

APPENDIX B

STRUCTURAL CALCULATIONS

WE ARE SYMMETRYS

STRUCTURAL CALCULATION PACKAGE

139 CRANLEY GARDENS
LONDON N10 3AH

REF: 21066 -SYM-XX-XX-SC-S-0001 REV C





Revision History

Revision	Description	Date	By	Checked
A	First issue	21.02.22	SB	DS
B	Second Issue	02.05.23	SB	CA
C	Third Issue	07.08.23	SB	CA



1.0 Introduction

1.1 Symmetrys were instructed by Crawford Partnership Architects to undertake the structural design in support of the planning application for a proposed basement extension at 139 Cranley Gardens, London N10 3AH, in the London Borough of Haringey.

1.2 The structural design works shall consist of the following:

- Design of the substructure of a 1-storey basement (excluding piling, which is to be designed by a specialist designer);

2.0 Design Codes

2.1 The following design codes were used to carry out the design:

- BS EN 1990 – Basis of Structural design
- BS EN 1991 – Actions on Structure.
- BS EN 1992 – Design of Concrete Structures;
- BS EN 1993 – Design of steel structures;
- BS EN 1997 – Geotechnical design.

3.0 Ground conditions

3.1 A Phase 2 site investigation has been carried out by “Site Analytical Services Ltd”, confirming Made Ground to a depth of 0.85m overlaying London Clay.

4.0 Substructure design

4.1 An allowable net bearing pressure of 250kN/m² has been considered for design purposes (accordingly to allowable net bearing pressure value provided in the “Site Analytical Services Ltd” Phase 2 report for a foundation formed 3.0m below ground level). At-rest pressure has been considered. Surcharge loadings from 137 Cranley Gardens have been accounted for. The groundwater level has been considered 1 meter below ground level for design purposes.

4.2 The following design parameters have been considered for the purpose of the substructure element design.

Stratum	Depth to top	Bulk Density (Mg/m ³) (γ)	Effective Angle of Internal Friction (Φ)
London Clay Formation	0.60	2.00	23

4.3 The uplift force resulting from buoyancy and hydrostatic pressure is dealt with by the self-weight of the basement concrete box, heave protection and tension piles. The tension piles design has been included although it must be considered indicative only and to be confirmed by a specialist designer.

5.0 Superstructure design

5.1 The design of the superstructure element is not included in this report.

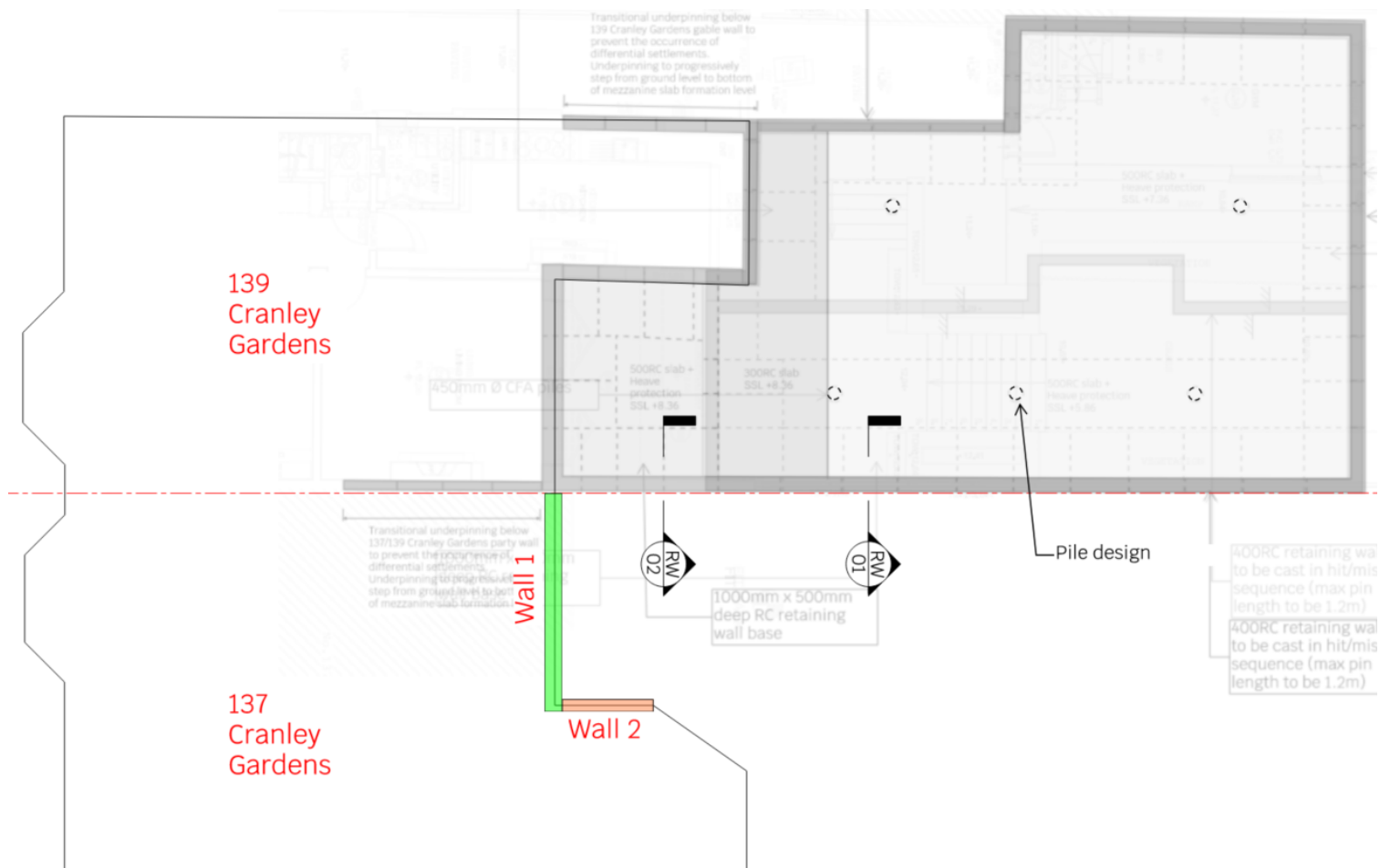


6.0 Loadings

Item	DL (kPa)	LL (kPa)
Timber roof		
Ceiling and services	0.2	
Joists	0.2	
Plyboard and finishes	0.3	
Insulation	0.2	
Imposed load (snow)		0.6
Total dead load (flat roof)	0.9	
Solid Masonry		
215mm thick	4.1	
15mm plaster	0.3	
Total	4.4	
Ground floor level (area above basement)		
Finishes	1.0	
400mm RC slab	10	
Imposed load		2.5
Total	11	2.5
Basement floor level (mezzanine, swimming pool side)		
Finishes	1.0	
500mm RC slab	12.5	
Imposed load		2.5
Total	13.5	2.5
Basement floor level (swimming pool)		
Finishes	1.0	
500mm RC slab	12.5	
Imposed load		2.5
Water (9.81kN/m ³ x 1.5m – in full capacity)		14.7
Total	13.5	17.2
Existing masonry	19*	
Reinforced concrete	25*	
Note: * = kN/m³		



7.0 Structural elements reference plan and calculations





		Job No.	Sheet No.	Revision
		21066	1 of 5	P1
Job Title	139 Cranley Gardens	Date	Made by	Checked by
Section	Uplift check (excavation level +5.86) - Swimming Pool	24/04/2023	SB	CA

Data (geometry and soil condition)

Excavation depth = 6.2 m
Water level (BGL) = 1.0 m (assumed)
Water density = 9.81 KN/m³
Soil density = 20.0 KN/m³
Overburden γ_G = 1.00 (SLS)
Water pressure γ_G = 1.00 (SLS)

Uplift check

Hydrostatic pressure = $9.81 \times (6.2 - 1.0) = 51.0 \text{ KN/m}^2$

Overburden pressure = $20.00 \times 6.2 = 124.0 \text{ KN/m}^2$

Assuming half soil heave occurs during construction:

Uplift = Hydrostatic pressure + $1/2 \times (\text{overburden pressure} - \text{hydrostatic pressure}) = 88 \text{ KN/m}^2$

Uplift SLS = $1.00 \times 87.5 = 87.5 \text{ KN/m}^2$

Gross area = 57m²

Net area (reduced of area above heave protection) = 38m²

Uplift force = 3325KN



		Job No.	Sheet No.	Revision
		21066	2 of 5	P1
Job Title	139 Cranley Gardens	Date	Made by	Checked by
Section	Uplift check (excavation level +7.36) - Pool side	24/04/2023	SB	CA

