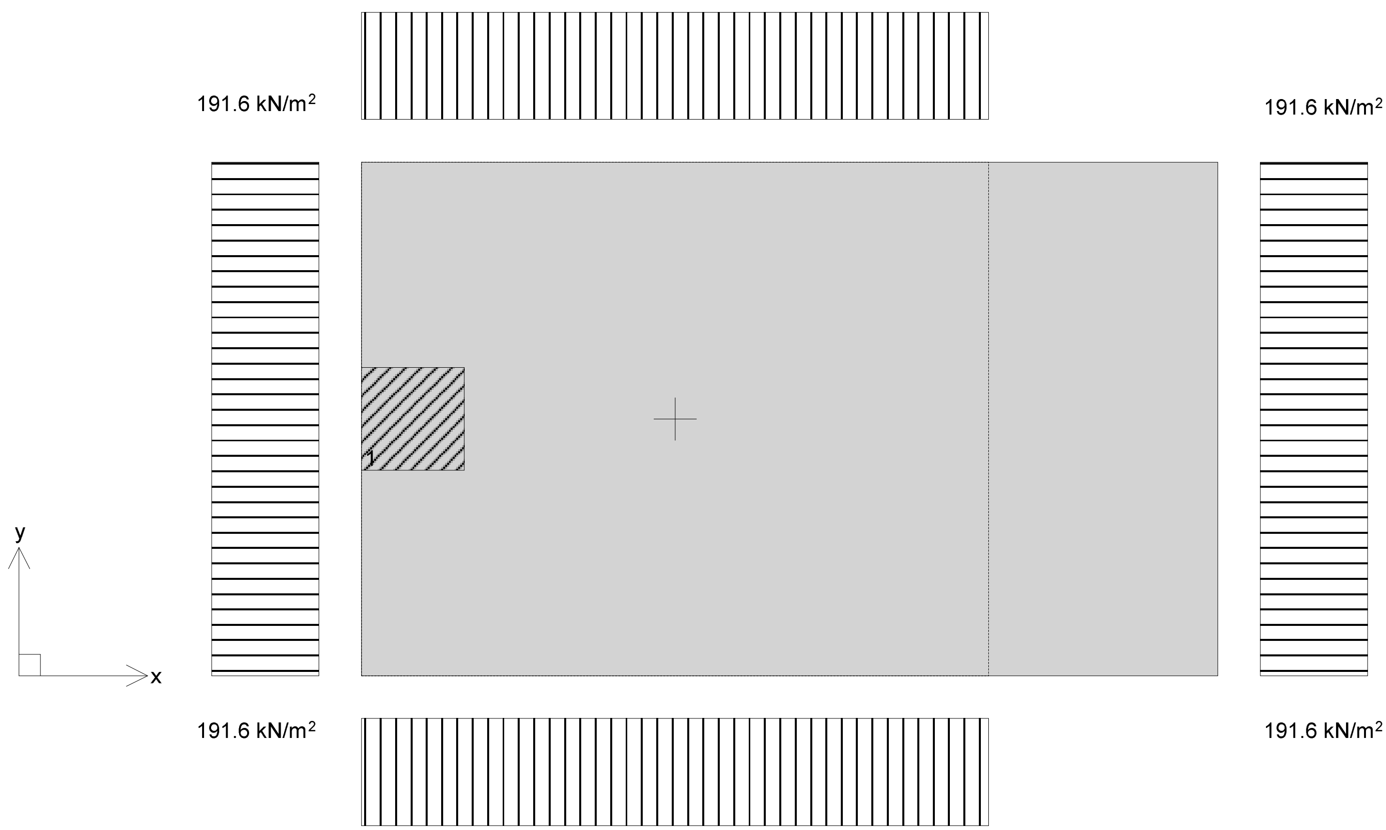
FOUNDATION ANALYSIS (EN1997-1:2004)

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the recommended values

Pad foundation details

Length of foundation;	L_x = **2500** mm
Width of foundation;	L_y = **1500** mm
Foundation area;	$A = L_x \times L_y$ = **3.750** m^2
Depth of foundation;	h = **500** mm
Depth of soil over foundation;	h_{soil} = **950** mm
Level of water;	h_{water} = **150** mm
Density of water;	γ_{water} = **9.8** kN/m^3
Density of concrete;	γ_{conc} = **24.5** kN/m^3



