APPENDIX B
STRUCTURAL CALCULATIONS

WE ARE SYMMETRYS

STRUCTURAL CALCULATION PACKAGE

70 LADY MARGARET ROAD

22276-SYM-XX-XX-RP-S-0001
MAY 2023
REV B

INTRODUCTION

The structure comprises a four-storey retaining structure and proposed three-storey extension structure. It is proposed to demolish the existing extension, replace by excavting and forming a new one-storey basement with three-storey superstructure, with 2 No. of stair, one to rear garden and one to retaining structure.

CODES OF PRACTICE

The following codes and standards have been used within this document:

BS EN 1990	Eurocode: Basis of structural design
BS EN 1991	Eurocode 1: Action on structures

EXECUTIVE SUMMARY

The proposed development involves the demolition of the existing extension structure. A new basement excavated 1-level underground. Above ground the proposed extension consists of ground plus 3 storeys attached to existing retained structure.

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Vertical Loads

for

70 Lady Margaret Road

Job No.	Sheet No.	Revision
22276	4	1
Date	Made by	Checked By
01/05/2023	KL	EB

SYMMETRYS STRUCTURAL / CIVIL ENGINEERS

Job Title 70 Lady Margaret Road

Section Vertical Loads

Loading

The retaining and propose structure for a multi-level structure. This four-storey building is divided into four sections with different and determine the appropriate load acting on each section.

live load

Area	Qk(kN/m^2)
Floors	1.5
Stairs	2.5
Balconies	2.5

Façade material	Loading (m^2)
Concrete	2.5
Brickwork	1

Proposed Structure

Storey	Height(m)	Floor material	Trib Area (m^2)	Dead Load (kN/m^2)	Live Load (kNm^2)	SSL (kN/m^2)	UDL(kN/m^2)
New Basement	2.84	250mm RC Slab	38.3	6.25	2	8.25	11.4375
Existing Basement	1.9	250mm RC Slab	19.87	6.25	2	8.25	11.4375
Ground	3.1	250mm RC Slab	131.74	6.25	2	8.25	11.4375
1st	3.1	250mm RC Slab	79.67	6.25	2	8.25	11.4375
2nd	3.1	250mm RC Slab	76.88	6.25	2	8.25	11.4375
3rd	3.1	250mm RC Slab	63.6	6.25	2	8.25	11.4375
Roof	/	250mm RC Slab	131.74	6.25	0.6	6.85	9.3375

Storey	Material	Height	Loading
New Basement	Concrete	2.84	7.1
Existing Basement	Concrete	1.9	4.75
Ground	Brickwork	3.1	3.1
1st	Brickwork	2.7	2.7
2nd	Brickwork	2.3	2.3
3rd	Brickwork	2.3	2.3

Loading Storey	Trib Area (m2)	Total SSL (kN/m^2)	Total USL (kN/m^2)
Basement & Ground floor	37.81	23.35	32.2125
Ground floor	9.56	15.1	20.775
Ground floor to 2nd floor	12.72	31.6	43.65
Ground to 3rd floor	70.89	39.85	55.0875
Basement to 3rd floor	19.87	48.1	66.525
Existing Stairs	4.87	47.5	65.625
New Stairs	1.98	23.75	32.8125

Wall	Loading Storey	Length	Façade Load (kN/m)
A	GF	3.3	3.1
B	GF	0.9	3.1
C	B1+GF	3.5	10.2
D	B1	2.7	7.1
E	B1+GF	6.8	10.2
F	B1	4.7	7.1
G	GF	7.4	3.1
H	B1	1.8	7.1
I	B1	1.9	7.1
J	B1+GF	2.8	10.2
K	GF+1F+2F	1.5	8.1
L	GF+1F+2F	2.3	8.1

Wall	Loading Storey	Length	Façade Load (kN/m)
M	GF+1F+2F	3.3	8.1
N	GF+1F+2F	3.3	8.1
O	GF+1F+2F+3F	2.8	10.4
P	GF+1F+2F+3F	3.8	10.4
Q	GF+1F+2F+3F	9.6	10.4
R	GF+1F+2F+3F	2.6	10.4
S	B1+GF+1F+2F+3F	2.3	15.15
T	B1+GF+1F+2F+3F	7.0	15.15
U	B1+GF+1F+2F+3F	8.7	15.15
V	GF+1F+2F+3F	4.4	10.4
W	B1+GF+1F+2F+3F	1.7	15.15
X	B1+GF+1F+2F+3F	2.2	15.15

