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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

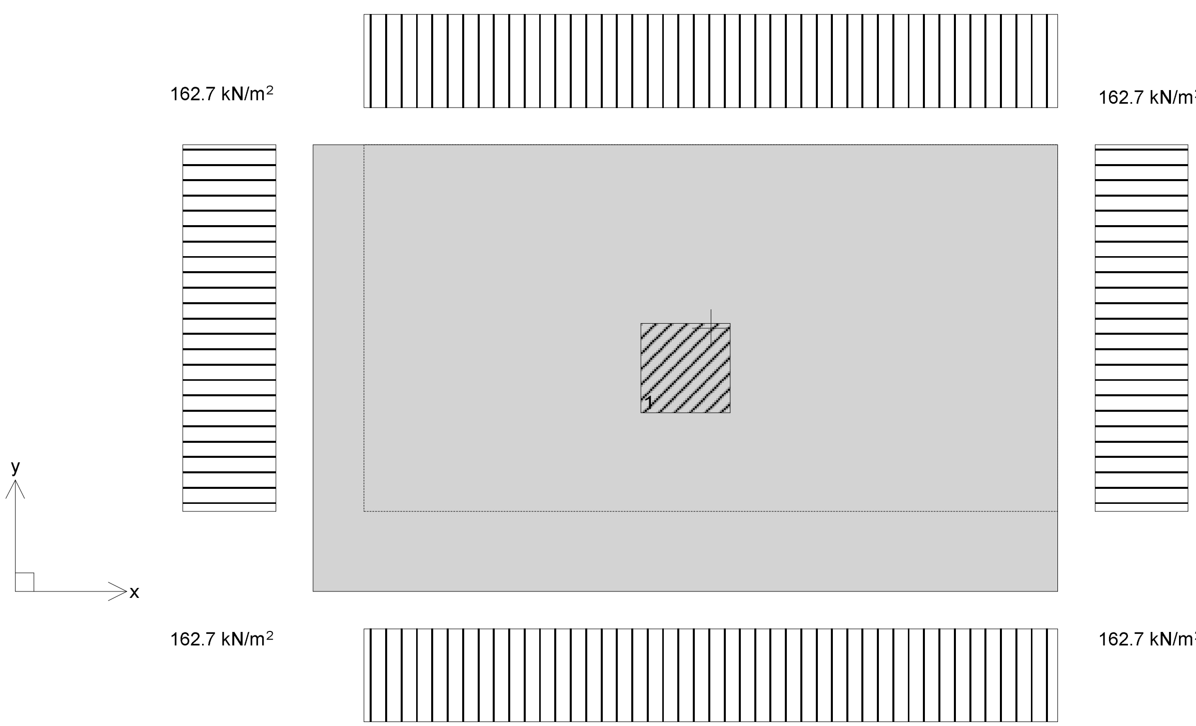
Depth of foundation; h = **400** mm

Depth of soil over foundation; h_{soil} = **200** mm

Level of water; h_{water} = **0** 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 = **1250** mm

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

Soil properties

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

Characteristic cohesion; $c'_k = \underline{\mathbf{0}}$ kN/m^2

Characteristic effective shear resistance angle; $\phi'_k = \underline{\mathbf{29}}$ deg

Characteristic friction angle; $\delta_k = \underline{\mathbf{22}}$ deg

Foundation loads

Self weight; $F_{swt} = h \times \gamma_{conc} = \underline{\mathbf{9.8}}$ kN/m^2

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil} = \underline{\mathbf{4.0}}$ kN/m^2

Column no.1 loads

Permanent load in x; $F_{Gx1} = \underline{\mathbf{10.0}}$ kN

Permanent load in y; $F_{Gy1} = \underline{\mathbf{5.0}}$ kN

Permanent load in z; $F_{Gz1} = \underline{\mathbf{200.0}}$ kN

Variable load in x; $F_{Qx1} = \underline{\mathbf{15.0}}$ kN

Variable load in y; $F_{Qy1} = \underline{\mathbf{20.0}}$ kN

Variable load in z; $F_{Qz1} = \underline{\mathbf{165.0}}$ kN

Permanent moment in x; $M_{Gx1} = \underline{\mathbf{15.0}}$ kNm

Permanent moment in y; $M_{Gy1} = \underline{\mathbf{40.0}}$ kNm

Variable moment in x; $M_{Qx1} = \underline{\mathbf{10.0}}$ kNm

Variable moment in y; $M_{Qy1} = \underline{\mathbf{8.0}}$ kNm

Partial factors on actions - Combination1

Permanent unfavourable action - Table A.3; $\gamma_G = \underline{\mathbf{1.35}}$

