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STRUCTURAL CALCULATION PACKAGE

41 SHELDON AVENUE
LONDON N6 4JP

REF: 23135-SYM-XX-XX-SC-S-0001 REV B





Revision History

Revision	Description	Date	By	Checked
A	First Issue	17.12.23	SB	VP
B	Second Issue	29.02.24	SB	VP



1.0 Introduction

- 1.1 Symmetry Limited have been engaged by Radcliffes to produce a Basement Impact Assessment in support of the planning application for the proposed works at 41 Sheldon Avenue, London N6, in the London Borough of Haringey.
- 1.2 The proposed scope consists of the demolition of an existing building three-storey detached house, and the construction of three-storey building plus one-storey basement replacing the existing. The construction of a swimming pool at the rear of the new building is also proposed.
- 1.3 Structural calculations have been prepared in support of the planning application, focusing on the temporary and permanent works relating to the basement construction.

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 The British Geological Survey (BGS) map indicates that the site is underlain by the Claygate Member, over the London Clay Formation.
- 3.2 A site investigation carried out by Geotechnical and Environmental Associates (GEA) has generally confirmed the expected ground condition, consisting of:
- Made Ground, comprising of brown locally organic sandy silty clay, extending to a maximum depth of 0.50m in the boreholes, and a depth greater than 1.40m in the trial pits.
 - Claygate Member, comprising of firm orange-brown mottled brown clay, with occasional pockets and partings of silt and sand, encountered at a depth between 2.50m to 3.50m.
 - London Clay, comprising of firm becoming stiff brown silty clay, encountered to the maximum depth investigated (20m).

4.0 Substructure design

- 4.1 The proposed substructure solution consists of bored piles, installed in various arrangement below the proposed building.
- 4.2 The contiguous pile wall proposed at the perimeter of the basement will provide resistance to the soil pressure, while the lining wall will provide resistance to water pressure.
- 4.3 The angle of shearing resistance $\phi_d = 24$ (refer to clause 6.1.1 of GEA report for soil parameter details) was used to design the waler and horizontal struts propping the contiguous piles wall capping beam in temporary condition. Being the capping beam propped prior to the basement excavation, the piled wall is unlikely to experience any significant movement. Hence, at rest pressure has been considered for design purposes, adopting the coefficient $K_0 = 0.593$.



Stratum	Bulk Density (kg/m ³)	Effective Cohesion (c' – kN/m ²)	Effective Friction Angle (ϕ' – degrees)
Made ground	1800	Zero	27
Claygate Member	1900	Zero	24
London Clay	1950	Zero	24

Table 1. Soil design parameters (source: GEA report J23243 Rev 2).

- 4.4 The preliminary piling arrangement has been determined based on the information provided by GEA within their report. The tables including ultimate skin friction and ultimate bearing capacity have been included in Table 1. The estimated pile resistance for 300mm, 450mm and 600mm diameter piles at various depths has been included in Table 2.

Stratum	Depths m [level m OD]	kN / m ²
Ultimate Skin Friction		
Basement Excavation	GL to 4.40	Ignore
London Clay	4.40 to 15.00	Increasing linearly from 30 to 82
Ultimate End Bearing		
London Clay	8.00 to 15.00	Increasing linearly from 860 to 1485

Table 2. Ultimate skin friction and end bearing capacity of bored piles (source: GEA report J23243 Rev 2).

Pile diameter mm	Toe Depth (m) [Level m OD]	Pile length (m)	R _{d,GEO} (kN)
300	8	3.6	75
	10	5.6	135
	15	10.6	215
450	8	3.6	110
	10	5.6	200
	15	10.6	310
600	8	3.6	235
	10	5.6	395
	15	10.6	585

Table 3. Estimated pile resistance for 300mm, 450mm and 600mm diameter piles at various depths (source: GEA report J23243 Rev 2).



- 4.5 Type B (integral) waterproofing has been assumed. As such, the lining wall has been designed as a 250mm RC wall, with a reinforcement limiting crack widths to 0.2mm.

5.0 Superstructure design

- 5.1 The superstructure will consist of a steel frame supporting timber floor and timber roof. The design of the superstructure is not discussed within this report.

6.0 Swimming pool design

- 6.1 The design of the swimming pool structure is not discussed within this report.



7.0 Loadings

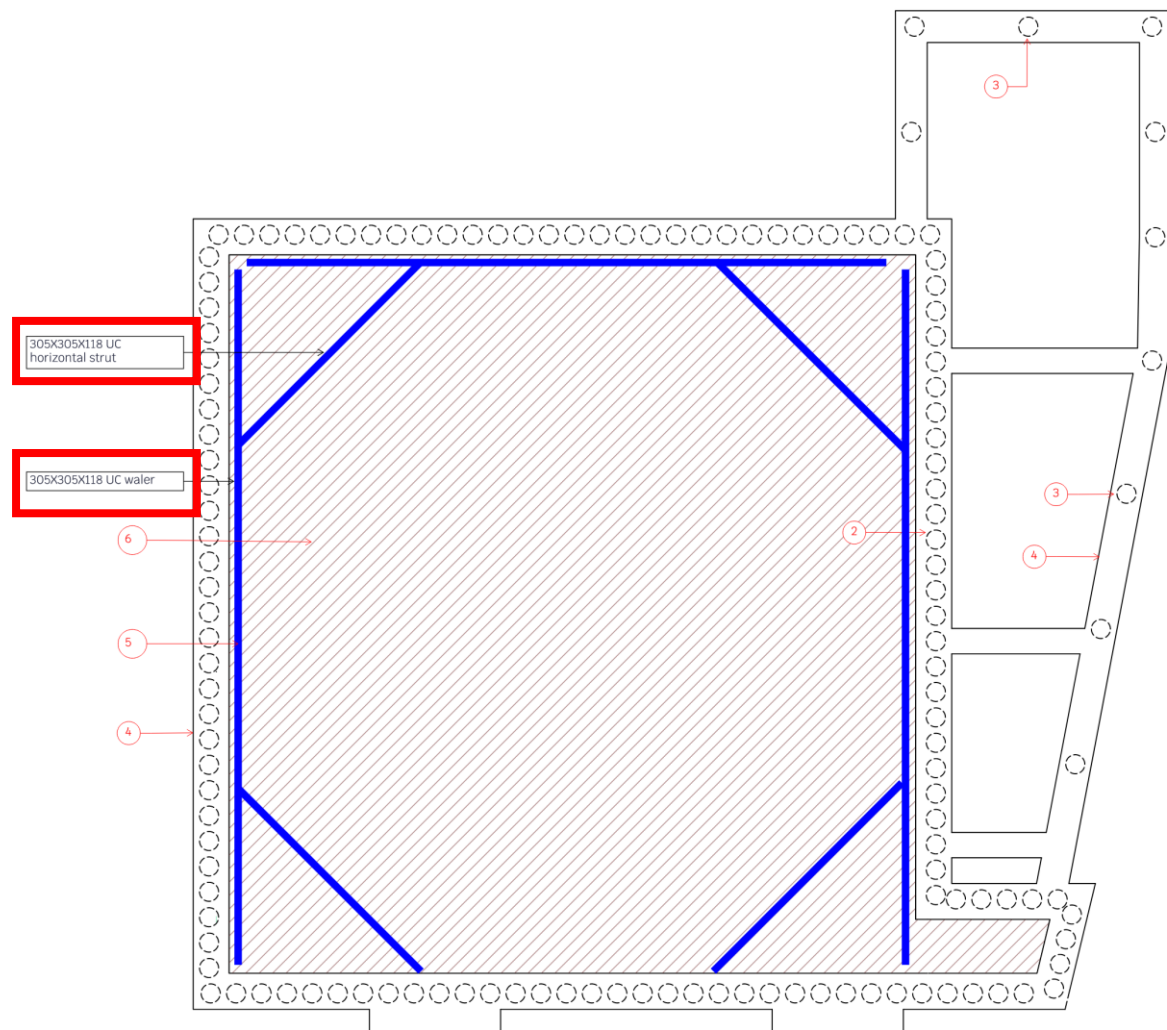
Item	DL (kPa U.N.O.)	LL (kPa)
Timber roof		
Ceiling and services	0.2	
Joists	0.2	
Plyboard and finishes	0.3	
Insulation	0.2	
Imposed load		0.6
Total dead load (flat roof)	0.9	
Total dead load (pitched roof, load on plan, 45 deg)	1.3	
Timber Floor (1st and 2nd floor)		
Ceiling and services	0.15	
Joists	0.2	
Plyboard and finishes	0.25	
Imposed load		1.5
Partition (self-weight assumed >1<2KN/m – EC1, 6.3.1.2 (8))		0.8
Total	0.6	2.3
Ground floor slab (above basement)		
250mm RC slab	6.3	
75mm screed	1.8	
Finishes	0.2	
Services	0.4	
Imposed load		1.5
Partition (self-weight assumed >1<2KN/m – EC1, 6.3.1.2 (8))		0.8
Total	8.7	2.3
Ground floor slab (side and rear wing)		
250mm RC slab	6.3	
75mm screed	1.8	
Finishes	0.2	
Imposed load		1.5
Partition (self-weight assumed >1<2KN/m – EC1, 6.3.1.2 (8))		0.8
Total	8.3	2.3
Basement slab		
300mm RC slab	7.5	
75mm screed	1.8	
Finishes	0.2	
Imposed load		1.5
Partition (self-weight assumed >1<2KN/m – EC1, 6.3.1.2 (8))		0.8
Total	9.5	2.3
Cavity Wall (brick + block)		
102 brick	2.1	
100 block	1.5	
Finishes	0.3	
Total	3.9	