SYMMETRYS
STRUCTURAL / CIVIL ENGINEERS

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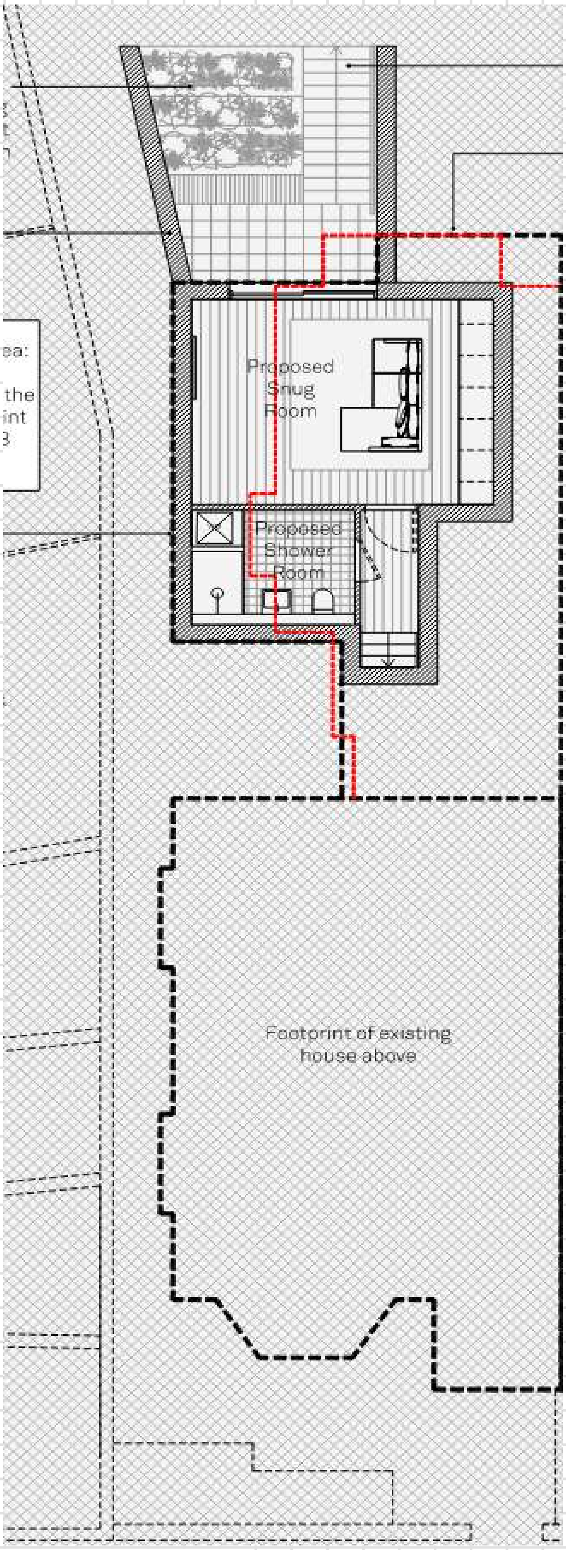
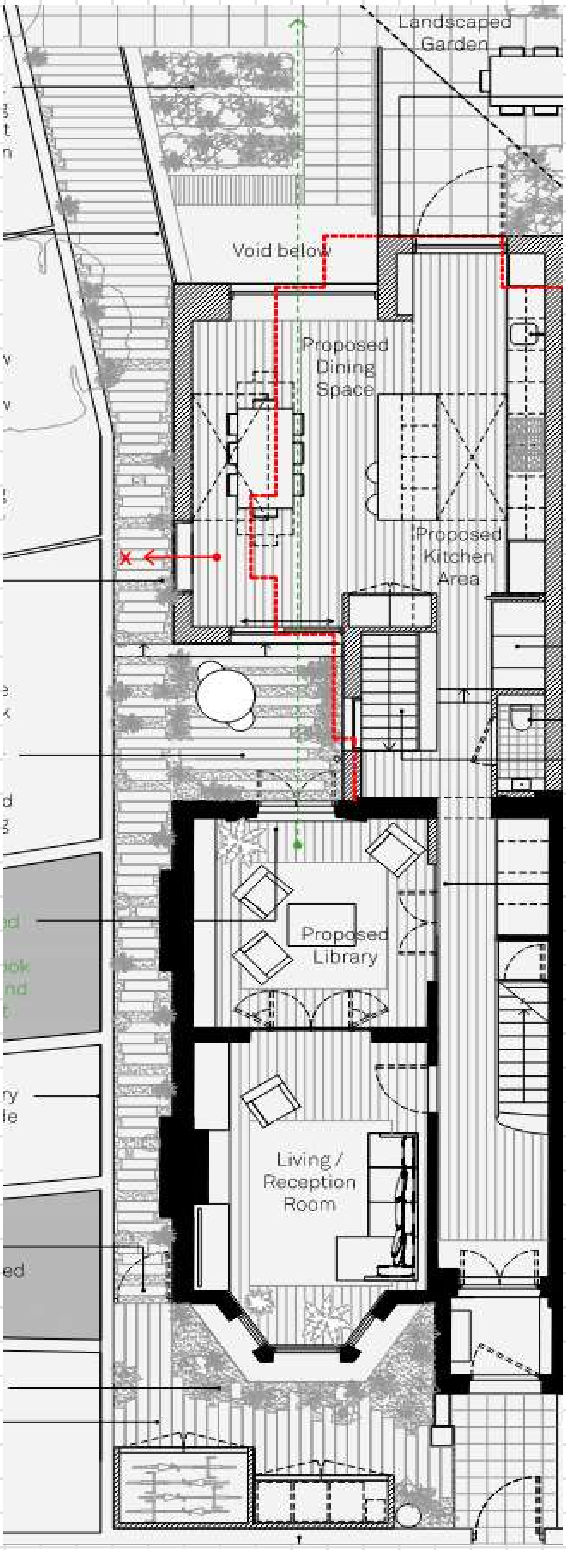
22276	5	1