Column no.1 details

Length of column; l_{x1} = **300** mm

Width of column; l_{y1} = **300** mm

position in x-axis; x_1 = **150** mm

position in y-axis; y_1 = **750** mm

Soil properties

Density of soil; γ_{soil} = **20.0** kN/m^3

Characteristic cohesion; c'_k = **0** kN/m^2

Characteristic effective shear resistance angle; ϕ'_k = **25** deg

Characteristic friction angle; δ_k = **19.3** deg

Foundation loads

Self weight; $F_{swt} = h \times \gamma_{conc} - h_{water} \times \gamma_{water}$ = **10.8** kN/m^2

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **19.0** kN/m^2

Column no.1 loads

Permanent load in z; F_{Gz1} = **200.0** kN

Variable load in z; F_{Qz1} = **165.0** kN

Permanent moment in x; M_{Gx1} = **150.0** kNm

Variable moment in x; M_{Qx1} = **100.0** kNm

Partial factors on actions - Combination1

Permanent unfavourable action - Table A.3; γ_G = **1.35**

Permanent favourable action - Table A.3; γ_{Gf} = **1.00**

Variable unfavourable action - Table A.3; γ_Q = **1.50**

Variable favourable action - Table A.3; γ_{Qf} = **0.00**

Partial factors for soil parameters - Combination1

Angle of shearing resistance - Table A.4; $\gamma_{\phi'}$ = **1.00**

Effective cohesion - Table A.4; $\gamma_{c'}$ = **1.00**

Weight density - Table A.4; γ_γ = **1.00**

Partial factors for spread foundations - Combination1

Bearing - Table A.5; $\gamma_{R.v}$ = **1.00**

Sliding - Table A.5; $\gamma_{R.h}$ = **1.00**

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1}$ = **668.3** kN

Moments on foundation

Moment in x-axis; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1}$ = **618.6** kNm

Moment in y-axis; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1$ = **501.2** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2$ = **-324** mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Effective area of base

Effective length; $L'_x = L_x + 2 \times e_x$ = **1851** mm

Effective width; $L'_y = L_y - 2 \times e_y$ = **1500** mm

Effective area; $A' = L'_x \times L'_y$ = **2.777** m^2

Pad base pressure

Design base pressure; $f_{dz} = F_{dz} / A'$ = **240.6** kN/m^2

Net ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance; $\phi'_d = \text{atan}(\tan(\phi'_k) / \gamma_{\phi'})$ = **25.000** deg

Design effective cohesion; $c'_d = c'_k / \gamma_{c'}$ = **0.000** kN/m^2

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water}$ = **27.529** kN/m^2

Design effective overburden pressure; $q' = q / \gamma_\gamma$ = **27.529** kN/m^2

Bearing resistance factors; $N_q = \text{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45 \text{ deg} + \phi'_d / 2))^2$ = **10.662**

$N_c = (N_q - 1) \times \cot(\phi'_d)$ = **20.721**

$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_d)$ = **9.011**

Foundation shape factors; $s_q = 1 + (L'_y / L'_x) \times \sin(\phi'_d)$ = **1.342**

$s_\gamma = 1 - 0.3 \times (L'_y / L'_x)$ = **0.757**

$s_c = (s_q \times N_q - 1) / (N_q - 1)$ = **1.378**

Load inclination factors; H = **0.0** kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)]$ = **1.552**

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)]$ = **1.448**

$m = m_x$ = **1.448**

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m$ = **1.000**

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1}$ = **1.000**

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d))$ = **1.000**

Net ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_{soil} - \gamma_{water}) \times L'_y \times N_\gamma \times s_\gamma \times i_\gamma$ = **446.1** kN/m^2