Item	DL (kPa U.N.O.)	LL (kPa)
Basement lining wall		
250RC wall	6.3	
Finishes	0.3	
Total	6.6	
Blue roof (rear pavilion)		
Ceiling and services	0.2	
Timber joists	0.2	
Plyboard and finishes	0.3	
Insulation	0.2	
Blue roof ballasted system	1.0	
Imposed load		0.6
Total dead load (flat roof)	1.9	
Mass concrete	24 KN/m3	
Reinforced concrete	25 KN/m3	



8.0 Basement construction – temporary works calculations



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Calcs for CONTIGUOUS PILED WALL (2D ANALYSIS)				Start page no./Revision 1	
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ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - Concrete (EC2 normal) - R 1000x450



Span	Length (m)	Section	Start Support	End Support
1	3.8	R 1000x450	Fixed	Roller Pin Z
R 1000x450: Area 4500 cm ² , Inertia Major 759375 cm ⁴ , Inertia Minor 3750000 cm ⁴ , Shear area parallel to Minor 3750 cm ² , Shear area parallel to Major = 3750 cm ²				
Concrete (EC2 normal): Density 2500 kg/m ³ , Youngs 34 kN/mm ² , Shear 14.2 kN/mm ² , Thermal 0.00001 °C ⁻¹				

Loading

Self weight included

Load combination factors

Load combination	Self Weight	Imposed	Permanent
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.50	1.35
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00
1.0G + 1.0ψ ₂ Q (Quasi)	1.00	0.30	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
RC wall	Permanent	VDL	GlobalX	11.3 kN/m to 0 kN/m

Results

Reactions

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-23.1	56.6	-14.5
2	5.8	0	0

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-17.1	41.9	-10.8
2	4.3	0	0

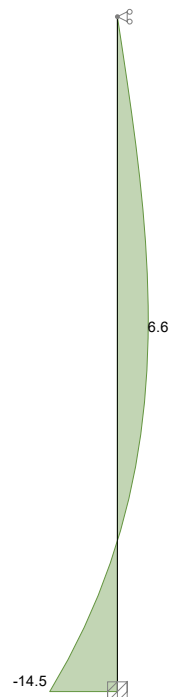
Load combination: 1.0G + 1.0 ψ_2 Q (Quasi)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-17.1	41.9	-10.8
2	4.3	0	0

Prop force at top of the wall
used to design the waler

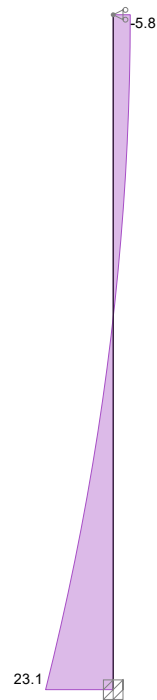
Forces

Strength combinations - Moment envelope (kNm)

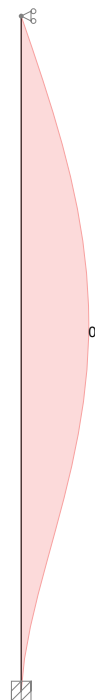


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Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)

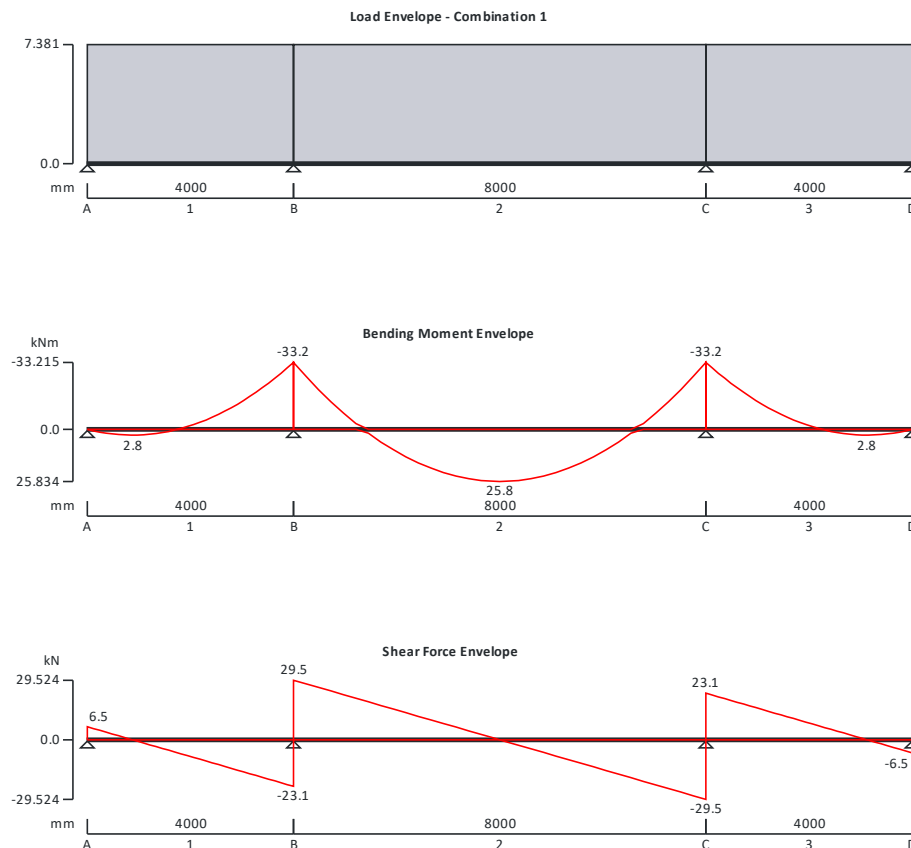


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Calcs for TEMPORARY WORKS - WALER BEAM				Start page no./Revision 1	
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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically restrained Rotationally free
Support D	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent self weight of beam $\times 1$ Permanent full UDL 4.3 kN/m
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Load combinations