		Date	Made by	Checked By
Job Title	70 Lady Margaret Road			
Section	Vertical Loads	01/05/2023	KL	EB

Site Plan



Proposed Basement Plan



Proposed Ground Floor Plan

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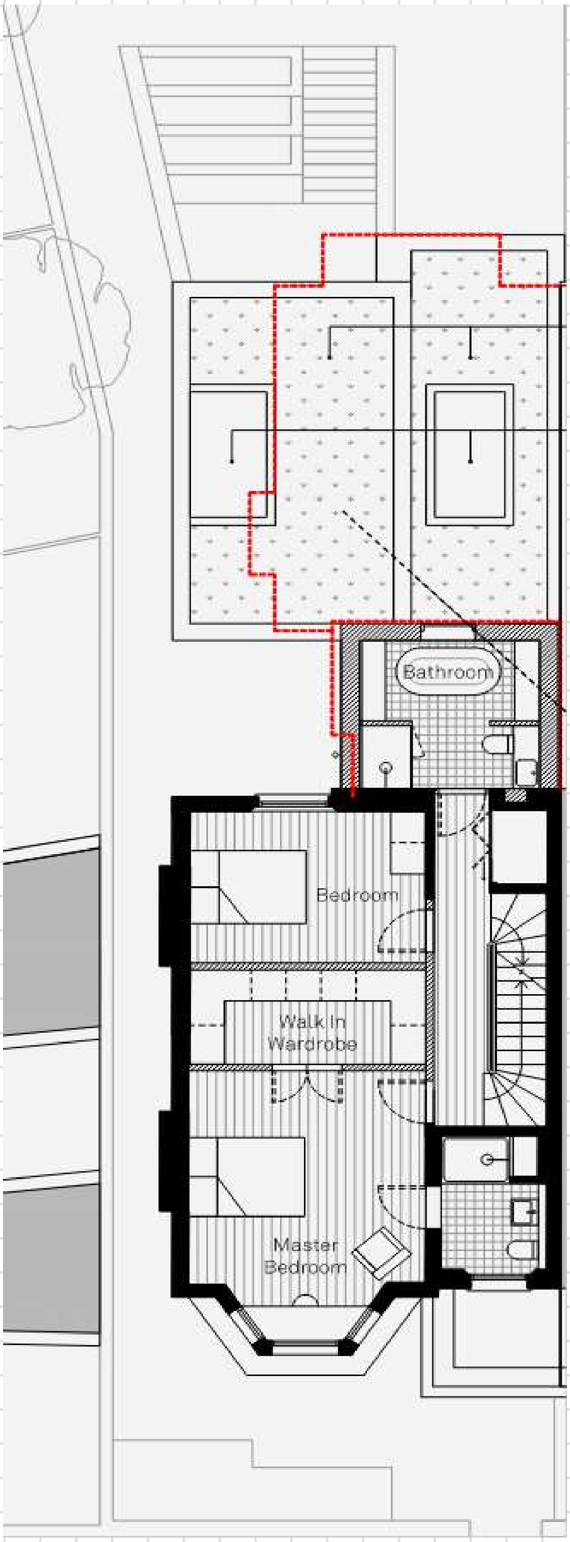
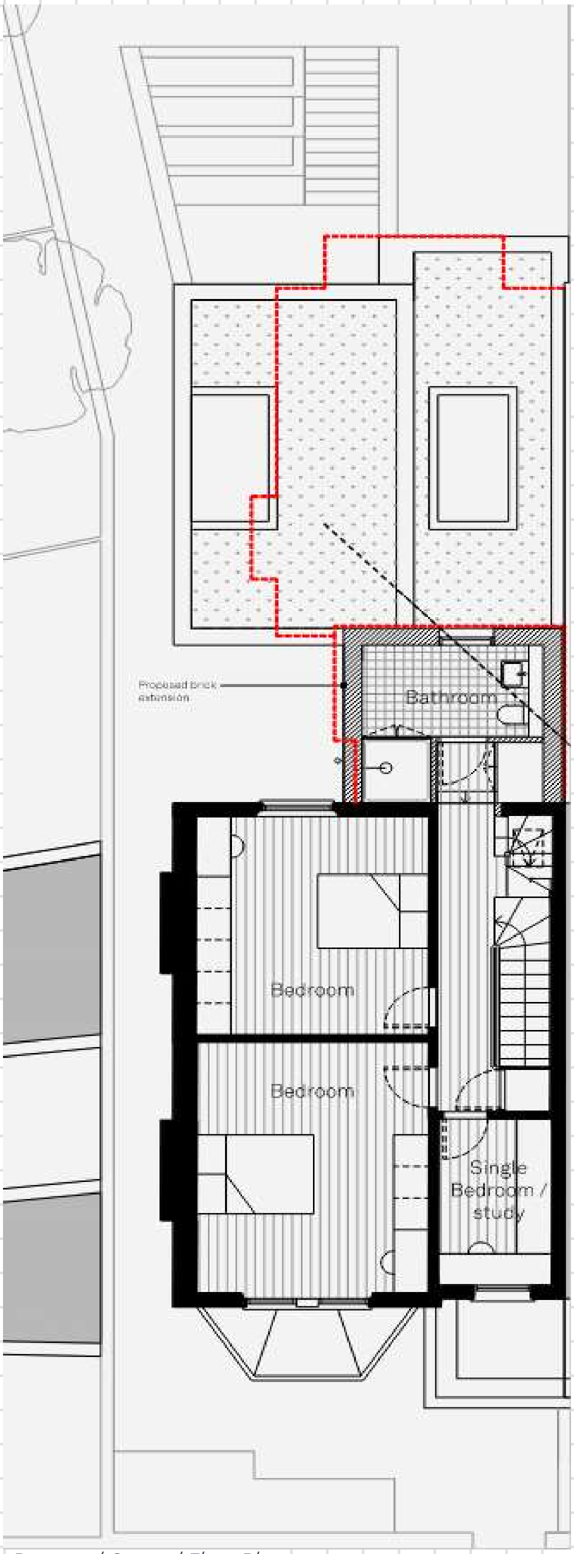
		22276	6	1
Job Title	70 Lady Margaret Road	Date	Made by	Checked By
Section	Vertical Loads	01/05/2023	KL	EB