PASS - Net ultimate bearing capacity exceeds design base pressure

Partial factors on actions - Combination2

Permanent unfavourable action - Table A.3; γ_G = **1.00**

Permanent favourable action - Table A.3; γ_{Gf} = **1.00**

Variable unfavourable action - Table A.3; γ_Q = **1.30**

Variable favourable action - Table A.3; γ_{Qf} = **0.00**

Partial factors for soil parameters - Combination2

Angle of shearing resistance - Table A.4; $\gamma_{\phi'}$ = **1.25**

Effective cohesion - Table A.4; $\gamma_{c'}$ = **1.25**

Weight density - Table A.4; γ_{γ} = **1.00**

Partial factors for spread foundations - Combination2

Bearing - Table A.5; $\gamma_{R.v}$ = **1.00**

Sliding - Table A.5; $\gamma_{R.h}$ = **1.00**

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1}$ = **526.2** kN

Moments on foundation

Moment in x-axis; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1}$ = **481.8** kNm

Moment in y-axis; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1$ = **394.6** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2$ = **-334** mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Effective area of base

Effective length; $L'_x = L_x + 2 \times e_x$ = **1831** mm

Effective width; $L'_y = L_y - 2 \times e_y$ = **1500** mm

Effective area; $A' = L'_x \times L'_y$ = **2.747** m^2

Pad base pressure

Design base pressure; $f_{dz} = F_{dz} / A'$ = **191.6** kN/m^2

Net ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance; $\phi'_d = \text{atan}(\tan(\phi'_k) / \gamma_{\phi'})$ = **20.458** deg

Design effective cohesion; $c'_d = c'_k / \gamma_{c'}$ = **0.000** kN/m^2

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water}$ = **27.529** kN/m^2

Design effective overburden pressure; $q' = q / \gamma_{\gamma}$ = **27.529** kN/m^2

Bearing resistance factors; $N_q = \text{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45\text{ deg} + \phi'_d / 2))^2$ = **6.698**

$N_c = (N_q - 1) \times \cot(\phi'_d)$ = **15.273**

$N_{\gamma} = 2 \times (N_q - 1) \times \tan(\phi'_d)$ = **4.251**

Foundation shape factors; $s_q = 1 + (L'_y / L'_x) \times \sin(\phi'_d)$ = **1.286**

$s_{\gamma} = 1 - 0.3 \times (L'_y / L'_x)$ = **0.754**

$s_c = (s_q \times N_q - 1) / (N_q - 1)$ = **1.337**

Load inclination factors; H = **0.0** kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)] = \underline{\mathbf{1.550}}$

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)] = \underline{\mathbf{1.450}}$

$m = m_x = \underline{\mathbf{1.450}}$

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m = \underline{\mathbf{1.000}}$

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1} = \underline{\mathbf{1.000}}$

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d)) = \underline{\mathbf{1.000}}$

Net ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_{soil} - \gamma_{water}) \times L'_y \times N_\gamma \times s_\gamma \times i_\gamma = \underline{\mathbf{261.7}}$ kN/m^2

PASS - Net ultimate bearing capacity exceeds design base pressure

FOUNDATION DESIGN (EN1992-1-1:2004)

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the recommended values

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class; C40/50

Characteristic compressive cylinder strength; $f_{ck} = \underline{\mathbf{40}}$ N/mm^2

Characteristic compressive cube strength; $f_{ck,cube} = \underline{\mathbf{50}}$ N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 $= \underline{\mathbf{48}}$ N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3} = \underline{\mathbf{3.5}}$ N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = \underline{\mathbf{2.5}}$ N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3} = \underline{\mathbf{35220}}$ N/mm^2

Partial factor for concrete (Table 2.1N); $\gamma_C = \underline{\mathbf{1.50}}$

Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = \underline{\mathbf{1.00}}$

Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \underline{\mathbf{26.7}}$ N/mm^2

Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = \underline{\mathbf{0.80}}$

Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = \underline{\mathbf{1.3}}$ N/mm^2

Maximum aggregate size; $h_{agg} = \underline{\mathbf{20}}$ mm

Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} = \underline{\mathbf{500}}$ N/mm^2

Modulus of elasticity of reinforcement; $E_s = \underline{\mathbf{210000}}$ N/mm^2

Partial factor for reinforcing steel (Table 2.1N); $\gamma_S = \underline{\mathbf{1.15}}$

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = \underline{\mathbf{435}}$ N/mm^2

Nominal cover to reinforcement; $c_{nom} = \underline{\mathbf{30}}$ mm

Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.x.max} = \underline{\mathbf{288.4}}$ kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2 = \underline{\mathbf{462}}$ mm

