in the base only if L ≥ b_c + 4d and B ≥ h_c + 4d: 1000.0 vs 1240 and 1000.0 vs 1240 mm **Punching shear cannot be checked, since the basic control perimeter 2d from the column face lies outside the base area** **Cannot verify punching shear - control perimeter exceeds footing extents; increase footing plan size or depth**	**u_1 = 4153 mm** **A_p = 1347978 mm²** **Punching shear cannot be checked**