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Project Title: 26 and 27 Kings Mews	Project No.: 99737
Stage: STAGE 3	
Issue Date/Revision: 03.05.2024_P1	Prepared By: WW

Introduction

The purpose of this letter is to set out the required groundwater mitigation measures required during the excavation of the basement for 26 Kings Mews.

Groundwater Conditions

During the excavation of a single borehole undertaken during Ground Engineering's site investigation water was struck at 4.00m bgl which then rose to 3.60m bgl within fifteen minutes.

Using a 7m deep standpipe the water level recorded after a 3-week period was 3.04m below ground level. This is 810mm above the final excavation level of the new basement, taking into consideration the thickness of the basement slab and blinding layer. Therefore, de-watering methods will need to be utilised during the construction of the proposed basement.

Construction of Underpins/Retaining Walls

To construct the front and rear underpins, temporary supports will be installed to the faces of the trenches. At the bottom of the trench a sump pump is to be installed to reduce the ground water level around the excavation (see figure 1). The pumped water will discharge into an observable v-notch tank which can then feed into a bowser or discharged at a low rate to the foul drainage system (control testing for contaminants and physical measurement of flow may prove necessary by local authority).

- To prevent boiling at the bottom of the trench the sheeting will be installed a minimum of 300mm below the foundation level of the excavation and an open pump surrounded by gravel will be used as the sump.
- To prevent the removal of fines from the soil, surround the suction inlet with a protected grade filter and reduce the flow rate through the soil, this is achieved by using an open pipe surrounded by gravel.
- The pumped water will be monitored taking water samples and checking proportion of fines being removed. If excessive fines are being continuously withdrawn or signs of trench instability sump pumping will be stopped.

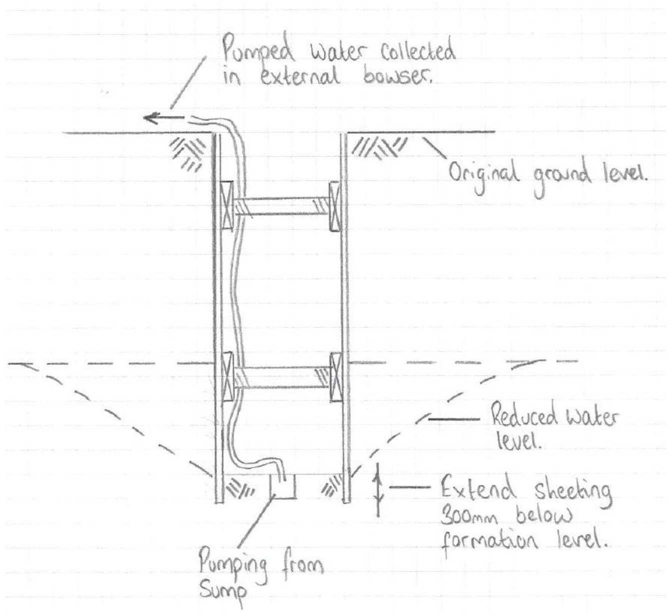

Figure 1 – Trench Sump Pump Section

Construction of Basement Slab

Once the retaining walls are cast and cured, the remainder of the basement excavation will take place. To control ground water levels during this stage a sump pump will again be used. The sump will be located in the corner of the excavation and drains leading to the sump will have a sufficient fall to prevent silting up and be cleaned out regularly.

A cage will be installed at the base of the sump and fill between the cage and the shaft with graded filter material.

- To prevent the removal of fines from the soil, surround the suction inlet with a protected grade filter.
- To prevent ground loss around the sumps install cage at base of sump and fill cage with well graded filter material, along with appropriately designed sump casing perforations.
- The pumped water will be monitored taking water samples and checking proportion of fines being removed if excessive fines are being continuously withdrawn.

Note: This report will need to be provided as part of the Tender document pack to the required contractors.

Prepared by:	**Reviewed by:**	**Approved by:**
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Sheet No. 1	Rev. P1
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Date	
Calcs by WW	Checked by AO