Site Plan



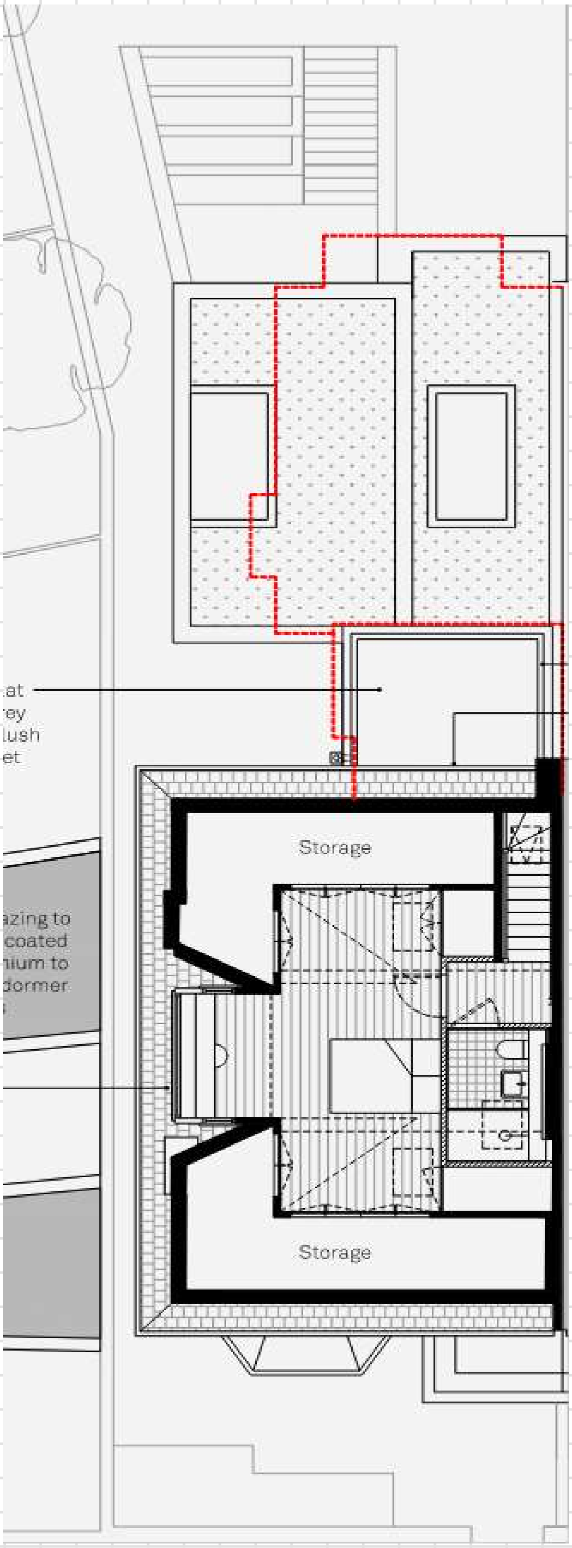
Proposed First Floor Plan



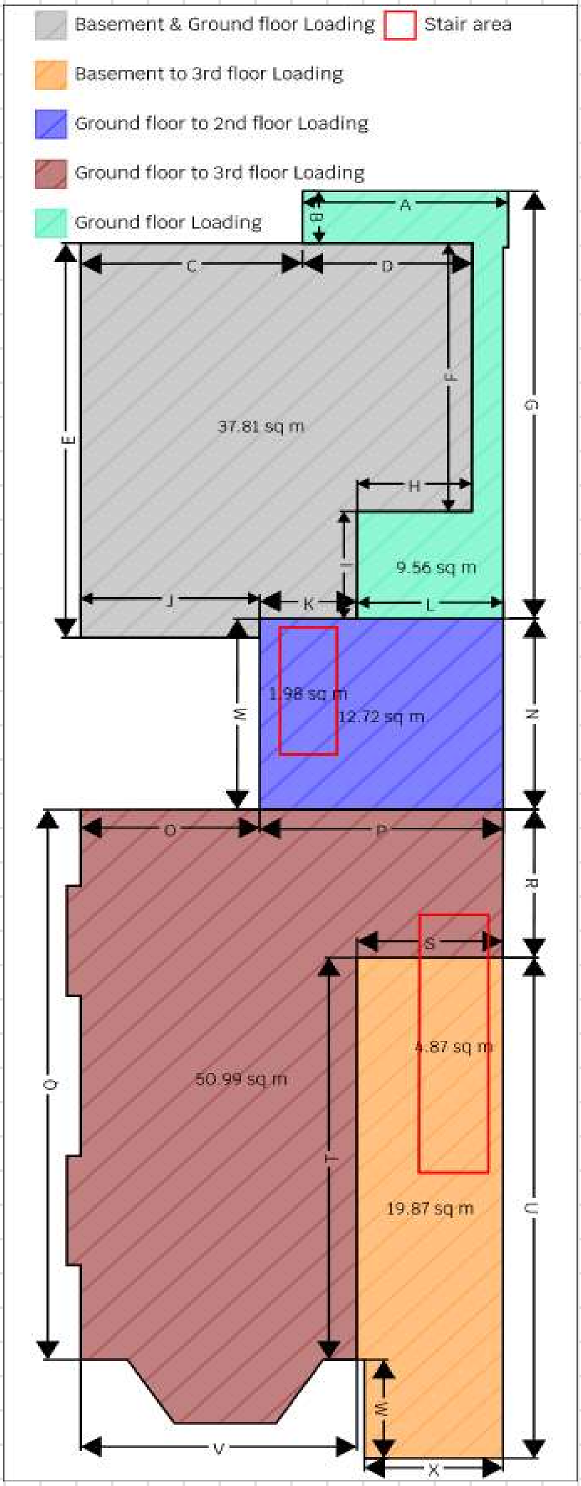
Proposed Second Floor Plan

SYMMETRYS STRUCTURAL / CIVIL ENGINEERS		Job No.	Sheet No.	Revision
		22276	7	1
Job Title	70 Lady Margaret Road	Date	Made by	Checked By
Section	Vertical Loads	01/05/2023	KL	EB

Loading Plan



Proposed Third Floor Plan



Loading section

Structural Calculations

for

70 Lady Margaret Road

Design of a basement slab for heave

<u>Expected heave forces</u>

Detailed analysis has been undertaken by GEA

Approximate conservative heave forces

Excavation depth	=	2.8	m					
Hydroestatic pressure	=	2.8	x	10	=	28	kN/m2	
Oberburden pressure	=	2.8	x	18	=	51	kN/m2	
Therefore expected heave	=	28	+	0.5*(51.12-28.	=	40	kN/m2	

<u>Basement Slab Selfweight</u>

Selfweight	=	0.3	x	25	=	6.3	kN/m2
Screed	=	0.075	x	24	=	1.8	kN/m2
		Total	=			8.1	kN/m2

<u>Uplift Design</u>

Overal Uplift	=	40	-	8.05	=	31.7	kN/m2
Therefore - ULS	=	31.7	x	1.5	=	47.6	kN/m2

The basement slab will be designed to transmit forces to the retaining wall foundtations spanning between both sides of the retaining walls.

The distance between the fondations is = 3.2 m

Therefore: Mmax = $\frac{47.565X(3.2^2)}{8}$ = **60.86kNm**