Permanent favourable action - Table A.3; $\gamma_{Gf} = \underline{\mathbf{1.00}}$

Variable unfavourable action - Table A.3; $\gamma_Q = \underline{\mathbf{1.50}}$

Variable favourable action - Table A.3; $\gamma_{Qf} = \underline{\mathbf{0.00}}$

Partial factors for soil parameters - Combination1

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

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

Weight density - Table A.4; $\gamma_\gamma = \underline{\mathbf{1.00}}$

Partial factors for spread foundations - Combination1

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

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

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in x-axis; $F_{dx} = \gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1} = \underline{\mathbf{36.0}}$ kN

Force in y-axis; $F_{dy} = \gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1} = \underline{\mathbf{36.8}}$ kN

Force in z-axis; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} = \underline{\mathbf{587.4}}$ kN

Moments on foundation

Moment in x-axis; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1} + (\gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1}) \times h = \underline{\mathbf{783.9}}$ kNm

Moment in y-axis; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_G \times M_{Gy1} + \gamma_Q \times F_{Qz1} \times y_1 + \gamma_Q \times M_{Qy1} + (\gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1}) \times h$ = **521.2** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2$ = **85** mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2$ = **137** mm

Effective area of base

Effective length; $L'_x = L_x - 2 \times e_x$ = **2331** mm

Effective width; $L'_y = L_y - 2 \times e_y$ = **1225** mm

Effective area; $A' = L'_x \times L'_y$ = **2.856** m^2

Pad base pressure

Design base pressure; $f_{dz} = F_{dz} / A'$ = **205.7** kN/m^2

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

Design angle of shearing resistance; $\phi'_d = atan(tan(\phi'_k) / \gamma_{\phi'})$ = **29.000** deg

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

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water}$ = **12.000** kN/m^2

Design effective overburden pressure; $q' = q / \gamma_{\gamma}$ = **12.000** kN/m^2

Bearing resistance factors; $N_q = Exp(\pi \times tan(\phi'_d)) \times (tan(45 deg + \phi'_d / 2))^2$ = **16.443**

$N_c = (N_q - 1) \times cot(\phi'_d)$ = **27.860**

$N_{\gamma} = 2 \times (N_q - 1) \times tan(\phi'_d)$ = **17.121**

Foundation shape factors; $s_q = 1 + (L'_y / L'_x) \times sin(\phi'_d)$ = **1.255**

$s_{\gamma} = 1 - 0.3 \times (L'_y / L'_x)$ = **0.842**

$s_c = (s_q \times N_q - 1) / (N_q - 1)$ = **1.271**

Load inclination factors; $H = [F_{dx}^2 + F_{dy}^2]^{0.5}$ = **51.4** kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)]$ = **1.655**

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)]$ = **1.345**

$m = m_x \times cos(atan(F_{dy} / F_{dx}))^2 + m_y \times sin(atan(F_{dy} / F_{dx}))^2$ = **1.503**

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times cot(\phi'_d))]^m$ = **0.871**

$i_{\gamma} = [1 - H / (F_{dz} + A' \times c'_d \times cot(\phi'_d))]^{m+1}$ = **0.795**

$i_c = i_q - (1 - i_q) / (N_c \times tan(\phi'_d))$ = **0.863**

Net ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_y \times N_{\gamma} \times s_{\gamma} \times i_{\gamma}$ = **356.2** kN/m^2

PASS - Net ultimate bearing capacity exceeds design base pressure

Sliding resistance (Section 6.5.3)

Forces on foundation

Force in x-axis; $F_{dx} = \gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1}$ = **36.0** kN

Force in y-axis; $F_{dy} = \gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1}$ = **36.8** kN

Force in z-axis; $F_{dz} = \gamma_{Gf} \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_{Qf} \times F_{Qz1} =$ **251.8** kN

Sliding resistance verification (Section 6.5.3)

Horizontal force on foundation; $H = [F_{dx}^2 + F_{dy}^2]^{0.5} =$ **51.4** kN

Sliding resistance (exp.6.3b); $R_{H.d} = F_{dz} \times \tan(\delta_k) / \gamma_{R.h} =$ **101.7** kN