Data (geometry and soil condition)

Excavation depth = 4.7 m
Water level (BGL) = 1.0 m (assumed)
Water density = 9.81 KN/m³
Soil density = 20.0 KN/m³
Overburden γ_G = 1.00
Water pressure γ_G = 1.00

Uplift check

Hydrostatic pressure = $9.81 \times (4.7 - 1.0) = 36.3$ KN/m²

Overburden pressure = $20.00 \times 4.7 = 94.0$ KN/m²

Assuming half soil heave occurs during construction:

Uplift = Hydrostatic pressure + $1/2 \times (\text{overburden pressure} - \text{hydrostatic pressure}) = 65$ KN/m²

Uplift ULS = $1.00 \times 65.1 = 65.1$ KN/m²

Gross area = 53m²

Net area (reduced of area above heave protection) = 35m²

Uplift force = 2278KN



		Job No.	Sheet No.	Revision
		21066	3 of 5	P1
Job Title	139 Cranley Gardens	Date	Made by	Checked by
Section	Uplift check (excavation level +8.36) - Mezzanine	24/04/2023	SB	CA

Data (geometry and soil condition)

Excavation depth = 3.8 m
Water level (BGL) = 1.0 m (assumed)
Water density = 9.81 KN/m³
Soil density = 20.0 KN/m³
Overburden γ_G = 1.00
Water pressure γ_G = 1.00

Uplift check

Hydrostatic pressure = $9.81 \times (3.8 - 1.0) = 27.0$ KN/m²

Overburden pressure = $20.00 \times 3.8 = 75.0$ KN/m²

Assuming half soil heave occurs during construction:

Uplift = Hydrostatic pressure + $1/2 \times (\text{overburden pressure} - \text{hydrostatic pressure}) = 51$ KN/m²

Uplift ULS = $1.00 \times 51 = 51$ KN/m²

Gross area = 11m²

Net area (reduced of area above heave protection) = 4m²

Uplift force = 204KN



		Job No.	Sheet No.	Revision
		21066	4 of 5	P1
Job Title	139 Cranley Gardens, London N10	Date	Made by	Checked by
Section	Uplift check - Analysis results	24.04.23	SB	CA

Loadings

Uplift pressure (SLS) = 5807KN

Derivation of loadings (characteristic) for uplift check

Floor	Loadings per sq m		Area (sq m)	Total point load per floor	
	DL	LL		DL	LL
Ground floor	11.0	2.5	121	1331.0	302.5
BL - Mezzanine	13.5	2.5	11	148.5	27.5
BL - Pool Side	13.5	2.5	53	715.5	132.5
BL - Pool	13.5	17.2	57	769.5	980.4
Roof (outbuilding)	0.9	0.6	24	21.6	14.4

Wall	Loadings per sq m		Height (m)	Length (m)	Total udl	
	DL	LL			DL	LL
BL - 400RC wall	10.0	0.0	4	52.0	2080.0	0.0
BL - 400RC wall (pool)	10.0	0.0	1.5	18.4	276.0	0.0

Extension (garden house)	Loadings per sq m		Height (m)	Length (m)	Total udl	
	DL	LL			DL	LL
215mm Masonry	4.1	0.0	3	20.0	246.0	0.0

	DL	LL
Total udl	5588.1	1457.3

Design for uplift at ultimate limit stage (EQU)

Actions:

Permanent (favourable) => $\gamma_G = 0.9$
 Permanent (unfavourable) => $\gamma_G = 1.1$
 Variable (favourable) => $\gamma_Q = 0.0$
 Variable (unfavourable) => $\gamma_Q = 1.5$

Assuming the uplift pressure as permanent unfavourable, the building permanent loads as favourable, and the building variable load as favourable:

Stabilizing action $0.9 * 5588.1 = 5029 \text{ KN}$

Destabilising action $= 1.1 * 5807 = 6387 \text{ KN}$

Stabilising action < destabilising action => EQU check is not satisfied. Tension piles required in permanent and temporary condition



Job Title	139 Cranley Gardens, London N10	Job No.	Sheet No.	Revision
		21066	5 of 5	P1
Section	Piles design	Date	Made by	Checked by
		24.04.23	SB	CA

CODE: BS EN 1997-1 : 2004 + A1 : 2013

APPROACH : M1, A1

q = ALLOWABLE BEARING PRESSURE = 250 kN/m^2 (*)

F.O.S. = 2,5 (*)

(*) = ACCORDING TO S.A.S. PHASE 2 REPORT, REF: 21/33613, MAY 2021

$$C_u = \frac{q \times \text{F.O.S.}}{6} = \frac{250 \cdot 2,5}{6} = 104 \text{ kN/m}^2$$

$$C_{u,k} = \gamma_{cu} \cdot C_u = 1,0 \cdot 104 = 104 \text{ kN/m}^2$$

ASSUMING $\phi 450\text{mm}$ PILES, 15m DEEP IN CLAY, TENSION RESISTANCE IS:

$$R_{t,k} = \frac{R_{s,cd1}}{\gamma_{R,d}} = \frac{\pi D L \alpha C_{u,k}}{\gamma_{R,d}} = \frac{\pi \cdot 0,45 \cdot 15 \cdot 0,5 \cdot 104}{1,4} = 787,6 \text{ kN}$$

$$R_{t,d} = \frac{R_{t,k}}{\gamma_{s,t}} = \frac{787,6}{2} = 393,8 \text{ kN}$$

TOTAL STABILIZING ACTION (GRAVITY + 5 NO TENSION PILES) = $5029 + 5 \cdot 393,8 = 6998 \text{ kN}$

DESTABILIZING ACTION (UPLIFT) = 6387 kN

$\Rightarrow 6998 \text{ kN} > 6387 \text{ kN} \therefore \text{OK}$

ADOPT 5 NO 450mm ϕ PILES, 15m DEEP IN CLAY

NOTE: DESIGN INDICATIVE FOR PLANNING ONLY. PILING DESIGN TO BE CONFIRMED BY SPECIALIST.



Job No.	Sheet No.	Revision
21066	1 of 2	P1
Date	Made by	Checked by
28.07.23	SB	CA

Job Title	139 Cranley Gardens, London N10
Section	Surcharge loadings from 137 Cranley Gardens

Derivation of loadings (characteristic) for Wall 1

Floor	Loadings per sq m		Span (m)	Total uld	
	DL	LL		DL	LL
Roof (35 deg)	1.2		2.3	2.8	0.0
Roof		0.6	2.3	0.0	1.4
2nd floor	0.6		2.3	1.4	0.0
2nd floor		2.5	2.3	0.0	5.8
1st floor	0.6		2.3	1.4	0.0
1st floor		2.5	2.3	0.0	5.8
215mm masonry wall	4.5		5.3	23.9	0.0
TOTAL UDL (KN/m)				29.4	12.9
TOTAL LOAD (KN)				117.5	51.5

Considering the total load applied at the centre of Wall 1 (i.e. 2m from proposed basement wall)
the resulting lateral pressure on the basement retaining wall can be calculated using Boussinesq equations

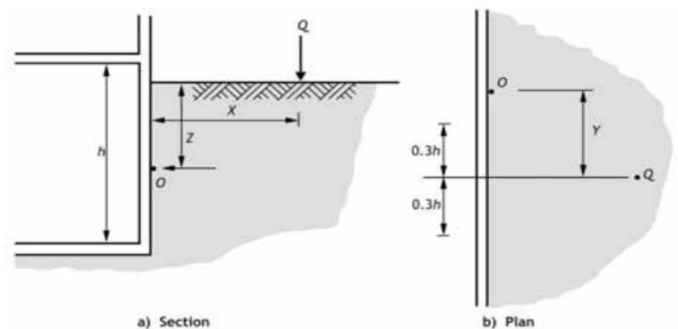
$$\sigma = 3QZ^3 / (2\pi R^5)$$

Where:

Q = load, KN

Z = depth, m

$$R = (X^2 + Y^2 + Z^2)^{0.5}$$



Loadings:

Gk= 117.5 KN

Qk= 51.5 KN

X= 2 m

Y assumed 0

Z (m)	R	σ (DL)	σ (LL)
0.0	2.0	0.0	0.0
0.5	2.1	0.2	0.1
1.0	2.2	1.0	0.4
1.5	2.5	1.9	0.9
2.0	2.8	2.5	1.1
2.5	3.2	2.6	1.1
3.0	3.6	2.5	1.1
3.5	4.0	2.3	1.0
4.0	4.5	2.0	0.9