Design of a baement slab for heave

Slab Properties

Design Moment, M	=	60.86	kNm	fcu	=	30	N/mm^2	$\gamma c =$	1.50
ßb	=	1.00		fy	=	500	N/mm^2	$\gamma \sigma =$	1.15
span	=	3200	mm	Steel Class	=	A			
Slab thickness, h	=	250	mm						
Bar Ø	=	12							
cover	=	50	mm						

Check

d = 250 - 50 - 12/2 = 194.0 mm

K' = 0.156 > K = 0.054 ok

z = 194.0 [0.5 + (0.25 - 0.054 /0.893)]^½ = 181.6 < 0.95d = 184.3 mm

As = 60.86E6 /500 /181.5 x 1.15 = 771 > min As = 325 mm²/m

PROVIDE H12 @ 150 = 754 mm²/m

fs = 2/3 x 500 x 771 /754 /1.00 = 341.0 N/mm²

Tens mod factor = 0.55 + (477 - 341.0) /120 /(0.9 + 1.617) = 1.000

Comp mod factor = 1 + 0.691/(3 + 0.691) = 1.187

Permissible L/d = 20.0 x 1.000 x 1.187 = 23.750

Actual L/d = 3200 /194.0 = 16.495 ok

Therefore:

Deflection is ok

Maximum spacing is ok

Minimum spacing is ok

ELEMENT DESIGN to BS 8110:2005
SOLID SLABS
Originated from RCC11.xls v4.0 © 2006 - 2010 TCC
mpa The Concrete Centre

INPUT Location 70 Lady Margaret Road NW5 2NP
Design moment, M 60.86 kNm/m fcu 30 N/mm² γc = 1.50
ßb 1.00 fy 500 N/mm² γs = 1.15
span 3200 mm steel class A
Height, h 250 mm Section location SIMPLY SUPPORTED SP
Bar Ø 12 mm Compression steel SPECIFY
cover 50 mm to these bars (deflection control only)