Load combination 1	Support A	Permanent $\times 1.35$ Variable $\times 1.50$ Permanent $\times 1.35$
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Support B	Variable $\times 1.50$
	Permanent $\times 1.35$
	Variable $\times 1.50$
	Permanent $\times 1.35$
	Variable $\times 1.50$
	Permanent $\times 1.35$
Support C	Variable $\times 1.50$
	Permanent $\times 1.35$
	Variable $\times 1.50$
	Permanent $\times 1.35$
	Variable $\times 1.50$
	Permanent $\times 1.35$
Support D	Variable $\times 1.50$
	Permanent $\times 1.35$
	Variable $\times 1.50$

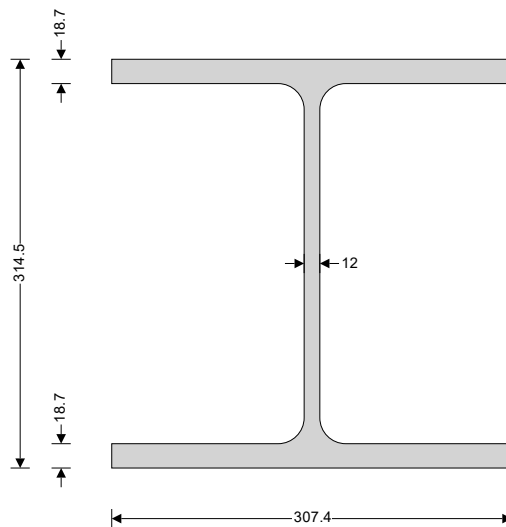
Analysis results

Maximum moment	$M_{\max} = 25.8 \text{ kNm}$	$M_{\min} = -33.2 \text{ kNm}$
Maximum moment span 1	$M_{s1_max} = 2.8 \text{ kNm}$	$M_{s1_min} = -33.2 \text{ kNm}$
Maximum moment span 2	$M_{s2_max} = 25.8 \text{ kNm}$	$M_{s2_min} = -33.2 \text{ kNm}$
Maximum moment span 3	$M_{s3_max} = 2.8 \text{ kNm}$	$M_{s3_min} = -33.2 \text{ kNm}$
Maximum shear	$V_{\max} = 29.5 \text{ kN}$	$V_{\min} = -29.5 \text{ kN}$
Maximum shear span 1	$V_{s1_max} = 6.5 \text{ kN}$	$V_{s1_min} = -23.1 \text{ kN}$
Maximum shear span 2	$V_{s2_max} = 29.5 \text{ kN}$	$V_{s2_min} = -29.5 \text{ kN}$
Maximum shear span 3	$V_{s3_max} = 23.1 \text{ kN}$	$V_{s3_min} = -6.5 \text{ kN}$
Deflection	$\delta_{\max} = 1.6 \text{ mm}$	$\delta_{\min} = 0.1 \text{ mm}$
Deflection span 1	$\delta_{s1_max} = 0 \text{ mm}$	$\delta_{s1_min} = 0.1 \text{ mm}$
Deflection span 2	$\delta_{s2_max} = 1.6 \text{ mm}$	$\delta_{s2_min} = 0 \text{ mm}$
Deflection span 3	$\delta_{s3_max} = 0 \text{ mm}$	$\delta_{s3_min} = 0.1 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 6.5 \text{ kN}$	$R_{A_min} = 6.5 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 4.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 52.6 \text{ kN}$	$R_{B_min} = 52.6 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 39 \text{ kN}$	
Maximum reaction at support C	$R_{C_max} = 52.6 \text{ kN}$	$R_{C_min} = 52.6 \text{ kN}$
Unfactored permanent load reaction at support C	$R_{C_Permanent} = 39 \text{ kN}$	
Maximum reaction at support D	$R_{D_max} = 6.5 \text{ kN}$	$R_{D_min} = 6.5 \text{ kN}$
Unfactored permanent load reaction at support D	$R_{D_Permanent} = 4.8 \text{ kN}$	

Section details

Section type	UC 305x305x118 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 18.7 \text{ mm}$
Nominal yield strength	$f_y = 265 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$

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Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only
Span 2 has full lateral restraint
Span 3 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LTA} = 1.000$
	$K_{LTB} = 1.000$
	$K_{LTC} = 1.000$
	$K_{LTD} = 1.000$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.94$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section	$c = d = 237.1 \text{ mm}$	
	$c / t_w = 21.0 \times \varepsilon \leq 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 = 127.7 \text{ mm}$	
	$c / t_f = 7.3 \times \varepsilon \leq 9 \times \varepsilon$	Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f = 277.1 \text{ mm}$
Shear area factor	$\eta = 1.000$
	$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 29.5 \text{ kN}$
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Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{4641 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{710.1 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{33.2 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{523.8 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.54}$$

$$C_1 = 1 / k_c^2 = \mathbf{3.429}$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = \mathbf{0.822}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{4000 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = \mathbf{8980.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{0.242}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

$$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling can be ignored}$$

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s2} / 500 = \mathbf{16 \text{ mm}}$$

Maximum deflection span 2

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{1.616 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

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STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.11

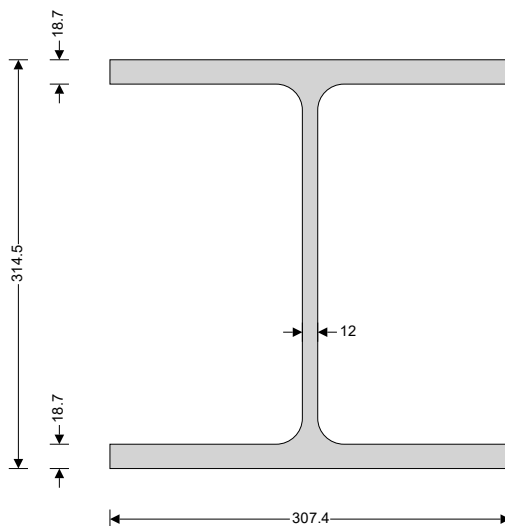
Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.1$

Design section 1

Section details

Section type	UC 305x305x118 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) = 18.7 \text{ mm}$
Nominal yield strength	$f_y = 345 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



UC 305x305x118 (British Steel Section Range 2022 (BS4-1))

Section depth, h , 314.5 mm
Section breadth, b , 307.4 mm
Mass of section, M , 119 kg/m
Flange thickness, t_f , 18.7 mm
Web thickness, t_w , 12 mm
Root radius, r , 20 mm
Area of section, A , 15165 mm²
Radius of gyration about y-axis, i_y , 135.703 mm
Radius of gyration about z-axis, i_z , 77.299 mm
Elastic section modulus about y-axis, $W_{el,y}$, 1775992 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 589553 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 1976765 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 897093 mm³
Second moment of area about y-axis, I_y , 279274688 mm⁴
Second moment of area about z-axis, I_z , 90614317 mm⁴