Average values of permanent and variable pressure due to surcharge from Wall 2 are:

$$\sigma \text{ (DL)} = 2.0$$

$$\sigma \text{ (LL)} = 0.9$$



Job Title	139 Cranley Gardens, London N10	Job No.	Sheet No.	Revision
		21066	2 of 2	P1
Section	Surcharge loadings from 137 Cranley Gardens	Date	Made by	Checked by
		28.07.23	SB	CA

Derivation of loadings (characteristic) for Wall 2

Floor	Loadings per sq m		Span (m)	Total uld	
	DL	LL		DL	LL
Roof (35 deg)	1.2		1.6	1.9	0.0
Roof		0.6	1.6	0.0	1.0
2nd floor	0.6		1.6	1.0	0.0
2nd flor		2.5	1.6	0.0	4.0
1st floor	0.6		1.6	1.0	0.0
1st floor		2.5	1.6	0.0	4.0
215mm masonry wall	4.5		5.3	23.9	0.0
TOTAL UDL (KN/m)				27.7	9.0
TOTAL LOAD (KN)				110.8	35.8

Considering the total load applied at the centre of Wall 2 (i.e. 4m from proposed basement wall)
the resulting lateral pressure on the basement retaining wall can be calculated using Boussinesq equations

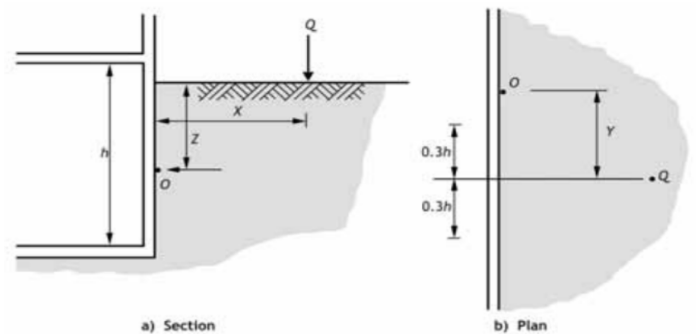
$$\sigma = 3QZ^3 / (2\pi R^5)$$

Where:

Q = load, KN

Z = depth, m

$$R = (X^2 + Y^2 + Z^2)^{0.5}$$



Loadings:

Gk= 110.8 KN

Qk= 35.8 KN

X= 4 m

Y assumed 0

Z (m)	R	σ (DL)	σ (LL)
0.0	4.0	0.00	0.00
0.5	4.0	0.01	0.00
1.0	4.1	0.04	0.01
1.5	4.3	0.13	0.04
2.0	4.5	0.24	0.08
2.5	4.7	0.35	0.11
3.0	5.0	0.46	0.15
3.5	5.3	0.54	0.17
4.0	5.7	0.58	0.19

Average values of permanent and variable pressure due to surcharge from Wall 2 are:

$$\sigma \text{ (DL)} = 0.2$$

$$\sigma \text{ (LL)} = 0.1$$



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Unit 6, The Courtyard, Lynton Road
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Project 139 CRANLEY GARDENS, LONDON N10 3AH				Job no. 21066	
Calcs for RETAINING WALL RW1				Start page no./Revision 1	
Calcs by SB	Calcs date 28/07/2023	Checked by CA	Checked date	Approved by CA	Approved date

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.17

Retaining wall details

Stem type	Propped cantilever
Stem height	$h_{\text{stem}} = 5400 \text{ mm}$
Prop height	$h_{\text{prop}} = 5400 \text{ mm}$
Stem thickness	$t_{\text{stem}} = 400 \text{ mm}$
Angle to rear face of stem	$\alpha = 90 \text{ deg}$
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	$l_{\text{toe}} = 1000 \text{ mm}$
Base thickness	$t_{\text{base}} = 500 \text{ mm}$
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	$h_{\text{ret}} = 5400 \text{ mm}$
Angle of soil surface	$\beta = 0 \text{ deg}$
Depth of cover	$d_{\text{cover}} = 0 \text{ mm}$
Height of water	$h_{\text{water}} = 4400 \text{ mm}$
Water density	$\gamma_w = 9.8 \text{ kN/m}^3$

Retained soil properties

Soil type	Firm clay
Moist density	$\gamma_{\text{mr}} = 20 \text{ kN/m}^3$
Saturated density	$\gamma_{\text{sr}} = 20 \text{ kN/m}^3$
Characteristic effective shear resistance angle	$\phi'_{r,k} = 23 \text{ deg}$
Characteristic wall friction angle	$\delta_{r,k} = 11.5 \text{ deg}$

Base soil properties

Soil type	Firm clay
Soil density	$\gamma_b = 20 \text{ kN/m}^3$
Characteristic effective shear resistance angle	$\phi'_{b,k} = 23 \text{ deg}$
Characteristic wall friction angle	$\delta_{b,k} = 11.5 \text{ deg}$
Characteristic base friction angle	$\delta_{bb,k} = 13.5 \text{ deg}$
Presumed bearing capacity	$P_{\text{bearing}} = 250 \text{ kN/m}^2$

Loading details

Permanent surcharge load	Surcharge _G = 2 kN/m ²	Surcharge from 137 Cranley Gardens, Wall 1 + Wall 2
Variable surcharge load	Surcharge _Q = 3.4 kN/m ²	Surcharge from 137 Cranley Gardens, Wall 1 + Wall 2 + 2.5 kN/m ²
Vertical line load at 1200 mm	$P_{G1} = 35 \text{ kN/m}$ $P_{Q1} = 8.8 \text{ kN/m}$	Ground floor slab. Span 3.2m

$$F_{P_v} = P_{G1} + P_{Q1} = \mathbf{43.8 \text{ kN/m}}$$



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Calcs for RETAINING WALL RW1				Start page no./Revision 3	
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Total

$$F_{\text{total}_v} = F_{\text{stem}} + F_{\text{base}} + F_{P_v} + F_{\text{water}_v} = \mathbf{115.3 \text{ kN/m}}$$

Horizontal forces on wall

Surcharge load

$$F_{\text{sur}_h} = K_0 \times \cos(\delta_{r,k}) \times (\text{Surcharge}_G + \text{Surcharge}_Q) \times h_{\text{eff}} = \mathbf{19 \text{ kN/m}}$$

Saturated retained soil

$$F_{\text{sat}_h} = K_0 \times \cos(\delta_{r,k}) \times (\gamma_{\text{sr}} - \gamma_w) \times (h_{\text{sat}} + h_{\text{base}})^2 / 2 = \mathbf{73 \text{ kN/m}}$$

Water

$$F_{\text{water}_h} = \gamma_w \times (h_{\text{water}} + d_{\text{cover}} + h_{\text{base}})^2 / 2 = \mathbf{117.8 \text{ kN/m}}$$

Moist retained soil

$$F_{\text{moist}_h} = K_0 \times \cos(\delta_{r,k}) \times \gamma_{\text{mr}} \times ((h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}})^2 / 2 + (h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}}) \times (h_{\text{sat}} + h_{\text{base}})) = \mathbf{64.5 \text{ kN/m}}$$

Base soil

$$F_{\text{pass}_h} = -K_P \times \cos(\delta_{b,k}) \times \gamma_b \times (d_{\text{cover}} + h_{\text{base}})^2 / 2 = \mathbf{-7.7 \text{ kN/m}}$$

Total

$$F_{\text{total}_h} = F_{\text{sur}_h} + F_{\text{sat}_h} + F_{\text{water}_h} + F_{\text{moist}_h} + F_{\text{pass}_h} = \mathbf{266.6 \text{ kN/m}}$$

Moments on wall

Wall stem

$$M_{\text{stem}} = F_{\text{stem}} \times X_{\text{stem}} = \mathbf{64.8 \text{ kNm/m}}$$

Wall base

$$M_{\text{base}} = F_{\text{base}} \times X_{\text{base}} = \mathbf{12.3 \text{ kNm/m}}$$

Surcharge load

$$M_{\text{sur}} = -F_{\text{sur}_h} \times X_{\text{sur}_h} = \mathbf{-56.1 \text{ kNm/m}}$$

Line loads

$$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{52.5 \text{ kNm/m}}$$

Saturated retained soil

$$M_{\text{sat}} = -F_{\text{sat}_h} \times X_{\text{sat}_h} = \mathbf{-119.3 \text{ kNm/m}}$$