ONE or TWO WAY SLAB

OUTPUT 70 Lady Margaret Road NW5 2NP Compression steel = H16@150(0.691%)
d = 250 - 50 - 12/2 = 194.0 mm
(3.4.4.4) K' = 0.156 > K = 0.054 ok
(3.4.4.4) z = 194.0 [0.5 + (0.25 - 0.054 /0.893)]^½ = 181.6 < 0.95d = 184.3 mm
(3.4.4.1) As = 60.86E6 /500 /181.5 x 1.15 = 771 > min As = 325 mm²/m
PROVIDE H12 @ 150 = 754 mm²/m
(Eqn 8) fs = 2/3 x 500 x 771 /754 /1.00 = 341.0 N/mm²
(Eqn 7) Tens mod factor = 0.55 + (477 - 341.0) /120 /(0.9 + 1.617) = 1.000
(Equation 9) Comp mod factor = 1 + 0.691/(3 + 0.691) = 1.187
(3.4.6.3) Permissible L/d = 20.0 x 1.000 x 1.187 = 23.750
Actual L/d = 3200 /194.0 = 16.495 ok

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 2.9.17

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **3000** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **4000** mm
Base thickness	t_{base} = **250** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2000** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Firm clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **22** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **11** deg

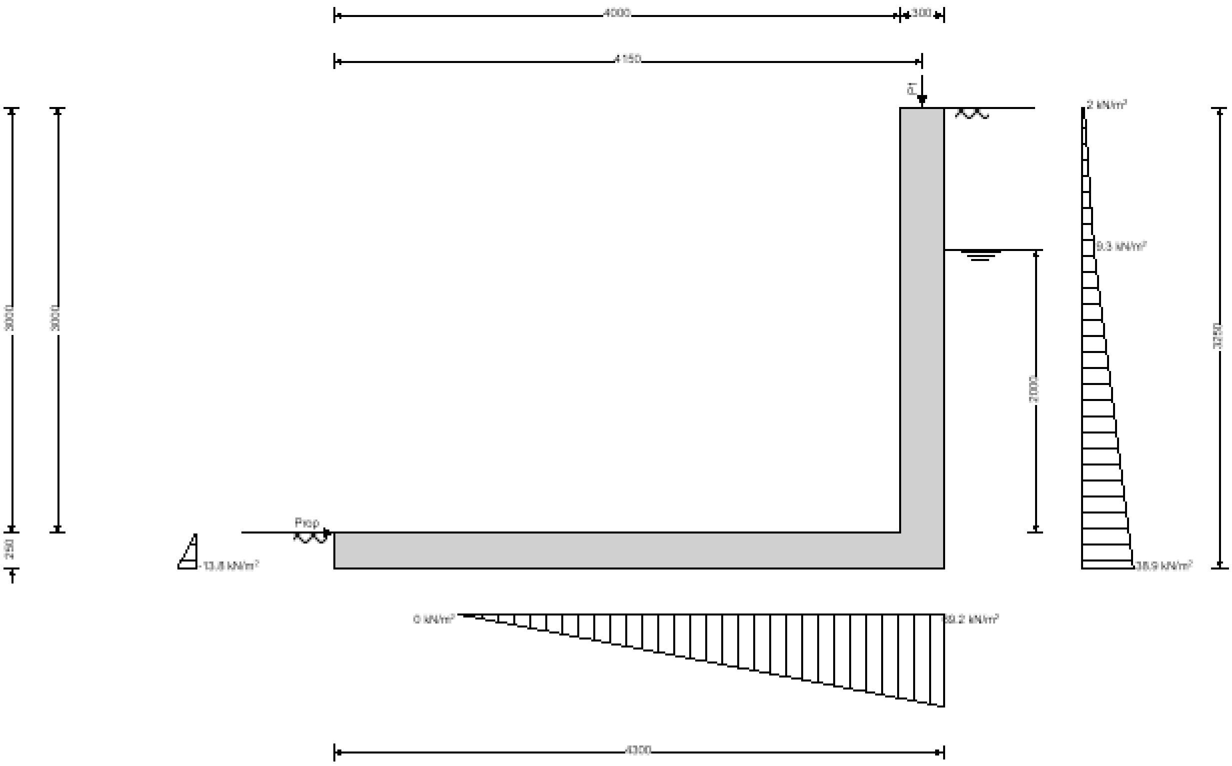
Base soil properties

Soil type	Stiff silty clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **22** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **11** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **14.7** deg
Presumed bearing capacity	$P_{bearing}$ = **150** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 4150 mm	P_{G1} = **60** kN/m
	P_{Q1} = **9** kN/m

Symmetry's Limited Unit 6, The Courtyard Lynton Road	Project 70 Lady Margaret Road				Job no. 22276	
	Calcs for Retaining Wall A				Start page no./Revision 2	
	Calcs by KL	Calcs date 22/05/2023	Checked by EB	Checked date 13/03/2023	Approved by	Approved date



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} = \mathbf{4300}$ mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} = \mathbf{2000}$ mm

Moist soil height $h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{0}$ mm

 - Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{4300}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3250}$ mm

 - Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{1625}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.9}$ m²

 - Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{4150}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{1.075}$ m²

 - Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{2150}$ mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.413}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.958}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{22.5}$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{26.9}$ kN/m

Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{69}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{118.4}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{6.6}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{8.4}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{24.8}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{20.1}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-1.7}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{58.2}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{93.4}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{57.8}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-10.7}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{286.4}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-6.3}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-18.6}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-27.9}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{374}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{58.2}$ kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{3159}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{1009}$ mm
Loaded length of base	$l_{load} = 3 \times (l_{base} - \bar{x}) = \mathbf{3423}$ mm
Bearing pressure at toe	$q_{toe} = \mathbf{0}$ kN/m^2
Bearing pressure at heel	$q_{heel} = 2 \times F_{total\_v} / l_{load} = \mathbf{69.2}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{2.169}$

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 2.9.17

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ 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	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm

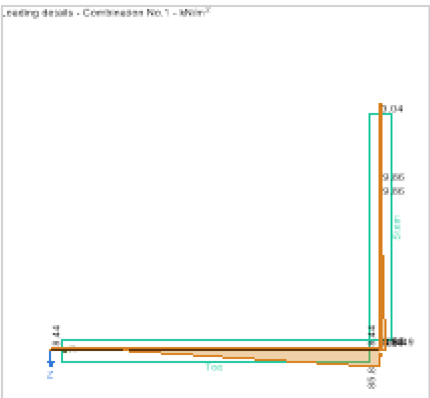
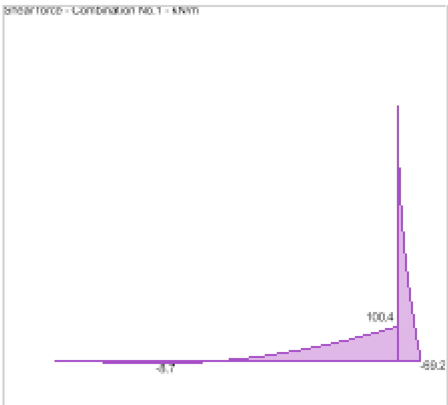
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **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**

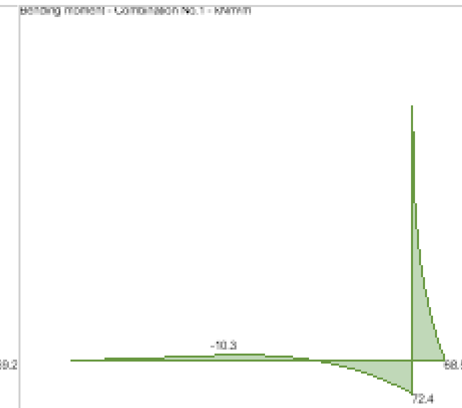
Reinforcement details

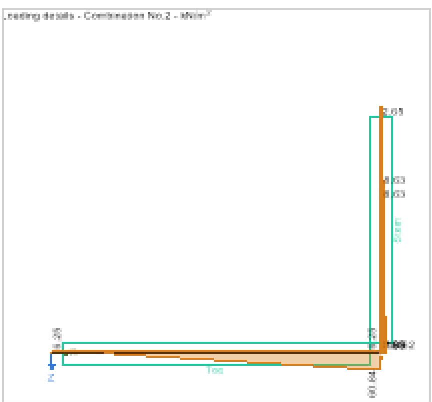
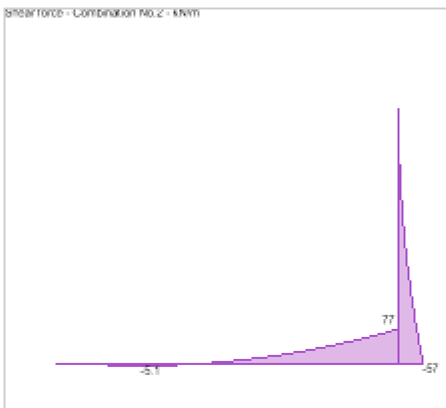
Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

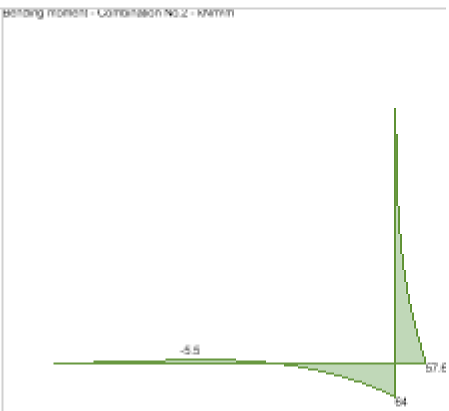
Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm





Check stem design at base of stem

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{68.5}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{242}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.039}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{230}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{30}$ mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{686}$ mm²/m

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

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

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{364}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h = \mathbf{12000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.682}$

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 — $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio — $\rho = A_{sr.req} / d = \mathbf{0.003}$

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{0.4}$

Reinforcement factor - exp.7.17 — $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.466}$

Limiting span to depth ratio - exp.7.16.a — $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$

Actual span to depth ratio — $h_{stem} / d = \mathbf{12.4}$

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 — $M_{sls} = \mathbf{46.1}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{199.5}$ N/mm²

Load duration — Long term

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

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

$A_{c.eff} = \mathbf{89917}$ mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.011}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{413}$ mm

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

$w_k = \mathbf{0.247}$ mm

w_k / w_{max} = **0.824**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **69.2** kN/m