Calculation sheet

Project 26/27 Kings Mews

Front Retaining Wall Design

Site Investigation Data

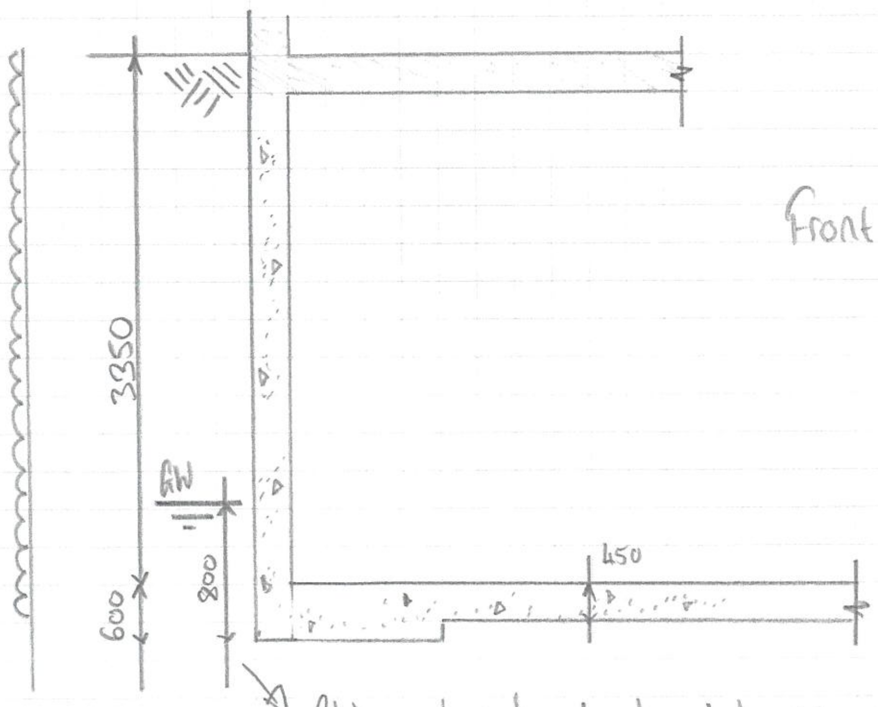
- Allowable soil bearing = $150\ kN/m^2$
- Soil Condition: Made ground: $18\ kN/m^3$, $c' = 0$, $\phi' = 28$, 0-3m

  Lynch hill gravel: $21\ kN/m^3$, $c' = 0$, $\phi' = 36$, 3-6m

  London Clay: $20\ kN/m^3$, $c' = 0{\cdot}2$, $\phi' = 22$, 6m+

→ Data from Ground Engineering SI

→ Retaining wall designed and analysed on tekla tedds, Calcs on following pages.



Front Retaining Wall Diagram.

GW analysed at 1m bgl as conservative calc.

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Calculation sheet	
Project	26/27 Kings Mews

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Loading on Retaining Wall

Surcharge Load

- Vehicular road access provides surcharge to front retaining wall.

→ UDL = 20 Kd $K_d = \dfrac{1 - \sin(28)}{1 + \sin(28)} = 0.36$

UDL = 20 × 0.36 = 7.2 kN/m^2

Vertical load from Structure

- From tekla model analysis: 450 kN (3 column point loads on wall).
- Length = 6.5m → Load spread = $\dfrac{450}{6.5m}$ = 69 kN/m

G_K = 39.8 kN/m Q_K = 10.6 kN/m

→ Bearing capacity analysis carried out on tekla designer.

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Sheet No. 3	Rev. P1

Calculation sheet	
Project 26/27 Kings Mews	

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Calcs by WhJ	Checked by AG

Reinforcement Cover Requirements

- Following Eurocode 1992-1-1 :

