47 SWEETCROFT LANE UB10 9LE

By CAD Engineering Solutions

Project No

CES_AV_2025/177

Date

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Document

Basement Impact Assessment

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Contents

1.0 – Introduction

CAD Engineering Solutions has been appointed to prepare a Basement Impact Assessment (BIA) for the property located at 47 Sweet croft Lane UB10 9LE, which is undergoing the construction of a new basement and 3 storey structure. The existing house on the site will be demolished.

This report specifically evaluates the construction methodology of the basement and its potential impacts on the surrounding areas. This BIA serves as a supporting document for the planning application submitted to Hillingdon Council. It is important to note that this document is intended solely for planning and development purposes and should not be considered a design document.

Figure 1: Proposed Building – Front Elevation

Figure 2: Proposed new building rear elevation

Figure 3: Proposed new building – 3D

Figure 4: Proposed section of the building

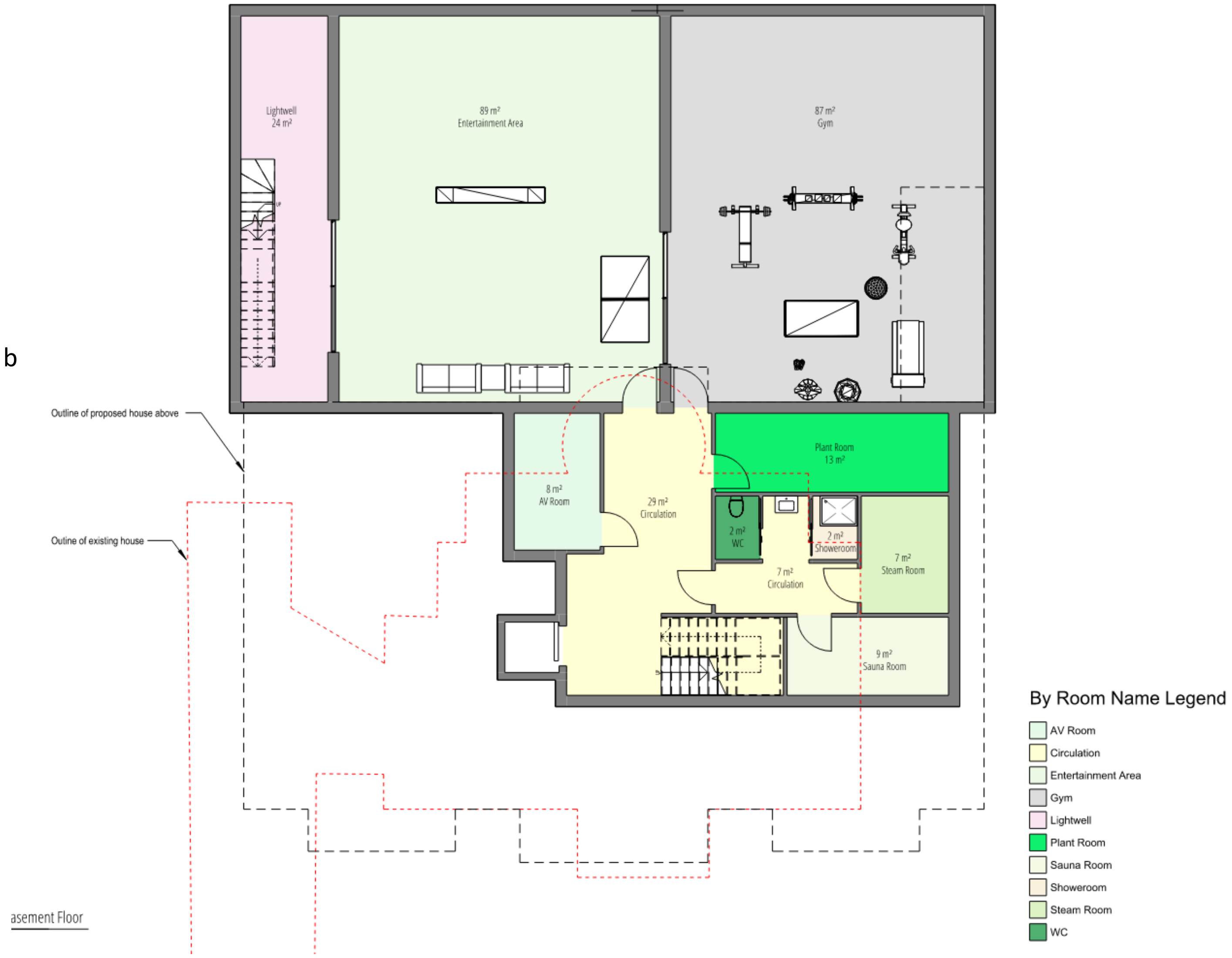


Figure 5: Proposed new basement – Architectural plan drawing.