Water

$$M_{\text{water}} = -F_{\text{water}_h} \times X_{\text{water}_h} = \mathbf{-192.4 \text{ kNm/m}}$$

Moist retained soil

$$M_{\text{moist}} = -F_{\text{moist}_h} \times X_{\text{moist}_h} = \mathbf{-174.6 \text{ kNm/m}}$$

Total

$$M_{\text{total}} = M_{\text{stem}} + M_{\text{base}} + M_{\text{sur}} + M_P + M_{\text{sat}} + M_{\text{water}} + M_{\text{moist}} = \mathbf{-412.8 \text{ kNm/m}}$$

Check bearing pressure

Propping force to stem

$$F_{\text{prop}_\text{stem}} = (F_{\text{total}_v} \times l_{\text{base}} / 2 - M_{\text{total}}) / (h_{\text{prop}} + t_{\text{base}}) = \mathbf{83.6 \text{ kN/m}}$$

Propping force to base

$$F_{\text{prop}_\text{base}} = F_{\text{total}_h} - F_{\text{prop}_\text{stem}} = \mathbf{183 \text{ kN/m}}$$

Moment from propping force

$$M_{\text{prop}} = F_{\text{prop}_\text{stem}} \times (h_{\text{prop}} + t_{\text{base}}) = \mathbf{493.5 \text{ kNm/m}}$$

Distance to reaction

$$\bar{x} = (M_{\text{total}} + M_{\text{prop}}) / F_{\text{total}_v} = \mathbf{700 \text{ mm}}$$

Eccentricity of reaction

$$e = \bar{x} - l_{\text{base}} / 2 = \mathbf{0 \text{ mm}}$$

Loaded length of base

$$l_{\text{load}} = l_{\text{base}} = \mathbf{1400 \text{ mm}}$$

Bearing pressure at toe

$$q_{\text{toe}} = F_{\text{total}_v} / l_{\text{base}} \times (1 - 6 \times e / l_{\text{base}}) = \mathbf{82.3 \text{ kN/m}^2}$$

Bearing pressure at heel

$$q_{\text{heel}} = F_{\text{total}_v} / l_{\text{base}} \times (1 + 6 \times e / l_{\text{base}}) = \mathbf{82.3 \text{ kN/m}^2}$$

Factor of safety

$$FoS_{bp} = P_{\text{bearing}} / \max(q_{\text{toe}}, q_{\text{heel}}) = \mathbf{3.037}$$

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 using user defined factors

Tedds calculation version 2.9.17

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

Concrete strength class

$$\mathbf{C32/40}$$

Characteristic compressive cylinder strength

$$f_{ck} = \mathbf{32 \text{ N/mm}^2}$$

Characteristic compressive cube strength

$$f_{ck,\text{cube}} = \mathbf{40 \text{ N/mm}^2}$$

Mean value of compressive cylinder strength

$$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = \mathbf{40 \text{ N/mm}^2}$$

Mean value of axial tensile strength

$$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.0 \text{ N/mm}^2}$$

5% fractile of axial tensile strength

$$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1 \text{ N/mm}^2}$$

Secant modulus of elasticity of concrete

$$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = \mathbf{33346 \text{ N/mm}^2}$$

Partial factor for concrete - Table 2.1N

$$\gamma_C = \mathbf{1.50}$$

Compressive strength coefficient - cl.3.1.6(1)

$$\alpha_{cc} = \mathbf{0.85}$$

Design compressive concrete strength - exp.3.15

$$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1 \text{ N/mm}^2}$$



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Project

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Job no.

21066

Calcs for

RETAINING WALL RW1

Start page no./Revision

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CA

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Maximum aggregate size

$h_{agg} = 20 \text{ mm}$

Ultimate strain - Table 3.1

$\epsilon_{cu2} = 0.0035$

Shortening strain - Table 3.1

$\epsilon_{cu3} = 0.0035$

Effective compression zone height factor

$\lambda = 0.80$

Effective strength factor

$\eta = 1.00$

Bending coefficient k_1

$K_1 = 0.40$

Bending coefficient k_2

$K_2 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$

Bending coefficient k_3

$K_3 = 0.40$

Bending coefficient k_4

$K_4 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$

Reinforcement details

Characteristic yield strength of reinforcement

$f_{yk} = 500 \text{ N/mm}^2$

Modulus of elasticity of reinforcement

$E_s = 200000 \text{ N/mm}^2$

Partial factor for reinforcing steel - Table 2.1N

$\gamma_s = 1.15$

Design yield strength of reinforcement

$f_{yd} = f_{yk} / \gamma_s = 435 \text{ N/mm}^2$

Cover to reinforcement

Front face of stem

$c_{sf} = 40 \text{ mm}$

Rear face of stem

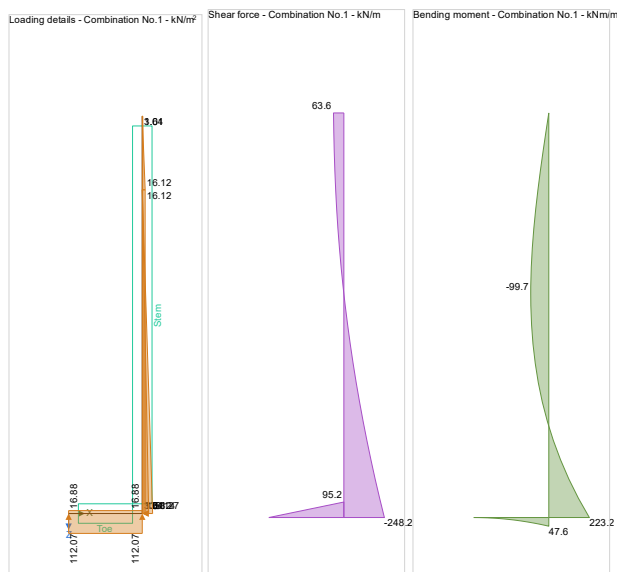
$c_{sr} = 50 \text{ mm}$

Top face of base

$c_{bt} = 50 \text{ mm}$

Bottom face of base

$c_{bb} = 75 \text{ mm}$



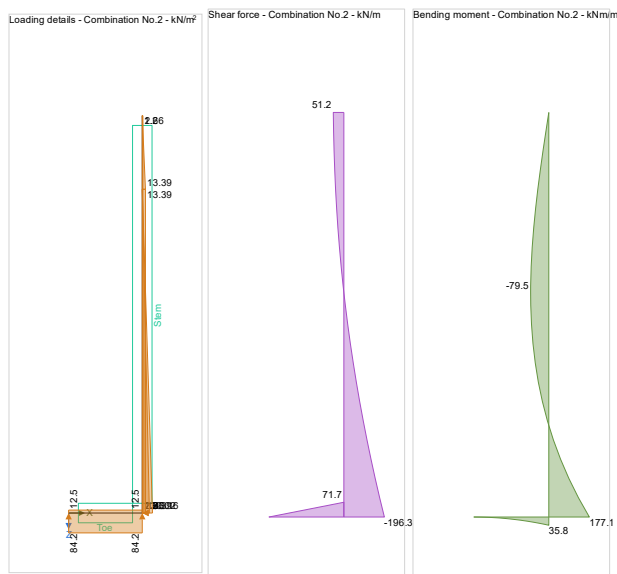


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Check stem design at 2735 mm

Depth of section

$h = 400 \text{ mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$M = 99.7 \text{ kNm/m}$

Depth to tension reinforcement

$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = 336 \text{ mm}$

$K = M / (d^2 \times f_{ck}) = 0.028$

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

$K' = 0.207$

$K' > K$ - No compression reinforcement is required

Lever arm

$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 319 \text{ mm}$

Depth of neutral axis

$x = 2.5 \times (d - z) = 42 \text{ mm}$

Area of tension reinforcement required

$A_{sfM,req} = M / (f_{yd} \times z) = 718 \text{ mm}^2/\text{m}$

Tension reinforcement provided

16 dia.bars @ 100 c/c

Area of tension reinforcement provided

$A_{sfM,prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = 2011 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N

$A_{sfM,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 528 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3)

$A_{sfM,max} = 0.04 \times h = 16000 \text{ mm}^2/\text{m}$

$\max(A_{sfM,req}, A_{sfM,min}) / A_{sfM,prov} = 0.357$

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

$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = 0.006$

Required tension reinforcement ratio

$\rho = A_{sfM,req} / d = 0.002$

Required compression reinforcement ratio

$\rho' = A_{sfM,2,req} / d_2 = 0.000$

Structural system factor - Table 7.4N

$K_b = 1$

Reinforcement factor - exp.7.17

$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sfM,req} / A_{sfM,prov}), 1.5) = 1.5$