$C_{Rd,c}$ = 0.18 / γ_C = **0.120**

k = min(1 + √(200 mm / d), 2) = **1.909**

Longitudinal reinforcement ratio — ρ_l = min($A_{sr.prov}$ / d, 0.02) = **0.004**

v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.506** N/mm^2

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

$V_{Rd.c}$ = **128.5** kN/m

V / $V_{Rd.c}$ = **0.539**

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 × $A_{sr.prov}$, 0.001 × t_{stem}) = **300** mm^2/m

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

Transverse reinforcement provided — 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov}$ = π × ϕ_{sx}^2 / (4 × s_{sx}) = **393** mm^2/m

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

Check base design at toe

Depth of section — h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **72.4** kNm/m

Depth to tension reinforcement — d = h - c_{bb} - ϕ_{bb} / 2 = **165** mm

K = M / (d^2 × f_{ck}) = **0.089**

K' = (2 × η × α_{cc}/γ_C)×(1 - λ × (δ - K_1)/(2 × K_2))×(λ × (δ - K_1)/(2 × K_2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — z = min(0.5 + 0.5 × (1 - 2 × K / (η × α_{cc} / γ_C))$^{0.5}$, 0.95) × d = **151** mm

Depth of neutral axis — x = 2.5 × (d – z) = **35** mm

Area of tension reinforcement required — $A_{bb.req}$ = M / (f_{yd} × z) = **1103** mm^2/m

Tension reinforcement provided — 20 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{bb.prov}$ = π × ϕ_{bb}^2 / (4 × s_{bb}) = **2094** mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **249** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max}$ = 0.04 × h = **10000** mm^2/m

max($A_{bb.req}$, $A_{bb.min}$) / $A_{bb.prov}$ = **0.527**

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} = **52.6** kNm/m

Tensile stress in reinforcement — σ_s = M_{sls} / ($A_{bb.prov}$ × z) = **166.3** 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 × (h - d), (h - x) / 3, h / 2)

$A_{c.eff}$ = **71574** mm^2/m

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

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.029}$

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

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 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{371}$ mm

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

$w_k = \mathbf{0.222}$ mm

$w_k / w_{max} = \mathbf{0.74}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{100.4}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{2.000}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.013}$

$v_{min} = 0.035 \text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.542}$ N/mm^2

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

$V_{Rd.c} = \mathbf{133.2}$ kN/m

$V / V_{Rd.c} = \mathbf{0.754}$

PASS - Design shear resistance exceeds design shear force

Check base design at toe

Depth of section $h = \mathbf{250}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{10.3}$ kNm/m

Depth to tension reinforcement $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{195}$ mm

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

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{185}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{24}$ mm

Area of tension reinforcement required $A_{bt.req} = M / (f_{yd} \times z) = \mathbf{128}$ mm^2/m

Tension reinforcement provided 10 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{524}$ mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{294}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bt.max} = 0.04 \times h = \mathbf{10000}$ mm^2/m

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov} = \mathbf{0.561}$

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

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

Serviceability bending moment $M_{sls} = \mathbf{0}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bt.prov} \times z) = \mathbf{0}$ 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 × (h - d), (h - x) / 3, h / 2)

$A_{c.eff}$ = **75208** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.007**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **6.091**

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 — $s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **414** 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** mm

w_k / w_{max} = **0**

PASS - Maximum crack width is less than limiting crack width

Secondary transverse reinforcement to base - Section 9.3

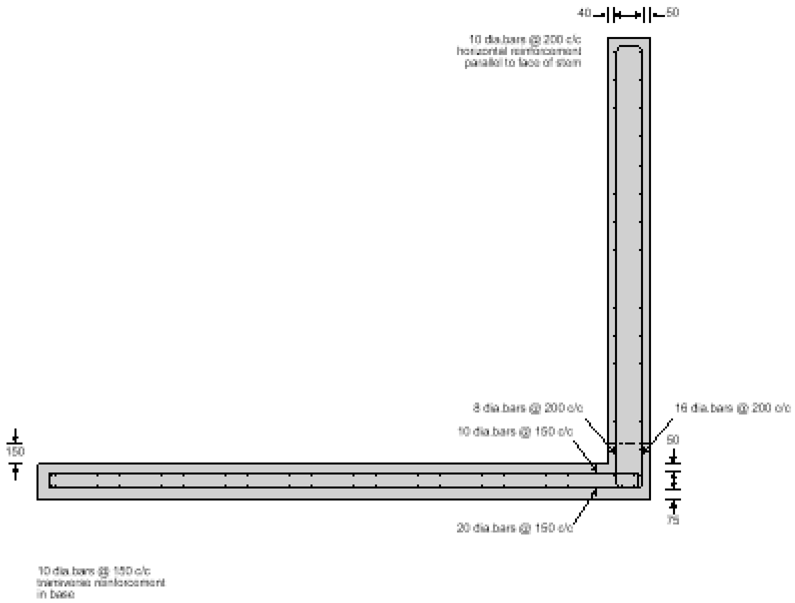
Minimum area of reinforcement – cl.9.3.1.1(2) — $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **419** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) — s_{bx_max} = **450** mm

Transverse reinforcement provided — 10 dia.bars @ 150 c/c

Area of transverse reinforcement provided — $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **524** mm²/m

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



Reinforcement details

CONTINUE THE CONVERSATION

TO DISCUSS YOUR PROJECT WITH US
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