Analysis results

Design bending moment - Major axis	$M_{y,Ed} = 5.4 \text{ kNm}$
Design axial tension force	$N_{Ed} = 54 \text{ kN}$

Restraint spacing

Major axis lateral restraint	$L_y = 6000 \text{ mm}$
Minor axis lateral restraint	$L_z = 6000 \text{ mm}$
Torsional restraint	$L_T = 6000 \text{ mm}$

Classification of cross sections - Section 5.5

$$\epsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.83$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section	$c = d = 237.1 \text{ mm}$
	$c / t_w = 19.8 = 23.9 \times \epsilon \leq 72 \times \epsilon \quad \text{Class 1}$

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Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 127.7 \text{ mm}$$

$$c / t_f = 6.8 = 8.3 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check tension - Section 6.2.3

Design tension force

$$N_{Ed} = 54 \text{ kN}$$

Reaction B of "Waler" / sin (45)

Design resistance of section - eq 6.6

$$N_{t,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 5232 \text{ kN}$$

$$N_{Ed} / N_{t,Rd} = 0.01$$

PASS - Design tension resistance exceeds design tension

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 5.4 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 682 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.008$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - User defined

$$k_c = 1$$

$$C_1 = 1 / k_c^2 = 1$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained effective length

$$L = 1.0 \times L_{z,s1} = 6000 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 1146.6 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.771$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.786$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.833$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.833$$

Design buckling resistance moment - eq 6.55

$$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 567.9 \text{ kNm}$$

$$M_{y,Ed} / M_{b,y,Rd} = 0.01$$

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

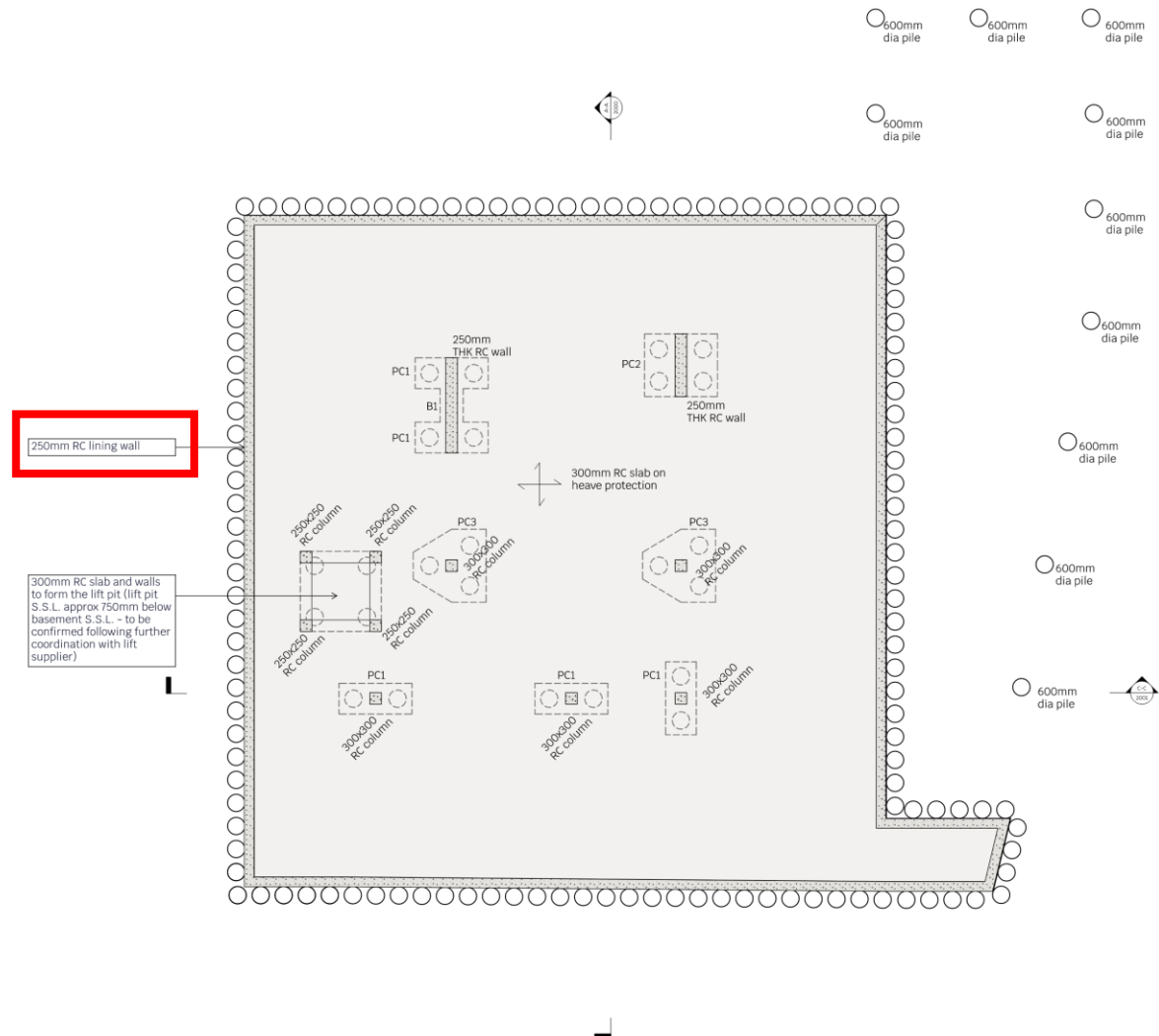
Bending and axial force check - eq.6.33 & eq.6.34

$$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) = 573.6 \text{ kN}$$

$$N_{Ed} / N_{y,lim} = 0.094$$

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

9.0 Basement construction - permanent works calculations

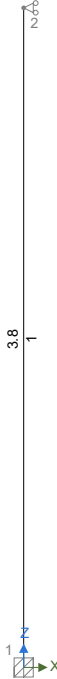


ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - Concrete (EC2 normal) - R 1000x250



Span	Length (m)	Section	Start Support	End Support
1	3.8	R 1000x250	Fixed	Roller Pin Z
R 1000x250: Area 2500 cm ² , Inertia Major 130208 cm ⁴ , Inertia Minor 2083333 cm ⁴ , Shear area parallel to Minor 2083 cm ² , Shear area parallel to Major = 2083 cm ²				
Concrete (EC2 normal): Density 2500 kg/m ³ , Youngs 34 kN/mm ² , Shear 14.2 kN/mm ² , Thermal 0.00001 °C ⁻¹				

Loading

Self weight included

Load combination factors

Load combination	Self Weight	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00
1.0G + 1.0ψ ₂ Q (Quasi)	1.00	0.30

Member Loads

Member	Load case	Load Type	Orientation	Description
RC wall	Imposed	VDL	GlobalX	37.3 kN/m to 0 kN/m

Results

Reactions

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-85	31.4	-53.7
2	21.3	0	0

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

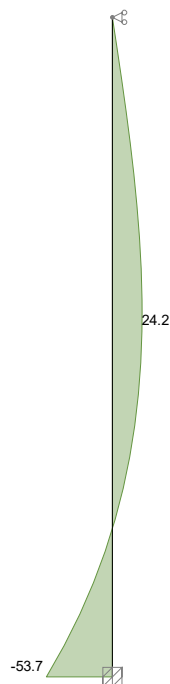
Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-56.7	23.3	-35.8
2	14.2	0	0