Limiting span to depth ratio - exp.7.16.a

$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 40$

Actual span to depth ratio

$h_{prop} / d = 16.1$

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



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Crack control - Section 7.3

Limiting crack width

$$w_{\max} = 0.3 \text{ mm}$$

Variable load factor - EN1990 – Table A1.1

$$\psi_2 = 0.6$$

Serviceability bending moment

$$M_{\text{sls}} = 71.8 \text{ kNm/m}$$

Tensile stress in reinforcement

$$\sigma_s = M_{\text{sls}} / (A_{\text{sfM,prov}} \times z) = 111.9 \text{ N/mm}^2$$

Load duration

Long term

Load duration factor

$$k_t = 0.4$$

Effective area of concrete in tension

$$A_{\text{c,eff}} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$$

$$A_{\text{c,eff}} = 119333 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength

$$f_{\text{ct,eff}} = f_{\text{ctm}} = 3.0 \text{ N/mm}^2$$

Reinforcement ratio

$$\rho_{\text{p,eff}} = A_{\text{sfM,prov}} / A_{\text{c,eff}} = 0.017$$

Modular ratio

$$\alpha_e = E_s / E_{\text{cm}} = 5.998$$

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

$$s_{\text{r,max}} = k_3 \times c_{\text{sr}} + k_1 \times k_2 \times k_4 \times \phi_{\text{sfM}} / \rho_{\text{p,eff}} = 297 \text{ mm}$$

Maximum crack width - exp.7.8

$$w_k = s_{\text{r,max}} \times \max(\sigma_s - k_t \times (f_{\text{ct,eff}} / \rho_{\text{p,eff}}) \times (1 + \alpha_e \times \rho_{\text{p,eff}}), 0.6 \times \sigma_s) / E_s$$

$$w_k = 0.1 \text{ mm}$$

$$w_k / w_{\max} = 0.333$$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section

$$h = 400 \text{ mm}$$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$$M = 223.2 \text{ kNm/m}$$

Depth to tension reinforcement

$$d = h - c_{\text{sr}} - \phi_{\text{sr}} / 2 = 338 \text{ mm}$$

$$K = M / (d^2 \times f_{\text{ck}}) = 0.061$$

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

$$K' = 0.207$$

$K' > K$ - No compression reinforcement is required

Lever arm

$$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{\text{cc}} / \gamma_c))^{0.5}, 0.95) \times d = 318 \text{ mm}$$

Depth of neutral axis

$$x = 2.5 \times (d - z) = 48 \text{ mm}$$

Area of tension reinforcement required

$$A_{\text{sr,req}} = M / (f_{\text{yd}} \times z) = 1614 \text{ mm}^2/\text{m}$$

Tension reinforcement provided

$$25 \text{ dia. bars @ } 100 \text{ c/c}$$

Area of tension reinforcement provided

$$A_{\text{sr,prov}} = \pi \times \phi_{\text{sr}}^2 / (4 \times s_{\text{sr}}) = 4909 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement - exp.9.1N

$$A_{\text{sr,min}} = \max(0.26 \times f_{\text{ctm}} / f_{\text{yk}}, 0.0013) \times d = 531 \text{ mm}^2/\text{m}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{\text{sr,max}} = 0.04 \times h = 16000 \text{ mm}^2/\text{m}$$

$$\max(A_{\text{sr,req}}, A_{\text{sr,min}}) / A_{\text{sr,prov}} = 0.329$$

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

$$\rho_0 = \sqrt{(f_{\text{ck}} / 1 \text{ N/mm}^2)} / 1000 = 0.006$$

Required tension reinforcement ratio

$$\rho = A_{\text{sr,req}} / d = 0.005$$

Required compression reinforcement ratio

$$\rho' = A_{\text{sr,2,req}} / d_2 = 0.000$$

Structural system factor - Table 7.4N

$$K_b = 1$$

Reinforcement factor - exp.7.17

$$K_s = \min(500 \text{ N/mm}^2 / (f_{\text{yk}} \times A_{\text{sr,req}} / A_{\text{sr,prov}}), 1.5) = 1.5$$



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Limiting span to depth ratio - exp.7.16.a

$$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{f_{ck} / 1 \text{ N/mm}^2}] \times \rho_0 / \rho + 3.2 \times \sqrt{f_{ck} / 1 \text{ N/mm}^2}) \times (\rho_0 / \rho - 1)^{3/2}, 40 \times K_b) = \mathbf{33.7}$$

Actual span to depth ratio

$$h_{prop} / d = \mathbf{16}$$

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

Crack control - Section 7.3

Limiting crack width

$$w_{max} = \mathbf{0.3 \text{ mm}}$$

Variable load factor - EN1990 – Table A1.1

$$\psi_2 = \mathbf{0.6}$$

Serviceability bending moment

$$M_{sls} = \mathbf{161.6 \text{ kNm/m}}$$

Tensile stress in reinforcement

$$\sigma_s = M_{sls} / (A_{sr,prov} \times z) = \mathbf{103.4 \text{ N/mm}^2}$$

Load duration

Long term

Load duration factor

$$k_t = \mathbf{0.4}$$

Effective area of concrete in tension

$$A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$$

$$A_{c,eff} = \mathbf{117213 \text{ mm}^2/\text{m}}$$

Mean value of concrete tensile strength

$$f_{ct,eff} = f_{ctm} = \mathbf{3.0 \text{ N/mm}^2}$$

Reinforcement ratio

$$\rho_{p,eff} = A_{sr,prov} / A_{c,eff} = \mathbf{0.042}$$

Modular ratio

$$\alpha_e = E_s / E_{cm} = \mathbf{5.998}$$

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

$$s_{r,max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p,eff} = \mathbf{271 \text{ mm}}$$

Maximum crack width - exp.7.8

$$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$$

$$w_k = \mathbf{0.091 \text{ mm}}$$

$$w_k / w_{max} = \mathbf{0.305}$$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$$V = \mathbf{248.2 \text{ kN/m}}$$

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

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.770}$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{sr,prov} / d, 0.02) = \mathbf{0.015}$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.466 \text{ N/mm}^2}$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = \mathbf{257.8 \text{ kN/m}}$$

$$V / V_{Rd,c} = \mathbf{0.963}$$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section

$$h = \mathbf{400 \text{ mm}}$$

Rectangular section in shear - Section 6.2

Design shear force

$$V = \mathbf{63.6 \text{ kN/m}}$$

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

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.770}$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{sr1,prov} / d, 0.02) = \mathbf{0.006}$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.466 \text{ N/mm}^2}$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = \mathbf{191.5 \text{ kN/m}}$$



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$$V / V_{Rd,c} = 0.332$$

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) $A_{sx,req} = \max(0.25 \times A_{sr,prov}, 0.001 \times t_{stem}) = 1227 \text{ mm}^2/\text{m}$

Maximum spacing of reinforcement – cl.9.6.3(2) $s_{sx,max} = 400 \text{ mm}$

Transverse reinforcement provided 16 dia.bars @ 100 c/c

Area of transverse reinforcement provided $A_{sx,prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 2011 \text{ mm}^2/\text{m}$

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

Check base design at toe

Depth of section $h = 500 \text{ mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = 47.6 \text{ kNm/m}$

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 = 417 \text{ mm}$

$$K = M / (d^2 \times f_{ck}) = 0.009$$

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

$$K' = 0.207$$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 396 \text{ mm}$

Depth of neutral axis $x = 2.5 \times (d - z) = 52 \text{ mm}$

Area of tension reinforcement required $A_{bb,req} = M / (f_{yd} \times z) = 276 \text{ mm}^2/\text{m}$

Tension reinforcement provided 16 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{bb,prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 2011 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N $A_{bb,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 656 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb,max} = 0.04 \times h = 20000 \text{ mm}^2/\text{m}$

$$\max(A_{bb,req}, A_{bb,min}) / A_{bb,prov} = 0.326$$

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 $w_{max} = 0.3 \text{ mm}$

Variable load factor - EN1990 – Table A1.1 $\psi_2 = 0.6$

Serviceability bending moment $M_{sls} = 34.9 \text{ kNm/m}$

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb,prov} \times z) = 43.8 \text{ N/mm}^2$