$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck}) = \underline{\mathbf{0.023}}$

$K' = \underline{\mathbf{0.207}}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) =$ **439** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **58** mm

Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z) =$ **1511** mm^2

Tension reinforcement provided; 8 No.16 dia.bars bottom (225 c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **1608** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d =$ **1264** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d =$ **27720** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 =$ **0.3**

Serviceability bending moment; $M_{sls.x.max} =$ **148.8** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z) =$ **210.8** N/mm^2

Load duration factor; $k_t =$ **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) =$ **95** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y =$ **142500** mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.5** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff} =$ **0.011**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.962**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 =$ **3.4**

$k_4 =$ **0.425**

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff} =$ **343** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$ $0.6 \times \sigma_s / E_s) =$ **0.207** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in flexure (Section 6.1)**

Design bending moment; $abs(M_{Ed.x.min}) =$ **15.2** kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{y.top} - \phi_{x.top} / 2 =$ **442** mm

$K = abs(M_{Ed.x.min}) / (L_y \times d^2 \times f_{ck}) =$ **0.001**

$K' =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) =$ **420** mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \underline{\mathbf{55}}$ mm

Area of tension reinforcement required; $A_{sx.top.req} = abs(M_{Ed.x.min}) / (f_{yd} \times z) = \underline{\mathbf{83}}$ mm^2

Tension reinforcement provided; 7 No.16 dia.bars top (250 c/c)

Area of tension reinforcement provided; $A_{sx.top.prov} = \underline{\mathbf{1407}}$ mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d = \underline{\mathbf{1210}}$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d = \underline{\mathbf{26520}}$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; $w_{max} = \underline{\mathbf{0.3}}$ mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 = \underline{\mathbf{0.3}}$

Serviceability bending moment; $abs(M_{sls.x.min}) = \underline{\mathbf{8.1}}$ kNm

Tensile stress in reinforcement; $\sigma_s = abs(M_{sls.x.min}) / (A_{sx.top.prov} \times z) = \underline{\mathbf{13.7}}$ N/mm^2

Load duration factor; $k_t = \underline{\mathbf{0.4}}$

Effective depth of concrete in tension; $h_{c.ef} = min(2.5 \times (h - d), (h - x) / 3, h / 2) = \underline{\mathbf{145}}$ mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y = \underline{\mathbf{217500}}$ mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \underline{\mathbf{3.5}}$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sx.top.prov} / A_{c.eff} = \underline{\mathbf{0.006}}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \underline{\mathbf{5.962}}$

Bond property coefficient; $k_1 = \underline{\mathbf{0.8}}$

Strain distribution coefficient; $k_2 = \underline{\mathbf{0.5}}$

$k_3 = \underline{\mathbf{3.4}}$

$k_4 = \underline{\mathbf{0.425}}$

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{y.top}) + k_1 \times k_2 \times k_4 \times \phi_{x.top} / \rho_{p.eff}$ $= \underline{\mathbf{590}}$ mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$ $0.6 \times \sigma_s / E_s) = \underline{\mathbf{0.023}}$ mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear (Section 6.2)

Design shear force; $abs(V_{Ed.x.min}) = \underline{\mathbf{293.5}}$ kN

$C_{Rd,c} = 0.18 / \gamma_C = \underline{\mathbf{0.120}}$

$k = min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \underline{\mathbf{1.673}}$

Longitudinal reinforcement ratio; $\rho_l = min(A_{sx.bot.prov} / (L_y \times d), 0.02) = \underline{\mathbf{0.002}}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \underline{\mathbf{0.479}}$ N/mm^2

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$

$V_{Rd.c} = \underline{\mathbf{317.5}}$ kN

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.y.max}$ = **62.1** kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **446** mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck})$ = **0.003**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ =
424 mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **56** mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z)$ = **337** mm^2

Tension reinforcement provided; 11 No.16 dia.bars bottom (250 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **2212** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d$ = **2034**
mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d$ = **44600** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; w_{max} = **0.3** mm

Variable load factor (EN1990 – Table A1.1); ψ_2 = **0.3**

Serviceability bending moment; $M_{sls.y.max}$ = **29.9** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z)$ = **31.9** N/mm^2