Load combination: 1.0G + 1.0 ψ_2 Q (Quasi)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	-17	23.3	-10.7
2	4.3	0	0

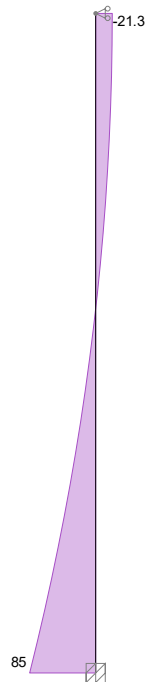
Forces

Strength combinations - Moment envelope (kNm)

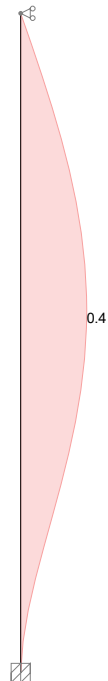


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Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



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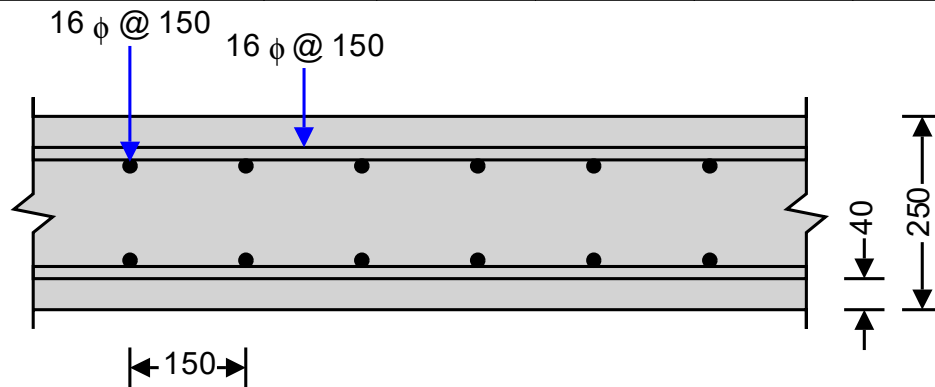
RC WALL DESIGN (EN 1992)

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.1.07

Design summary

Description	Unit	Allowable	Actual	Utilisation	Result
Moment capacity	kNm/m	106.22	53.95	0.51	PASS
Crack width	mm	0.30	0.13	0.44	PASS



Wall input details

Wall geometry

Thickness	$h = 250$ mm
Length	$b = 1000$ mm/m
Clear height between restraints	$l = 3800$ mm
Stability about minor axis	Braced

Concrete details

Concrete strength class	C32/40
Partial safety factor for concrete (2.4.2.4(1))	$\gamma_c = 1.50$
Coefficient α_{cc} (3.1.6(1))	$\alpha_{cc} = 0.85$
Maximum aggregate size	$d_g = 20$ mm

Reinforcement details

Reinforcement in outer layer	Horizontal
Nominal cover to outer layer	$c_{nom} = 40$ mm
Vertical bar diameter	$\phi_v = 16$ mm
Spacing of vertical reinforcement	$s_v = 150$ mm
Area of vertical reinforcement (per face)	$A_{sv} = 1340$ mm ² /m
Horizontal bar diameter	$\phi_h = 16$ mm
Spacing of horizontal reinforcement	$s_h = 150$ mm
Area of horizontal reinforcement (per face)	$A_{sh} = 1340$ mm ² /m
Characteristic yield strength	$f_{yk} = 500$ N/mm ²
Partial safety factor for reinf (2.4.2.4(1))	$\gamma_s = 1.15$
Modulus of elasticity of reinf (3.2.7(4))	$E_s = 200.0$ kN/mm ²

Fire resistance details

Fire resistance period	R = 60 min
Exposure to fire	Exposed on one side

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Ratio of fire design axial load to design resistance $\mu_{fi} = 0.70$

Axial load and bending moments from frame analysis

Design axial load $N_{Ed} = 31.4$ kN/m
Moment about minor axis at top $M_{top} = 0.0$ kNm/m
Moment about minor axis at bottom $M_{btm} = 53.7$ kNm/m

Wall effective length factor

Effective length factor for buckling about minor axis $f = 0.85$

Crack width details

Axial load due to quasi-permanent SLS. $N_{Ed_SLS} = 23.3$ kN/m
Moment at top due to quasi-permanent SLS. $M_{top_SLS} = 0.0$ kNm/m
Moment at btm due to quasi-permanent SLS. $M_{btm_SLS} = 31.4$ kNm/m
Duration of applied loading **Long term**
Maximum allowable crack width $w_{k_max} = 0.3$ mm

Calculated wall properties

Concrete properties

Area of concrete $A_c = h \times b = 250000$ mm²/m
Characteristic compression cylinder strength $f_{ck} = 32$ N/mm²
Design compressive strength (3.1.6(1)) $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_c = 18.1$ N/mm²
Mean value of cylinder strength (Table 3.1) $f_{cm} = f_{ck} + 8$ MPa = 40.0 N/mm²
Mean value of tensile strength $f_{ctm} = 3.02$ N/mm²
Secant modulus of elasticity (Table 3.1) $E_{cm} = 22000$ MPa $\times (f_{cm} / 10 \text{ MPa})^{0.3} = 33.3$ kN/mm²

Rectangular stress block factors

Depth factor (3.1.7(3)) $\lambda_{sb} = 0.8$
Stress factor (3.1.7(3)) $\eta = 1.0$

Strain limits

Compression strain limit (Table 3.1) $\epsilon_{cu3} = 0.00350$
Pure compression strain limit (Table 3.1) $\epsilon_{c3} = 0.00175$

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) $f_{yd} = f_{yk} / \gamma_s = 434.8$ MPa

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (4.4.1.2(3)) $C_{min,b} = \max(\phi_h, \phi_v - \phi_h) = 16$ mm
Min axis distance for fire (EN1992-1-2 T 5.4) $a_{fi} = 10$ mm
Allowance for deviations from min cover (4.4.1.3) $\Delta C_{dev} = 5$ mm
Min allowable nominal cover $C_{nom_min} = \max(a_{fi} - \phi_h / 2, C_{min,b} + \Delta C_{dev}) = 21$ mm
PASS - the nominal cover is greater than the minimum required

Effective depth of vertical bars

Effective depth $d = h - C_{nom} - \phi_h - \phi_v / 2 = 186$ mm
Depth to compression face bars $d' = C_{nom} + \phi_h + \phi_v / 2 = 64$ mm

Wall effective length

Wall effective length $l_0 = f \times l = 3230$ mm

Column slenderness

Radius of gyration about minor axis $i = h / \sqrt{12} = 7.2$ cm
Minor axis slenderness ratio (5.8.3.2(1)) $\lambda = l_0 / i = 44.8$

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Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections	$e_i = l_0 / 400 = 8.1 \text{ mm}$
Minimum end moment about minor axis	$M_{01} = \min(\text{abs}(M_{\text{top}}), \text{abs}(M_{\text{btm}})) + e_i \times N_{\text{Ed}} = 0.3 \text{ kNm/m}$
Maximum end moment about minor axis	$M_{02} = \max(\text{abs}(M_{\text{top}}), \text{abs}(M_{\text{btm}})) + e_i \times N_{\text{Ed}} = 54.0 \text{ kNm/m}$