Load duration Long term

Load duration factor $k_t = 0.4$

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

$$A_{c,eff} = 149292 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$

Reinforcement ratio $\rho_{p,eff} = A_{bb,prov} / A_{c,eff} = 0.013$

Modular ratio $\alpha_e = E_s / E_{cm} = 5.998$

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 $s_{r,max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p,eff} = 457 \text{ mm}$

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

$$w_k = 0.06 \text{ mm}$$



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$$w_k / w_{\max} = 0.2$$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$$V = 95.2 \text{ kN/m}$$

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

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.693$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{bb,prov} / d, 0.02) = 0.005$$

$$v_{\min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.436 \text{ N/mm}^2$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{\min}) \times d$$

$$V_{Rd,c} = 210.8 \text{ kN/m}$$

$$V / V_{Rd,c} = 0.451$$

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)

$$A_{bx,req} = 0.2 \times A_{bb,prov} = 402 \text{ mm}^2/\text{m}$$

Maximum spacing of reinforcement – cl.9.3.1.1(3)

$$s_{bx,max} = 450 \text{ mm}$$

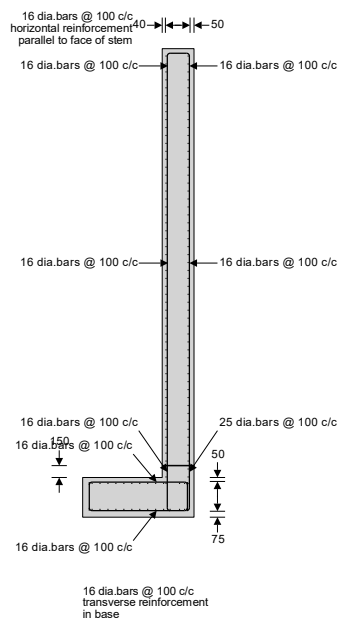
Transverse reinforcement provided

$$16 \text{ dia. bars @ } 100 \text{ c/c}$$

Area of transverse reinforcement provided

$$A_{bx,prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 2011 \text{ mm}^2/\text{m}$$

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



Reinforcement details



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RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.17

Retaining wall details

Stem type	Propped cantilever
Stem height	$h_{\text{stem}} = 3700 \text{ mm}$
Prop height	$h_{\text{prop}} = 3700 \text{ mm}$
Stem thickness	$t_{\text{stem}} = 400 \text{ mm}$
Angle to rear face of stem	$\alpha = 90 \text{ deg}$
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	$l_{\text{toe}} = 1000 \text{ mm}$
Base thickness	$t_{\text{base}} = 500 \text{ mm}$
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	$h_{\text{ret}} = 3700 \text{ mm}$
Angle of soil surface	$\beta = 0 \text{ deg}$
Depth of cover	$d_{\text{cover}} = 0 \text{ mm}$
Height of water	$h_{\text{water}} = 2700 \text{ mm}$
Water density	$\gamma_w = 9.8 \text{ kN/m}^3$

Retained soil properties

Soil type	Firm clay
Moist density	$\gamma_{\text{mr}} = 20 \text{ kN/m}^3$
Saturated density	$\gamma_{\text{sr}} = 20 \text{ kN/m}^3$
Characteristic effective shear resistance angle	$\phi'_{r,k} = 23 \text{ deg}$
Characteristic wall friction angle	$\delta_{r,k} = 11.5 \text{ deg}$

Base soil properties

Soil type	Firm clay
Soil density	$\gamma_b = 20 \text{ kN/m}^3$
Characteristic effective shear resistance angle	$\phi'_{b,k} = 23 \text{ deg}$
Characteristic wall friction angle	$\delta_{b,k} = 11.5 \text{ deg}$
Characteristic base friction angle	$\delta_{bb,k} = 13.5 \text{ deg}$
Presumed bearing capacity	$P_{\text{bearing}} = 250 \text{ kN/m}^2$

Loading details

Permanent surcharge load	Surcharge _G = 2.2 kN/m ²	Surcharge from 137 Cranley Gardens, Wall 1 + Wall 2
Variable surcharge load	Surcharge _Q = 3.5 kN/m ²	Surcharge from 137 Cranley Gardens, Wall 1 + Wall 2 + 2.5 kN/m ²
Vertical line load at 1200 mm	$P_{G1} = 20 \text{ kN/m}$ $P_{Q1} = 5 \text{ kN/m}$	Ground floor slab. Span 1.9m



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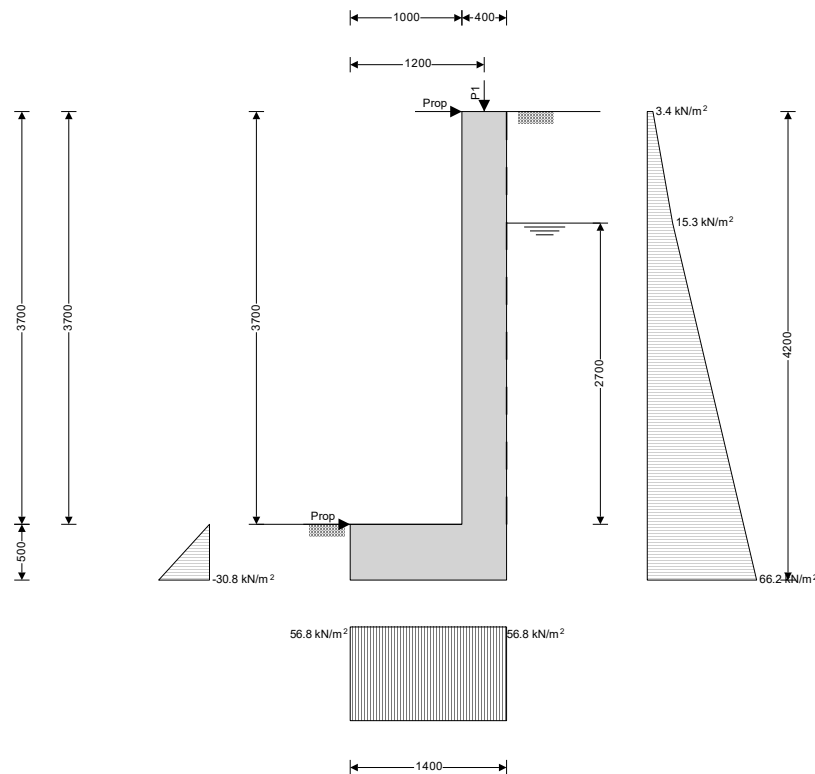
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General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length

$$l_{\text{base}} = l_{\text{toe}} + t_{\text{stem}} = 1400 \text{ mm}$$

Saturated soil height

$$h_{\text{sat}} = h_{\text{water}} + d_{\text{cover}} = 2700 \text{ mm}$$

Moist soil height

$$h_{\text{moist}} = h_{\text{ret}} - h_{\text{water}} = 1000 \text{ mm}$$

Length of surcharge load

$$l_{\text{sur}} = l_{\text{heel}} = 0 \text{ mm}$$

- Distance to vertical component

$$x_{\text{sur}_v} = l_{\text{base}} - l_{\text{heel}} / 2 = 1400 \text{ mm}$$

Effective height of wall

$$h_{\text{eff}} = h_{\text{base}} + d_{\text{cover}} + h_{\text{ret}} = 4200 \text{ mm}$$

- Distance to horizontal component

$$x_{\text{sur}_h} = h_{\text{eff}} / 2 = 2100 \text{ mm}$$

Area of wall stem

$$A_{\text{stem}} = h_{\text{stem}} \times t_{\text{stem}} = 1.48 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{stem}} = l_{\text{toe}} + t_{\text{stem}} / 2 = 1200 \text{ mm}$$

Area of wall base

$$A_{\text{base}} = l_{\text{base}} \times t_{\text{base}} = 0.7 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{base}} = l_{\text{base}} / 2 = 700 \text{ mm}$$

Using Coulomb theory

At rest pressure coefficient

$$K_0 = 1 - \sin(\phi'_{r,k}) = 0.609$$

Passive pressure coefficient

$$K_P = \sin(90 - \phi'_{b,k})^2 / (\sin(90 + \delta_{b,k}) \times [1 - \sqrt{[\sin(\phi'_{b,k} + \delta_{b,k}) \times \sin(\phi'_{b,k}) / (\sin(90 + \delta_{b,k}))]}]^2) = 3.140$$

Bearing pressure check

Vertical forces on wall

Wall stem

$$F_{\text{stem}} = A_{\text{stem}} \times \gamma_{\text{stem}} = 37 \text{ kN/m}$$