PASS - Foundation is not subject to failure by sliding

Partial factors on actions - Combination2

Permanent unfavourable action - Table A.3; $\gamma_G =$ **1.00**

Permanent favourable action - Table A.3; $\gamma_{Gf} =$ **1.00**

Variable unfavourable action - Table A.3; $\gamma_Q =$ **1.30**

Variable favourable action - Table A.3; $\gamma_{Qf} =$ **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; $\gamma_\gamma =$ **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 x-axis; $F_{dx} = \gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1} =$ **29.5** kN

Force in y-axis; $F_{dy} = \gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1} =$ **31.0** kN

Force in z-axis; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} =$ **466.3** kN

Moments on foundation

Moment in x-axis; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1} + (\gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1}) \times h =$ **622.6** kNm

Moment in y-axis; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_G \times M_{Gy1} + \gamma_Q \times F_{Qz1} \times y_1 + \gamma_Q \times M_{Qy1} + (\gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1}) \times h =$ **412.5** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 =$ **85** mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 =$ **135** mm

Effective area of base

Effective length; $L'_x = L_x - 2 \times e_x =$ **2329** mm

Effective width; $L'_y = L_y - 2 \times e_y =$ **1231** mm

Effective area; $A' = L'_x \times L'_y =$ **2.866** m^2

Pad base pressure

Design base pressure; $f_{dz} = F_{dz} / A' = \underline{\mathbf{162.7}}$ kN/m^2

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

Design angle of shearing resistance; $\phi'_d = \text{atan}(\tan(\phi'_k) / \gamma_{\phi'}) = \underline{\mathbf{23.915}}$ deg

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

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water} = \underline{\mathbf{12.000}}$ kN/m^2

Design effective overburden pressure; $q' = q / \gamma_\gamma = \underline{\mathbf{12.000}}$ kN/m^2

Bearing resistance factors; $N_q = \text{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45 \text{ deg} + \phi'_d / 2))^2 = \underline{\mathbf{9.519}}$

$N_c = (N_q - 1) \times \cot(\phi'_d) = \underline{\mathbf{19.210}}$

$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_d) = \underline{\mathbf{7.555}}$

Foundation shape factors; $s_q = 1 + (L'_y / L'_x) \times \sin(\phi'_d) = \underline{\mathbf{1.214}}$

$s_\gamma = 1 - 0.3 \times (L'_y / L'_x) = \underline{\mathbf{0.842}}$

$s_c = (s_q \times N_q - 1) / (N_q - 1) = \underline{\mathbf{1.239}}$

Load inclination factors; $H = [F_{dx}^2 + F_{dy}^2]^{0.5} = \underline{\mathbf{42.8}}$ kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)] = \underline{\mathbf{1.654}}$

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)] = \underline{\mathbf{1.346}}$

$m = m_x \times \cos(\text{atan}(F_{dy} / F_{dx}))^2 + m_y \times \sin(\text{atan}(F_{dy} / F_{dx}))^2 = \underline{\mathbf{1.508}}$

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m = \underline{\mathbf{0.865}}$

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1} = \underline{\mathbf{0.786}}$

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d)) = \underline{\mathbf{0.849}}$

Net ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_y \times N_\gamma \times s_\gamma \times i_\gamma = \underline{\mathbf{181.4}}$ kN/m^2

PASS - Net ultimate bearing capacity exceeds design base pressure

Sliding resistance (Section 6.5.3)

Forces on foundation

Force in x-axis; $F_{dx} = \gamma_G \times F_{Gx1} + \gamma_Q \times F_{Qx1} = \underline{\mathbf{29.5}}$ kN

Force in y-axis; $F_{dy} = \gamma_G \times F_{Gy1} + \gamma_Q \times F_{Qy1} = \underline{\mathbf{31.0}}$ kN

Force in z-axis; $F_{dz} = \gamma_{Gf} \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_{Qf} \times F_{Qz1} = \underline{\mathbf{251.8}}$ kN

Sliding resistance verification (Section 6.5.3)

Horizontal force on foundation; $H = [F_{dx}^2 + F_{dy}^2]^{0.5} = \underline{\mathbf{42.8}}$ kN

Sliding resistance (exp.6.3b); $R_{H.d} = F_{dz} \times \tan(\delta_k) / \gamma_{R.h} = \underline{\mathbf{101.7}}$ kN

PASS - Foundation is not subject to failure by sliding

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; C30/37

Characteristic compressive cylinder strength; f_{ck} = **30** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **37** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8\ N/mm^2$ = **38** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck}/\ 1\ N/mm^2)^{2/3}$ = **2.9** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.0** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22\ kN/mm^2 \times [f_{cm}/10\ N/mm^2]^{0.3}$ = **32837** N/mm^2