Slenderness limit for buckling about minor axis (cl. 5.8.3.1)

Factor A	$A = 0.7$
Mechanical reinforcement ratio	$\omega = 2 \times A_{\text{sv}} \times f_{\text{yd}} / (A_c \times f_{\text{cd}}) = 0.257$
Factor B	$B = \sqrt{1 + 2 \times \omega} = 1.231$
Moment ratio	$r_m = M_{01} / M_{02} = 0.005$
Factor C	$C = 1.7 - r_m = 1.695$
Relative normal force	$n = N_{\text{Ed}} / (A_c \times f_{\text{cd}}) = 0.007$
Slenderness limit	$\lambda_{\text{lim}} = 20 \times A \times B \times C / \sqrt{n} = 350.9$
$\lambda < \lambda_{\text{lim}}$ - Second order effects may be ignored	

Design bending moment

Design moment about minor axis	$M_{\text{Ed}} = \max(M_{02}, N_{\text{Ed}} \times \max(h/30, 20 \text{ mm})) = 54.0 \text{ kNm/m}$
--------------------------------	---

Moment capacity about minor axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-	
Position of neutral axis	$z = 54.1 \text{ mm}$
Concrete compression force (3.1.7(3))	$F_c = \eta \times f_{\text{cd}} \times \min(\max(\lambda_{\text{sb}} \times z, 0 \text{ mm}), h) \times b = 785.0 \text{ kN/m}$
Moment of resistance	$M_{\text{Rdc}} = F_c \times [h / 2 - (\min(\lambda_{\text{sb}} \times z, h)) / 2] = 81.1 \text{ kNm/m}$

Moment of resistance of reinforcement

Strain in tension face bars	$\epsilon = \epsilon_{\text{cu3}} \times (1 - d / z) = -0.00853$
Stress in tension face bars	$\sigma = \text{if}(\epsilon < 0, \max(-1 \times f_{\text{yd}}, E_s \times \epsilon), \min(f_{\text{yd}}, E_s \times \epsilon)) = -434.8 \text{ N/mm}^2$
Force in tension face bars	$F_s = \text{if}(d > \lambda_{\text{sb}} \times z, A_{\text{sv}} \times \sigma, A_{\text{sv}} \times (\sigma - \eta \times f_{\text{cd}})) = -582.8 \text{ kN/m}$
Strain in compression face bars	$\epsilon' = \epsilon_{\text{cu3}} \times (1 - d' / z) = -0.00064$
Stress in compression face bars	$\sigma' = \text{if}(\epsilon' < 0, \max(-1 \times f_{\text{yd}}, E_s \times \epsilon'), \min(f_{\text{yd}}, E_s \times \epsilon')) = -127.9 \text{ N/mm}^2$
Force in compression face bars	$F_s' = \text{if}(d' > \lambda_{\text{sb}} \times z, A_{\text{sv}} \times \sigma', A_{\text{sv}} \times (\sigma' - \eta \times f_{\text{cd}})) = -171.5 \text{ kN/m}$
Resultant force on cross-section	$F_{\text{res}} = N_{\text{Ed}} - (F_c + F_s + F_s') = 0.729 \text{ kN/m}$
PASS - This is less than 1 kN therefore say OK	
Moment of resistance of tension face bars	$M_{\text{Rds}} = F_s \times (d - h / 2) = -35.6 \text{ kNm/m}$
Moment of resistance of compression face bars	$M_{\text{Rds}}' = F_s' \times (h / 2 - d') = -10.5 \text{ kNm/m}$

Combined moment of resistance

Moment of resistance about minor axis	$M_{\text{Rd}} = M_{\text{Rdc}} + M_{\text{Rds}}' - M_{\text{Rds}} = 106.2 \text{ kNm/m}$
PASS - The moment capacity exceeds the design bending moment	

Crack widths

Slenderness limit (cl. 5.8.3.1)

Min 1 st order moment about minor axis	$M_{01_SLS} = \min(\text{abs}(M_{\text{top_SLS}}), \text{abs}(M_{\text{btm_SLS}})) + e_i \times N_{\text{Ed_SLS}} = 0.2 \text{ kNm/m}$
Max 1 st order moment about minor axis	$M_{02_SLS} = \max(\text{abs}(M_{\text{top_SLS}}), \text{abs}(M_{\text{btm_SLS}})) + e_i \times N_{\text{Ed_SLS}} = 31.6 \text{ kNm/m}$
Moment ratio	$r_{m_SLS} = M_{01_SLS} / M_{02_SLS} = 0.006$
Factor C	$C_{\text{SLS}} = 1.7 - r_{m_SLS} = 1.694$
Relative normal force	$n_{\text{SLS}} = N_{\text{Ed_SLS}} / (A_c \times f_{\text{cd}}) = 0.005$

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Slenderness limit

$$\lambda_{lim_SLS} = 20 \times A \times B \times C_{SLS} / \sqrt{(n_{SLS})} = 407.1$$

$\lambda < \lambda_{lim_SLS}$ - Second order effects may be ignored

Design bending moment (cl. 7.3.4)

Design moment about minor axis

$$M_{Ed_SLS} = M_{02_SLS} = 31.6 \text{ kNm/m}$$

Cover to tension reinforcement

$$c = h - d - \phi_v / 2 = 56.0 \text{ mm}$$

Ratio of steel to concrete moduli

$$\alpha_e = E_s / E_{cm} = 6.0$$

Area of reinf in concrete units

$$A_{s,eff} = 2 \times \alpha_e \times A_{sv} = 16079 \text{ mm}^2/\text{m}$$

Combined area of steel/conc in conc units

$$A_{eff} = b \times h + A_{s,eff} = 266079 \text{ mm}^2/\text{m}$$

Reinforcement ratio per face

$$\rho = A_{sv} / (b \times d) = 0.007$$

Neutral axis depth with pure bending

$$x_b = d \times [-2 \times \alpha_e \times \rho + \sqrt{(4 \times \alpha_e^2 \times \rho^2 + 2 \times \alpha_e \times \rho \times (1 + d'/d))}] = 49.3 \text{ mm}$$

Second moment of area of cracked section

$$I_c = b \times x_b^3 / 3 + \alpha_e \times \rho \times b \times d \times [(x_b - d')^2 + (d - x_b)^2] = 191911106 \text{ mm}^4/\text{m}$$

Strain in tension face steel due to bending

$$\epsilon_{sb} = M_{Ed_SLS} \times (x_b - d) / (E_{cm} \times I_c) = -0.00067$$

Strain in comp face steel due to bending

$$\epsilon_{sb}' = M_{Ed_SLS} \times (x_b - d') / (E_{cm} \times I_c) = -0.00007$$

Strain due to axial load

$$\epsilon_{axial} = N_{Ed_SLS} / (A_{eff} \times E_{cm}) = 0.00000$$

Resultant strain in tension face steel

$$\epsilon_s = \epsilon_{sb} + \epsilon_{axial} = -0.00067$$

Resultant strain in comp face steel

$$\epsilon_s' = \epsilon_{sb}' + \epsilon_{axial} = -0.00007$$

Stress in tension steel

$$\sigma_s = \min(f_{yd}, \text{abs}(E_s \times \epsilon_s)) = 134.4 \text{ MPa}$$