$C_{nom} = C_{min} + \Delta C_{dev} = \underline{35mm}$

$C_{dev} = 10mm$

$C_{min} = \max(16;\ 25+0+0+0;\ 10) = 25mm$

→ Exposure class = XC2 → Structural class = S4

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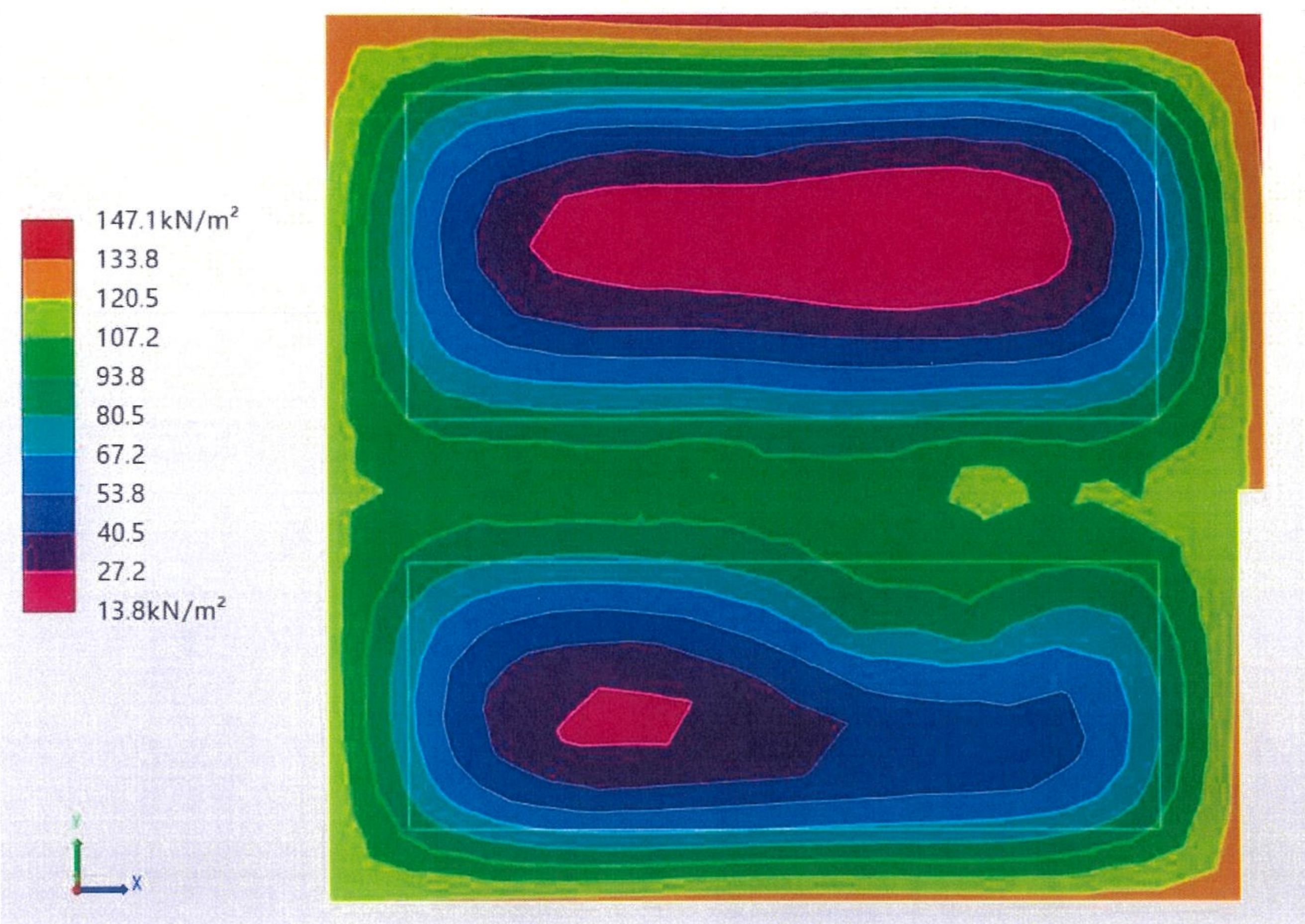
Calculation sheet			
Project	26/27 Kings Mews		
Project No.	99737		
Date			
Calcs by	WW	Checked by	AO



Bearing Pressure Diagram

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.21

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3550** mm
Prop height	h_{prop} = **3550** mm
Stem thickness	t_{stem} = **250** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1450** mm
Base thickness	t_{base} = **600** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3550** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2550** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **14** deg

Base soil properties

Soil type	Medium dense fine or silty sand
Soil density	γ_b = **21** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **36** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **18** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **20** deg
Presumed bearing capacity	$P_{bearing}$ = **300** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **7.2** kN/m^2
Vertical line load at 1575 mm	P_{G1} = **39.8** kN/m
	P_{Q1} = **10.6** kN/m

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem}$ = **1700** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover}$ = **2550** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water}$ = **1000** mm
Length of surcharge load	$l_{sur} = l_{heel}$ = **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **1700** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **4150** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **2075** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **0.888** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1575** mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{1.02}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{850}$ mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.326}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{8.022}$

Bearing pressure check → Refer to bearing pressure diagram from tSD (page 4).