Partial factor for concrete (Table 2.1N); γ_C = **1.50**

Compressive strength coefficient (cl.3.1.6(1)); α_{cc} = **1.00**

Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **20.0** N/mm^2

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

Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C$ = **1.1** N/mm^2

Maximum aggregate size; h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement; E_s = **210000** N/mm^2

Partial factor for reinforcing steel (Table 2.1N); γ_S = **1.15**

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

Nominal cover to reinforcement; c_{nom} = **30** mm

Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.x.max}$ = **135.5** kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2$ = **364** mm

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **346** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **45** mm

Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z)$ = **902** mm^2

Tension reinforcement provided; 8 No.12 dia.bars bottom (225 c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **905** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d$ = **822** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d$ = **21840** 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; $M_{sls.x.max}$ = **65.3** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z)$ = **208.7** N/mm^2

Load duration factor; k_t = **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **90** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y$ = **135000** mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2

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

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **6.395**

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); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff}$ = **406** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$ $0.6 \times \sigma_s / E_s)$ = **0.242** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear (Section 6.2)

Design shear force; $abs(V_{Ed.x.min})$ = **169.2** kN

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.745**

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.442** N/mm^2

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

$V_{Rd.c}$ = **238.7** kN

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.y.max}$ = **77.9** kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **352** mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck})$ = **0.008**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **334** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **44** mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z)$ = **536** mm^2

Tension reinforcement provided; 12 No.12 dia.bars bottom (225 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **1357** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d$ = **1325** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d$ = **35200** 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; $M_{sls.y.max}$ = **39.4** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z)$ = **86.8** N/mm^2

Load duration factor; k_t = **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **119** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x$ = **296667** mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff}$ = **0.005**

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **6.395**

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); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff}$ = **589** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = **0.146** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in flexure (Section 6.1)

Design bending moment; $abs(M_{Ed.y.min})$ = **1.9** kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.top} - \phi_{y.top} / 2$ = **340** mm

$K = abs(M_{Ed.y.min}) / (L_x \times d^2 \times f_{ck})$ = **0.000**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **323** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **43** mm

Area of tension reinforcement required; $A_{sy.top.req} = abs(M_{Ed.y.min}) / (f_{yd} \times z)$ = **14** mm^2

Tension reinforcement provided; 10 No.20 dia.bars top (275 c/c)

Area of tension reinforcement provided; $A_{sy.top.prov}$ = **3142** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d$ = **1280** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d$ = **34000** 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; $abs(M_{sls.y.min})$ = **1.9** kNm

Tensile stress in reinforcement; $\sigma_s = abs(M_{sls.y.min}) / (A_{sy.top.prov} \times z)$ = **1.9** N/mm^2

Load duration factor; k_t = **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **119** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x$ = **297917** mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2

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

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **6.395**

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); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.top}) + k_1 \times k_2 \times k_4 \times \phi_{y.top} / \rho_{p.eff}$ = **492** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = **0.003** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear (Section 6.2)

Design shear force; $abs(V_{Ed.y.min})$ = **11.9** kN

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

$k = \min(1 + \sqrt{(200 mm / d)}, 2)$ = **1.767**

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

$v_{min} = 0.035 N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.450** N/mm^2

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

$V_{Rd.c}$ = **382.7** kN

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250 N/mm^2]$ = **0.528**

Average depth to reinforcement; d = **358** mm

Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd}$ = **5.280** N/mm^2

$k = \min(1 + \sqrt{(200 mm / d)}, 2)$ = **1.747**

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

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

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

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{\underline{0.443}}\ N/mm^2$

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) = \mathbf{\underline{0.443}}\ N/mm^2$

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = \mathbf{\underline{1200}}$ mm

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

Maximum punching shear force; $V_{Ed.max} = \mathbf{\underline{500.7}}$ kN

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

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = \mathbf{\underline{1.748}}\ N/mm^2$

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

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

Punching shear perimeter; $u_2 = \mathbf{\underline{3446}}$ mm

Area within punching shear perimeter; $A_2 = \mathbf{\underline{2.367}}\ m^2$

Design punching shear force; $V_{Ed.2} = \mathbf{\underline{152.3}}$ kN

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

Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = \mathbf{\underline{0.185}}\ N/mm^2$

PASS - Design punching shear resistance exceeds design punching shear stress