Depth to neutral axis

$$x = [(\epsilon_s' \times d) - (\epsilon_s \times d')] / (\epsilon_s' - \epsilon_s) = 49.9 \text{ mm}$$

Effective depth of concrete in tension

$$h_{c,ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 66.7 \text{ mm}$$

Effective area of concrete in tension

$$A_{c,eff} = h_{c,ef} \times b = 66713 \text{ mm}^2/\text{m}$$

Load duration factor

$$k_t = 0.4$$

Reinforcement ratio

$$\rho_{p,eff} = A_{sv} / A_{c,eff} = 0.020$$

Mean value of conc tensile strength

$$f_{ct,eff} = f_{ctm} = 3.02 \text{ MPa}$$

Difference between reinf and concrete strains

$$\epsilon_{diff} = \max([\sigma_s - k_t \times f_{ct,eff} \times (1 + \alpha_e \times \rho_{p,eff}) / \rho_{p,eff}] / E_s, 0.6 \times \sigma_s / E_s) = 0.00040$$

Greater tensile strain

$$\epsilon_1 = \epsilon_s \times (h - x) / (d - x) = -0.00099$$

Lesser tensile strain

$$\epsilon_2 = \min(0, \epsilon_s' \times x / (x - d')) = 0.00000$$

Factor k₁

$$k_{1cs} = 0.8$$

Factor k₂

$$k_{2cs} = (\epsilon_1 + \epsilon_2) / (2 \times \epsilon_1) = 0.500$$

Factor k₃

$$k_{3cs} = 3.40 = 3.4$$

Factor k₄

$$k_{4cs} = 0.425$$

Maximum crack spacing

$$s_{r,max} = k_{3cs} \times c + k_{1cs} \times k_{2cs} \times k_{4cs} \times \phi_v / \rho_{p,eff} = 325.8 \text{ mm}$$

Crack width

$$w_k = s_{r,max} \times \epsilon_{diff} = 0.131 \text{ mm}$$

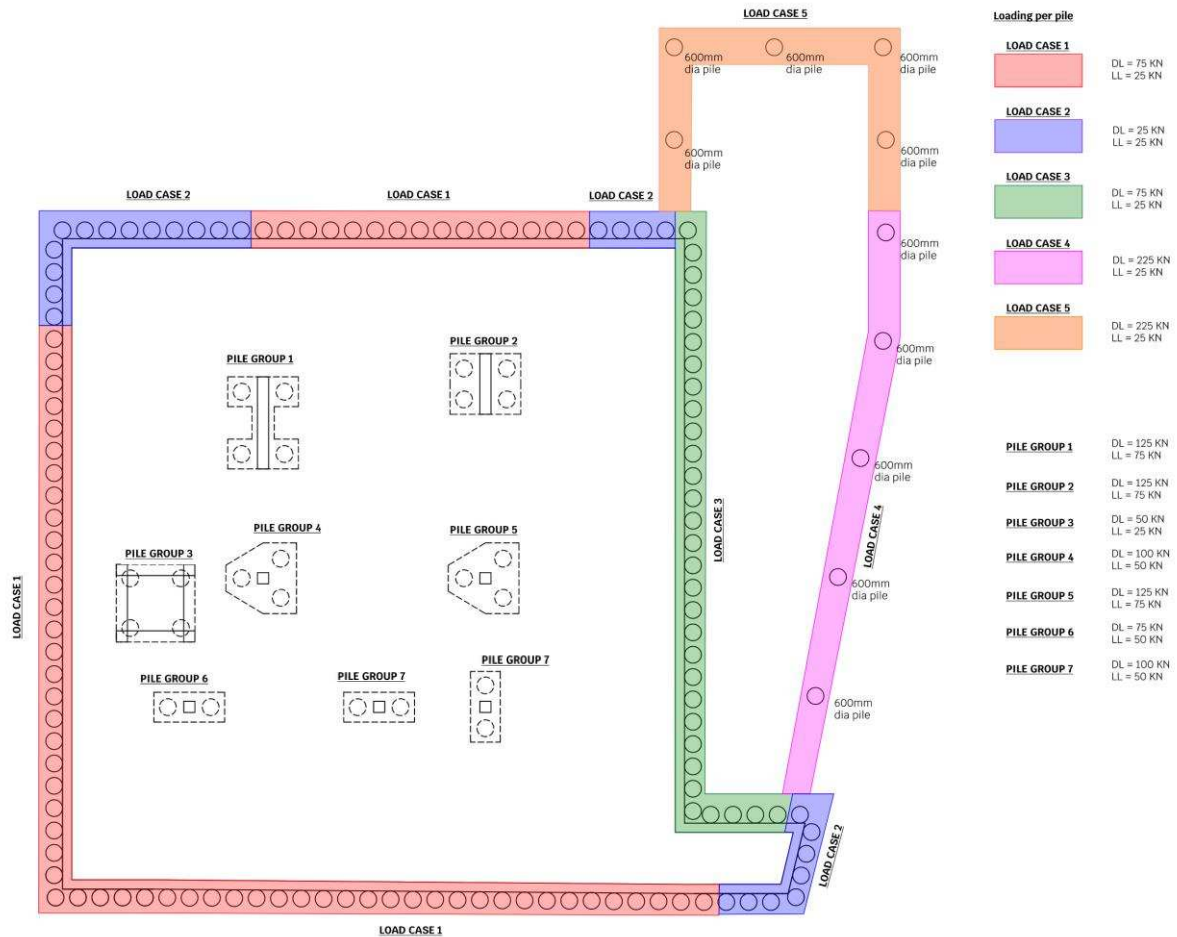
Allowable crack width

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

PASS - The maximum crack width is less than the maximum allowable



10.0 Piling load take down and analysis.





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Load case 1

Floor	Loadings per sq m		Span (m)	Total load per floor (KN/m)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	2.8	3.6	1.7
2F level	0.6	2.3	2.8	1.7	6.4
1F level	0.6	2.3	2.8	1.7	6.4
GF (above BL)	8.7	2.3	2.8	24.4	6.4
Basement level	9.5	2.3	2.8	26.6	6.4

Wall	Loadings per sq m		Height (m)	Total load (KN/m)	
	DL	LL		DL	LL
Cavity wall	3.9	0.0	6	23.4	0.0
RC lining wall	6.9	0.0	3.8	26.2	0.0

	DL	LL
Total load (KN/m)	107.6	27.4

	DL	LL
Total load per pile (KN)	64.4	16.4

	DL (KN)	LL (KN)
Total load per pile rounded up to the closest multiple of 25	75.0	25.0

Ultimate load per pile = 139KN >> Adopt 15m long, 450mm diameter pile



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Load case 2

Floor	Loadings per sq m		Span (m)	Total load per floor (KN/m)	
	DL	LL		DL	LL
Basement level	9.5	2.3	1.5	14.3	3.5

Wall	Loadings per sq m		Height (m)	Total load (KN/m)	
	DL	LL		DL	LL
RC lining wall	6.9	0.0	3.8	26.2	0.0

	DL	LL
Total load (KN/m)	40.5	3.5

	DL	LL
Total load per pile (KN)	24.2	2.1

	DL (KN)	LL (KN)
Total load per pile rounded up to the closest multiple of 25	25.0	25.0