Wall base

$$F_{\text{base}} = A_{\text{base}} \times \gamma_{\text{base}} = 17.5 \text{ kN/m}$$

Line loads

$$F_{P_v} = P_{G1} + P_{Q1} = 25 \text{ kN/m}$$



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Total

$$F_{\text{total}_v} = F_{\text{stem}} + F_{\text{base}} + F_{P_v} + F_{\text{water}_v} = 79.5 \text{ kN/m}$$

Horizontal forces on wall

Surcharge load

$$F_{\text{sur}_h} = K_0 \times \cos(\delta_{r,k}) \times (\text{Surcharge}_G + \text{Surcharge}_Q) \times h_{\text{eff}} = 14.3 \text{ kN/m}$$

Saturated retained soil

$$F_{\text{sat}_h} = K_0 \times \cos(\delta_{r,k}) \times (\gamma_{sr} - \gamma_w) \times (h_{\text{sat}} + h_{\text{base}})^2 / 2 = 31.1 \text{ kN/m}$$

Water

$$F_{\text{water}_h} = \gamma_w \times (h_{\text{water}} + d_{\text{cover}} + h_{\text{base}})^2 / 2 = 50.2 \text{ kN/m}$$

Moist retained soil

$$F_{\text{moist}_h} = K_0 \times \cos(\delta_{r,k}) \times \gamma_{mr} \times ((h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}})^2 / 2 + (h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}}) \times (h_{\text{sat}} + h_{\text{base}})) = 44.2 \text{ kN/m}$$

Base soil

$$F_{\text{pass}_h} = -K_P \times \cos(\delta_{b,k}) \times \gamma_b \times (d_{\text{cover}} + h_{\text{base}})^2 / 2 = -7.7 \text{ kN/m}$$

Total

$$F_{\text{total}_h} = F_{\text{sur}_h} + F_{\text{sat}_h} + F_{\text{water}_h} + F_{\text{moist}_h} + F_{\text{pass}_h} = 132.2 \text{ kN/m}$$

Moments on wall

Wall stem

$$M_{\text{stem}} = F_{\text{stem}} \times X_{\text{stem}} = 44.4 \text{ kNm/m}$$

Wall base

$$M_{\text{base}} = F_{\text{base}} \times X_{\text{base}} = 12.3 \text{ kNm/m}$$

Surcharge load

$$M_{\text{sur}} = -F_{\text{sur}_h} \times X_{\text{sur}_h} = -30 \text{ kNm/m}$$

Line loads

$$M_P = (P_{G1} + P_{Q1}) \times p_1 = 30 \text{ kNm/m}$$

Saturated retained soil

$$M_{\text{sat}} = -F_{\text{sat}_h} \times X_{\text{sat}_h} = -33.2 \text{ kNm/m}$$

Water

$$M_{\text{water}} = -F_{\text{water}_h} \times X_{\text{water}_h} = -53.6 \text{ kNm/m}$$

Moist retained soil

$$M_{\text{moist}} = -F_{\text{moist}_h} \times X_{\text{moist}_h} = -82.2 \text{ kNm/m}$$

Total

$$M_{\text{total}} = M_{\text{stem}} + M_{\text{base}} + M_{\text{sur}} + M_P + M_{\text{sat}} + M_{\text{water}} + M_{\text{moist}} = -112.4 \text{ kNm/m}$$

Check bearing pressure

Propping force to stem

$$F_{\text{prop}_\text{stem}} = (F_{\text{total}_v} \times l_{\text{base}} / 2 - M_{\text{total}}) / (h_{\text{prop}} + t_{\text{base}}) = 40 \text{ kN/m}$$

Propping force to base

$$F_{\text{prop}_\text{base}} = F_{\text{total}_h} - F_{\text{prop}_\text{stem}} = 92.1 \text{ kN/m}$$

Moment from propping force

$$M_{\text{prop}} = F_{\text{prop}_\text{stem}} \times (h_{\text{prop}} + t_{\text{base}}) = 168 \text{ kNm/m}$$

Distance to reaction

$$\bar{x} = (M_{\text{total}} + M_{\text{prop}}) / F_{\text{total}_v} = 700 \text{ mm}$$

Eccentricity of reaction

$$e = \bar{x} - l_{\text{base}} / 2 = 0 \text{ mm}$$

Loaded length of base

$$l_{\text{load}} = l_{\text{base}} = 1400 \text{ mm}$$

Bearing pressure at toe

$$q_{\text{toe}} = F_{\text{total}_v} / l_{\text{base}} \times (1 - 6 \times e / l_{\text{base}}) = 56.8 \text{ kN/m}^2$$

Bearing pressure at heel

$$q_{\text{heel}} = F_{\text{total}_v} / l_{\text{base}} \times (1 + 6 \times e / l_{\text{base}}) = 56.8 \text{ kN/m}^2$$

Factor of safety

$$FoS_{bp} = P_{\text{bearing}} / \max(q_{\text{toe}}, q_{\text{heel}}) = 4.403$$

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 using user defined factors

Tedds calculation version 2.9.17

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

Concrete strength class

C32/40

Characteristic compressive cylinder strength

$$f_{ck} = 32 \text{ N/mm}^2$$

Characteristic compressive cube strength

$$f_{ck,\text{cube}} = 40 \text{ N/mm}^2$$

Mean value of compressive cylinder strength

$$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 40 \text{ N/mm}^2$$

Mean value of axial tensile strength

$$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 3.0 \text{ N/mm}^2$$

5% fractile of axial tensile strength

$$f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.1 \text{ N/mm}^2$$

Secant modulus of elasticity of concrete

$$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = 33346 \text{ N/mm}^2$$

Partial factor for concrete - Table 2.1N

$$\gamma_C = 1.50$$

Compressive strength coefficient - cl.3.1.6(1)

$$\alpha_{cc} = 0.85$$

Design compressive concrete strength - exp.3.15

$$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 18.1 \text{ N/mm}^2$$

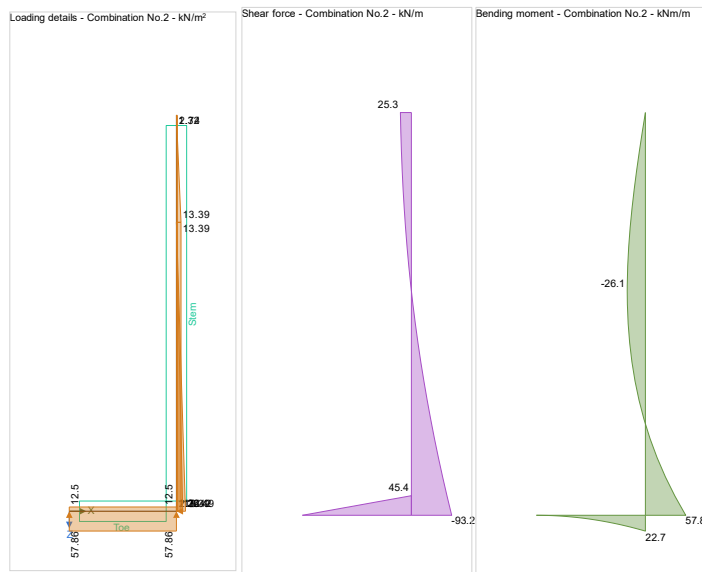


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Check stem design at 1802 mm

Depth of section

$h = 400 \text{ mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$M = 32.3 \text{ kNm/m}$

Depth to tension reinforcement

$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = 336 \text{ mm}$

$K = M / (d^2 \times f_{ck}) = 0.009$

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

$K' = 0.207$

$K' > K$ - No compression reinforcement is required

Lever arm

$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 319 \text{ mm}$

Depth of neutral axis

$x = 2.5 \times (d - z) = 42 \text{ mm}$

Area of tension reinforcement required

$A_{sfM,req} = M / (f_{yd} \times z) = 233 \text{ mm}^2/\text{m}$

Tension reinforcement provided

16 dia.bars @ 100 c/c

Area of tension reinforcement provided

$A_{sfM,prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = 2011 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N

$A_{sfM,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 528 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3)

$A_{sfM,max} = 0.04 \times h = 16000 \text{ mm}^2/\text{m}$

$\max(A_{sfM,req}, A_{sfM,min}) / A_{sfM,prov} = 0.263$

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

$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = 0.006$

Required tension reinforcement ratio

$\rho = A_{sfM,req} / d = 0.001$

Required compression reinforcement ratio

$\rho' = A_{sfM,2,req} / d_2 = 0.000$

Structural system factor - Table 7.4N

$K_b = 1$

Reinforcement factor - exp.7.17

$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sfM,req} / A_{sfM,prov}), 1.5) = 1.5$