Load duration factor; k_t = **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **135** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x$ = **337500** mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **3.5** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff}$ = **0.007**

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **5.962**

Bond property coefficient; k_1 = **0.8**

Strain distribution coefficient; k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff}$
= **571** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$
$0.6 \times \sigma_s / E_s)$ = **0.052** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear (Section 6.2)**

Design shear force; $V_{Ed.y.max}$ = **53.5** kN

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \underline{\mathbf{1.659}}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sy.bot.prov} / (L_x \times d), 0.02) = \underline{\mathbf{0.002}}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \underline{\mathbf{0.473}}\ N/mm^2$

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$

$V_{Rd.c} = \underline{\mathbf{544.1}}\ kN$

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250\ N/mm^2] = \underline{\mathbf{0.504}}$

Average depth to reinforcement; $d = \underline{\mathbf{454}}\ mm$

Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} = \underline{\mathbf{6.720}}\ N/mm^2$

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \underline{\mathbf{1.664}}$

Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) = \underline{\mathbf{0.002}}$

$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) = \underline{\mathbf{0.002}}$

$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = \underline{\mathbf{0.002}}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \underline{\mathbf{0.475}}\ N/mm^2$

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) = \underline{\mathbf{0.475}}\ N/mm^2$

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = \underline{\mathbf{900}}\ mm$

Area within punching shear perimeter; $A_0 = \underline{\mathbf{0.090}}\ m^2$

Maximum punching shear force; $V_{Ed.max} = \underline{\mathbf{499.5}}\ kN$

Punching shear stress factor (fig 6.21N); $\beta = \underline{\mathbf{1.500}}$

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = \underline{\mathbf{1.834}}\ N/mm^2$

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Column No.1 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = \underline{\mathbf{1611}}\ mm$

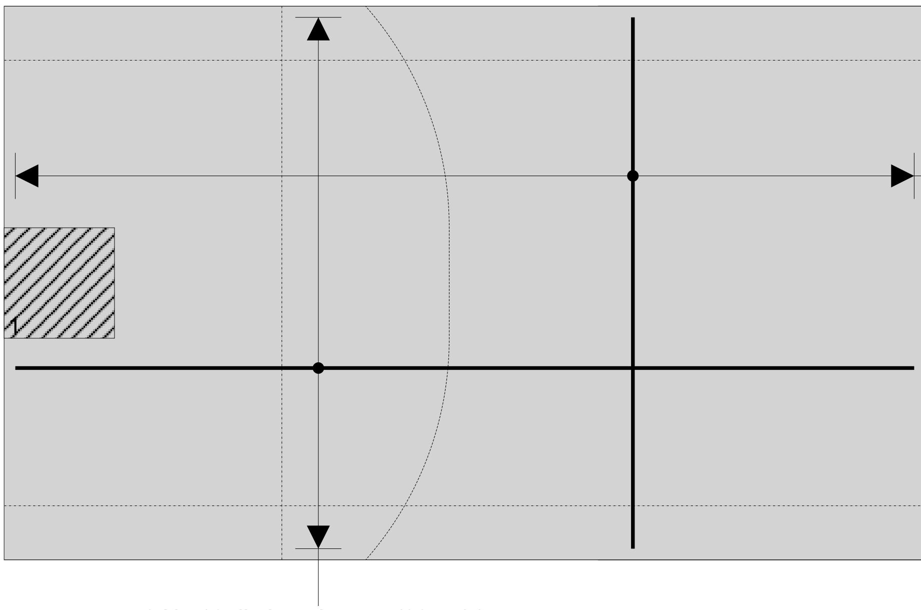
Area within punching shear perimeter; $A_2 = \underline{\mathbf{1.726}}\ m^2$

Design punching shear force; $V_{Ed.2} = \underline{\mathbf{171.4}}\ kN$

Punching shear stress factor (fig 6.21N); $\beta = \underline{\mathbf{1.500}}$

Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = \underline{\mathbf{0.352}}\ N/mm^2$

PASS - Design punching shear resistance exceeds design punching shear stress



1
11 No.16 dia.bars bottom (250 c/c)
8 No.16 dia.bars bottom (225 c/c)
7 No.16 dia.bars top (250 c/c)