Ultimate load per pile = 71 KN >> Adopt 15m long, 450mm diameter pile



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Load case 3

Floor	Loadings per sq m		Span (m)	Total load per floor (KN/m)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	4.6	6.0	2.8
2F level	0.6	2.3	4.6	2.8	10.6
1F level	0.6	2.3	4.6	2.8	10.6
GF (above BL)	8.7	2.3	2.3	20.0	5.3
GF	8.3	2.3	2.3	19.1	5.3
Basement level	9.5	2.3	2.3	21.9	5.3

Wall	Loadings per sq m		Height (m)	Total load (KN/m)	
	DL	LL		DL	LL
RC lining wall	6.9	0.0	3.8	26.2	0.0

	DL	LL
Total load (KN/m)	98.7	39.8

	DL	LL
Total load per pile (KN)	59.1	23.8

	DL (KN)	LL (KN)
Total load per pile rounded up to the closest multiple of 25	75.0	25.0

Ultimate load per pile = 139 KN >> Adopt 15m long, 450mm diameter pile



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Load case 4

Floor	Loadings per sq m		Span (m)	Total load per floor (KN/m)	
	DL	LL		DL	LL
Flat roof	0.9	0.6	1.9	1.7	1.1
GF	8.7	2.3	1.9	16.5	4.4

Wall	Loadings per sq m		Height (m)	Total load (KN/m)	
	DL	LL		DL	LL
Cavity wall	3.9	0.0	3	11.7	0.0

RC beam	Loadings per sq m		Span (m)	Total load (KN/m)	
	DL	LL		DL	LL
750x600	11.3	0.0	3	33.9	0.0

	DL	LL
Total load (KN/m)	63.8	5.5

	DL	LL
Total load per pile (KN) [piles at approx 3.2m centres]	204.3	17.6

	DL (KN)	LL (KN)
Total load per pile rounded up to the closest multiple of 25	225.0	25.0

Ultimate load per pile = 341 KN >> Adopt 10m long, 600mm diameter pile



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Load case 5

Floor	Loadings per sq m		Span (m)	Total load per floor (KN/m)	
	DL	LL		DL	LL
Blue roof	1.9	0.6	2.5	4.8	1.5
GF	8.7	2.3	2.5	21.8	5.8

Wall	Loadings per sq m		Height (m)	Total load (KN/m)	
	DL	LL		DL	LL
Cavity wall	3.9	0.0	3	11.7	0.0

RC beam	Loadings per sq m		Span (m)	Total load (KN/m)	
	DL	LL		DL	LL
750x600	11.3	0.0	2.5	28.3	0.0

	DL	LL
Total load (KN/m)	66.5	7.3

	DL	LL
Total load per pile (KN) [piles at approx 3.2m centres]	212.6	23.2

	DL (KN)	LL (KN)
Total load per pile rounded up to the closest multiple of 25	225.0	25.0

Ultimate load per pile = 341 KN >> Adopt 10m long, 600mm diameter pile



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Pile group 1

Floor	Loadings per sq m		Area (sq m)	Total load per floor (KN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	23	29.9	13.8
2F level	0.6	2.3	23	13.8	52.9
1F level	0.6	2.3	23	13.8	52.9
GF (above BL)	8.7	2.3	23	200.1	52.9
Basement level	9.5	2.3	23	218.5	52.9

	DL	LL
Total load	476.1	225.4

	DL	LL
Total load per pile	119.0	56.4

	DL	LL
Total load per pile rounded up to the closest multiple of 25	125.0	75.0

Ultimate load per pile = 281KN >> Adopt 15m long, 450mm diameter pile



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Pile group 2

Floor	Loadings per sq m		Area (sq m)	Total load per floor (kN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	23	29.9	13.8
2F level	0.6	2.3	23	13.8	52.9
1F level	0.6	2.3	23	13.8	52.9
GF (above BL)	8.7	2.3	23	200.1	52.9
Basement level	9.5	2.3	23	218.5	52.9

	DL	LL
Total load	476.1	225.4

	DL	LL
Total load per pile	119.0	56.4

	DL	LL
Total load per pile rounded up to the closest multiple of 25	125.0	75.0

Ultimate load per pile = 281kN >> Adopt 15m long, 450mm diameter pile



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Pile group 3

Floor	Loadings per sq m		Area (sq m)	Total load per floor (KN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	8	10.4	4.8
2F level	0.6	2.3	8	4.8	18.4
1F level	0.6	2.3	8	4.8	18.4
GF (above BL)	8.7	2.3	8	69.6	18.4
Basement level	9.5	2.3	8	76.0	18.4

	DL	LL
Total load	165.6	78.4

	DL	LL
Total load per pile	41.4	19.6

	DL	LL
Total load per pile rounded up to the closest multiple of 25	50.0	25.0

Ultimate load per pile = 105 KN >> Adopt 15m long, 450mm diameter pile



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Pile group 4

Floor	Loadings per sq m		Area (sq m)	Total load per floor (KN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	12.3	16.0	7.4
2F level	0.6	2.3	12.3	7.4	28.3
1F level	0.6	2.3	12.3	7.4	28.3
GF (above BL)	8.7	2.3	12.3	107.0	28.3
Basement level	9.5	2.3	12.3	116.9	28.3

	DL	LL
Total load	254.6	120.5

	DL	LL
Total load per pile	84.9	40.2

	DL	LL
Total load per pile rounded up to the closest multiple of 25	100.0	50.0

Ultimate load per pile = 210 KN >> Adopt 15m long, 450mm diameter pile



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Pile group 5

Floor	Loadings per sq m		Area (sq m)	Total load per floor (kN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	21	27.3	12.6
2F level	0.6	2.3	21	12.6	48.3
1F level	0.6	2.3	21	12.6	48.3
GF (above BL)	8.7	2.3	21	182.7	48.3
Basement level	9.5	2.3	21	199.5	48.3

	DL	LL
Total load	434.7	205.8

	DL	LL
Total load per pile	108.7	51.5

	DL	LL
Total load per pile rounded up to the closest multiple of 25	125.0	75.0

Ultimate load per pile = 281kN >> Adopt 15m long, 450mm diameter pile



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Pile group 6

Floor	Loadings per sq m		Area (sq m)	Total load per floor (KN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	12	15.6	7.2
2F level	0.6	2.3	12	7.2	27.6
1F level	0.6	2.3	12	7.2	27.6
GF (above BL)	8.7	2.3	12	104.4	27.6
Basement level	9.5	2.3	12	114.0	27.6

	DL	LL
Total load	248.4	117.6

	DL	LL
Total load per pile	62.1	29.4

	DL	LL
Total load per pile rounded up to the closest multiple of 25	75.0	50.0

Ultimate load per pile = 176 KN >> Adopt 15m long, 450mm diameter pile



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Pile group 7

Floor	Loadings per sq m		Area (sq m)	Total load per floor (KN)	
	DL	LL		DL	LL
Pitched roof	1.3	0.6	15	19.5	9.0
2F level	0.6	2.3	15	9.0	34.5
1F level	0.6	2.3	15	9.0	34.5
GF (above BL)	8.7	2.3	15	130.5	34.5
Basement level	9.5	2.3	15	142.5	34.5

	DL	LL
Total load	310.5	147.0

	DL	LL
Total load per pile	77.6	36.8

	DL	LL
Total load per pile rounded up to the closest multiple of 25	100.0	50.0

Ultimate load per pile = 210 KN >> Adopt 15m long, 450mm diameter pile