Limiting span to depth ratio - exp.7.16.a

$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 40$

Actual span to depth ratio

$h_{prop} / d = 11$

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



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Crack control - Section 7.3

Limiting crack width

$$w_{\max} = 0.3 \text{ mm}$$

Variable load factor - EN1990 – Table A1.1

$$\psi_2 = 0.6$$

Serviceability bending moment

$$M_{\text{sls}} = 22.9 \text{ kNm/m}$$

Tensile stress in reinforcement

$$\sigma_s = M_{\text{sls}} / (A_{\text{sfM,prov}} \times z) = 35.7 \text{ N/mm}^2$$

Load duration

Long term

Load duration factor

$$k_t = 0.4$$

Effective area of concrete in tension

$$A_{\text{c,eff}} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$$

$$A_{\text{c,eff}} = 119333 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength

$$f_{\text{ct,eff}} = f_{\text{ctm}} = 3.0 \text{ N/mm}^2$$

Reinforcement ratio

$$\rho_{\text{p,eff}} = A_{\text{sfM,prov}} / A_{\text{c,eff}} = 0.017$$

Modular ratio

$$\alpha_e = E_s / E_{\text{cm}} = 5.998$$

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

$$s_{\text{r,max}} = k_3 \times c_{\text{sr}} + k_1 \times k_2 \times k_4 \times \phi_{\text{sfM}} / \rho_{\text{p,eff}} = 297 \text{ mm}$$

Maximum crack width - exp.7.8

$$w_k = s_{\text{r,max}} \times \max(\sigma_s - k_t \times (f_{\text{ct,eff}} / \rho_{\text{p,eff}}) \times (1 + \alpha_e \times \rho_{\text{p,eff}}), 0.6 \times \sigma_s) / E_s$$

$$w_k = 0.032 \text{ mm}$$

$$w_k / w_{\max} = 0.106$$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section

$$h = 400 \text{ mm}$$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$$M = 72.2 \text{ kNm/m}$$

Depth to tension reinforcement

$$d = h - c_{\text{sr}} - \phi_{\text{sr}} / 2 = 338 \text{ mm}$$

$$K = M / (d^2 \times f_{\text{ck}}) = 0.020$$

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

$$K' = 0.207$$

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{\text{cc}} / \gamma_c))^{0.5}, 0.95) \times d = 321 \text{ mm}$$

Depth of neutral axis

$$x = 2.5 \times (d - z) = 42 \text{ mm}$$

Area of tension reinforcement required

$$A_{\text{sr,req}} = M / (f_{\text{yd}} \times z) = 518 \text{ mm}^2/\text{m}$$

Tension reinforcement provided

25 dia.bars @ 100 c/c

Area of tension reinforcement provided

$$A_{\text{sr,prov}} = \pi \times \phi_{\text{sr}}^2 / (4 \times s_{\text{sr}}) = 4909 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement - exp.9.1N

$$A_{\text{sr,min}} = \max(0.26 \times f_{\text{ctm}} / f_{\text{yk}}, 0.0013) \times d = 531 \text{ mm}^2/\text{m}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{\text{sr,max}} = 0.04 \times h = 16000 \text{ mm}^2/\text{m}$$

$$\max(A_{\text{sr,req}}, A_{\text{sr,min}}) / A_{\text{sr,prov}} = 0.108$$

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

$$\rho_0 = \sqrt{(f_{\text{ck}} / 1 \text{ N/mm}^2)} / 1000 = 0.006$$

Required tension reinforcement ratio

$$\rho = A_{\text{sr,req}} / d = 0.002$$

Required compression reinforcement ratio

$$\rho' = A_{\text{sr,2,req}} / d_2 = 0.000$$

Structural system factor - Table 7.4N

$$K_b = 1$$

Reinforcement factor - exp.7.17

$$K_s = \min(500 \text{ N/mm}^2 / (f_{\text{yk}} \times A_{\text{sr,req}} / A_{\text{sr,prov}}), 1.5) = 1.5$$



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Limiting span to depth ratio - exp.7.16.a

$$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{f_{ck} / 1 \text{ N/mm}^2}] \times \rho_0 / \rho + 3.2 \times \sqrt{f_{ck} / 1 \text{ N/mm}^2}) \times (\rho_0 / \rho - 1)^{3/2}, 40 \times K_b) = 40$$

Actual span to depth ratio

$$h_{prop} / d = 11$$

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

Crack control - Section 7.3

Limiting crack width

$$w_{max} = 0.3 \text{ mm}$$

Variable load factor - EN1990 – Table A1.1

$$\psi_2 = 0.6$$

Serviceability bending moment

$$M_{sls} = 51.6 \text{ kNm/m}$$

Tensile stress in reinforcement

$$\sigma_s = M_{sls} / (A_{sr,prov} \times z) = 32.8 \text{ N/mm}^2$$

Load duration

Long term

Load duration factor

$$k_t = 0.4$$

Effective area of concrete in tension

$$A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$$

$$A_{c,eff} = 119271 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength

$$f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$$

Reinforcement ratio

$$\rho_{p,eff} = A_{sr,prov} / A_{c,eff} = 0.041$$

Modular ratio

$$\alpha_e = E_s / E_{cm} = 5.998$$

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

$$s_{r,max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p,eff} = 273 \text{ mm}$$

Maximum crack width - exp.7.8

$$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$$

$$w_k = 0.027 \text{ mm}$$

$$w_k / w_{max} = 0.09$$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$$V = 116.9 \text{ kN/m}$$

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

$$k = \min(1 + \sqrt{200 \text{ mm} / d}, 2) = 1.770$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{sr,prov} / d, 0.02) = 0.015$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.466 \text{ N/mm}^2$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = 257.8 \text{ kN/m}$$

$$V / V_{Rd,c} = 0.453$$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section

$$h = 400 \text{ mm}$$

Rectangular section in shear - Section 6.2

Design shear force

$$V = 31.1 \text{ kN/m}$$

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

$$k = \min(1 + \sqrt{200 \text{ mm} / d}, 2) = 1.770$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{sr1,prov} / d, 0.02) = 0.006$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.466 \text{ N/mm}^2$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = 191.5 \text{ kN/m}$$



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$$V / V_{Rd,c} = 0.162$$

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) $A_{sx,req} = \max(0.25 \times A_{sr,prov}, 0.001 \times t_{stem}) = 1227 \text{ mm}^2/\text{m}$

Maximum spacing of reinforcement – cl.9.6.3(2) $s_{sx,max} = 400 \text{ mm}$

Transverse reinforcement provided 16 dia.bars @ 100 c/c

Area of transverse reinforcement provided $A_{sx,prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 2011 \text{ mm}^2/\text{m}$

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

Check base design at toe

Depth of section $h = 500 \text{ mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = 30.2 \text{ kNm/m}$

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 = 417 \text{ mm}$

$$K = M / (d^2 \times f_{ck}) = 0.005$$

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

$$K' = 0.207$$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 396 \text{ mm}$

Depth of neutral axis $x = 2.5 \times (d - z) = 52 \text{ mm}$

Area of tension reinforcement required $A_{bb,req} = M / (f_{yd} \times z) = 175 \text{ mm}^2/\text{m}$

Tension reinforcement provided 16 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{bb,prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 2011 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N $A_{bb,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 656 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb,max} = 0.04 \times h = 20000 \text{ mm}^2/\text{m}$

$$\max(A_{bb,req}, A_{bb,min}) / A_{bb,prov} = 0.326$$

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 $w_{max} = 0.3 \text{ mm}$

Variable load factor - EN1990 – Table A1.1 $\psi_2 = 0.6$

Serviceability bending moment $M_{sls} = 22.1 \text{ kNm/m}$

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb,prov} \times z) = 27.8 \text{ N/mm}^2$

Load duration Long term

Load duration factor $k_t = 0.4$

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

$$A_{c,eff} = 149292 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$

Reinforcement ratio $\rho_{p,eff} = A_{bb,prov} / A_{c,eff} = 0.013$

Modular ratio $\alpha_e = E_s / E_{cm} = 5.998$

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 $s_{r,max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p,eff} = 457 \text{ mm}$

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

$$w_k = 0.038 \text{ mm}$$

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