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{22.2}$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{25.5}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{50.4}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{98.1}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{9.5}$ kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{12.9}$ kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{48.7}$ kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{20.8}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-28.8}$ kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{63}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{34.9}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{21.7}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-19.6}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{79.4}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-13.5}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-51.1}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-38.2}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{13.5}$ kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = \mathbf{17.5}$ kN/m

Propping force to base $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{45.4}$ kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{72.7}$ kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{879}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{29}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{1700}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{51.8}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{63.6}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{4.714}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.21

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

Concrete strength class	C35/45
Characteristic compressive cylinder strength	f_{ck} = **35** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **45** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}$ / 1 N/mm$^2)^{2/3}$ = **3.2** N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm}$ / 10 N/mm$^2)^{0.3}$ = **34077** N/mm^2
Partial factor for concrete - Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** 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

Cover to reinforcement

Front face of stem	c_{sf} = **35** mm
Rear face of stem	c_{sr} = **35** mm
Top face of base	c_{bt} = **35** mm
Bottom face of base	c_{bb} = **35** mm

Check stem design at base of stem

Depth of section	h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **131.8** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **202** mm
	$K = M / (d^2 \times f_{ck})$ = **0.092**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**
	K' > K - No compression reinforcement is required
Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **184** mm

Depth of neutral axis	$x = 2.5 \times (d - z) = \mathbf{45}$ mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) = \mathbf{1644}$ mm²/m
Tension reinforcement provided	25 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{2454}$ mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{338}$ mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h = \mathbf{10000}$ mm²/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.67}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = \mathbf{0.006}$
Required tension reinforcement ratio	$\rho = A_{sr.req} / d = \mathbf{0.008}$
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$
Structural system factor - Table 7.4N	$K_b = \mathbf{1}$
Reinforcement factor - exp.7.17	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.493}$
Limiting span to depth ratio - exp.7.16.b	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / (\rho - \rho') + \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \sqrt{(\rho' / \rho_0)} / 12], 40 \times K_b) = \mathbf{26.1}$
Actual span to depth ratio	$h_{prop} / d = \mathbf{17.5}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment	$M_{sls} = \mathbf{88.4}$ kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{195.3}$ N/mm²
Load duration	Long term
Load duration factor	$k_t = \mathbf{0.4}$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{68320}$ mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.036}$
Modular ratio	$\alpha_e = E_s / E_{cm} = \mathbf{5.869}$
Bond property coefficient	$k_1 = \mathbf{0.8}$
Strain distribution coefficient	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{237}$ mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.18}$ mm
	$w_k / w_{max} = \mathbf{0.601}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{97.9}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.994}$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.012}$
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.583}$ N/mm²

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

$V_{Rd.c}$ = **169** kN/m

$V / V_{Rd.c}$ = **0.579**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section — h = **250** mm

Rectangular section in shear - Section 6.2

Design shear force — V = **7.9** kN/m

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

$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.994**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.005**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.583** N/mm²

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

$V_{Rd.c}$ = **125.5** kN/m

$V / V_{Rd.c}$ = **0.063**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **614** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

Transverse reinforcement provided — 12 dia.bars @ 150 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **754** mm²/m

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

Check base design at toe

Depth of section — h = **600** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **148.3** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **557** mm

$K = M / (d^2 \times f_{ck})$ = **0.014**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **529** mm

Depth of neutral axis — $x = 2.5 \times (d – z)$ = **70** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z)$ = **645** mm²/m

Tension reinforcement provided — 16 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1005** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **930** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h$ = **24000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.925**

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

Library item: Rectangular single output

Crack control - Section 7.3

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

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **99.9** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{187.8}$ N/mm^2

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{107500}$ mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.009}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

Bond property coefficient — $k_1 = \mathbf{0.8}$

Strain distribution coefficient — $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{410}$ mm

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

$w_k = \mathbf{0.231}$ mm

$w_k / w_{max} = \mathbf{0.77}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{104.6}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.599}$

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.002}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.419}$ N/mm^2

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

$V_{Rd.c} = \mathbf{233.2}$ kN/m

$V / V_{Rd.c} = \mathbf{0.449}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2) — $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{201}$ mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) — $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided — 12 dia.bars @ 150 c/c

Area of transverse reinforcement provided — $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{754}$ mm^2/m

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

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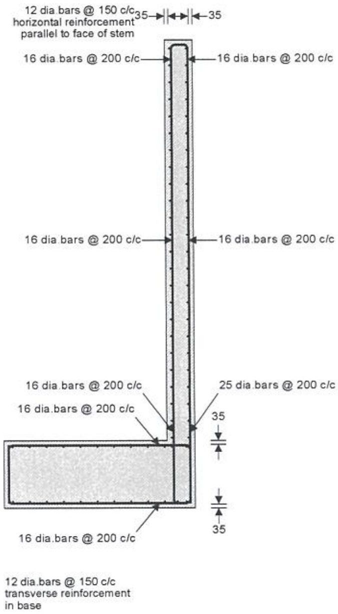
	Calcs by: WW	Calcs date: 03/05/2024	Checked by: AO	Checked date: 02/05/2024	Approved by: AO	Approved date: 02/05/2024



Reinforcement details