1.1 – Site Description

This existing structure will be completely demolished to facilitate the construction of the new proposed development, which will consist of a basement and three additional storeys.

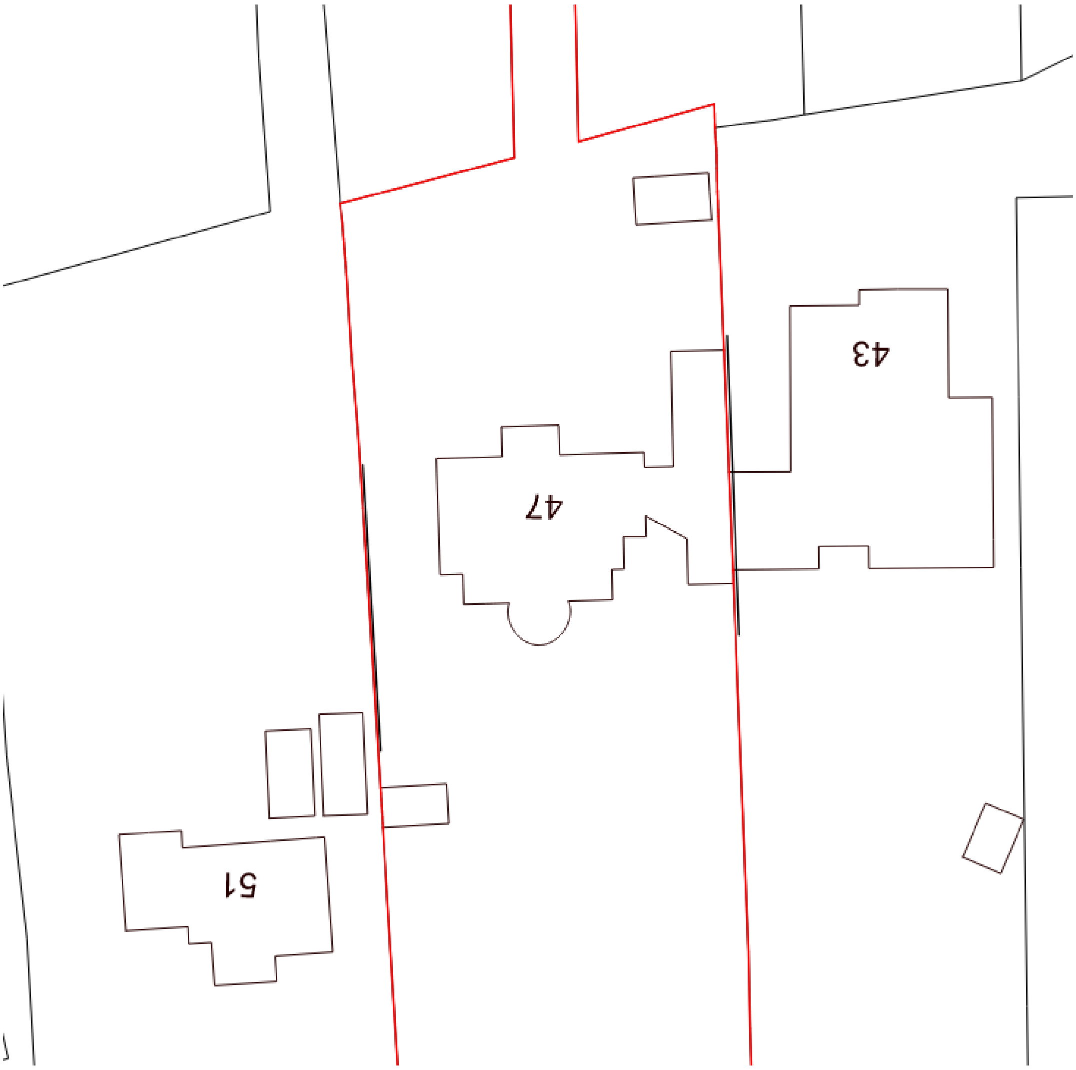


Figure 6: Existing Site Plan

Figure 7: Proposed Site Plan – I

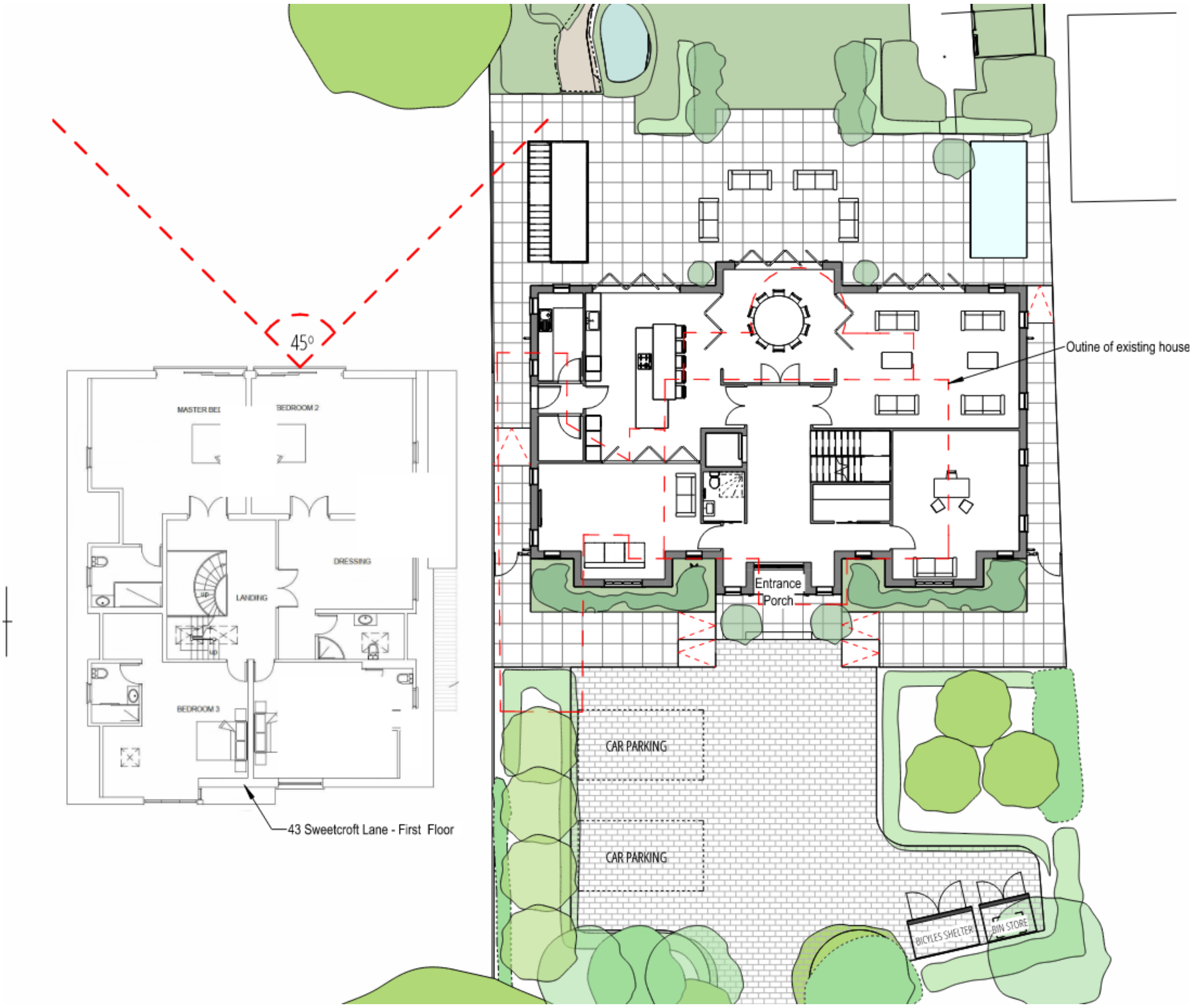


Figure 8: Proposed Site Plan – II

The proposed site is situated adjacent to the properties at 43 Sweetcroft lane and 51 Sweetcroft lane,

2.0 – Site Geological and Hydrogeological Conditions

2.1 – Site Geology

The site investigation report is currently unavailable. However, based on the British Geological Survey (BGS) maps, the area is primarily composed of London Clay and Harwich formations.

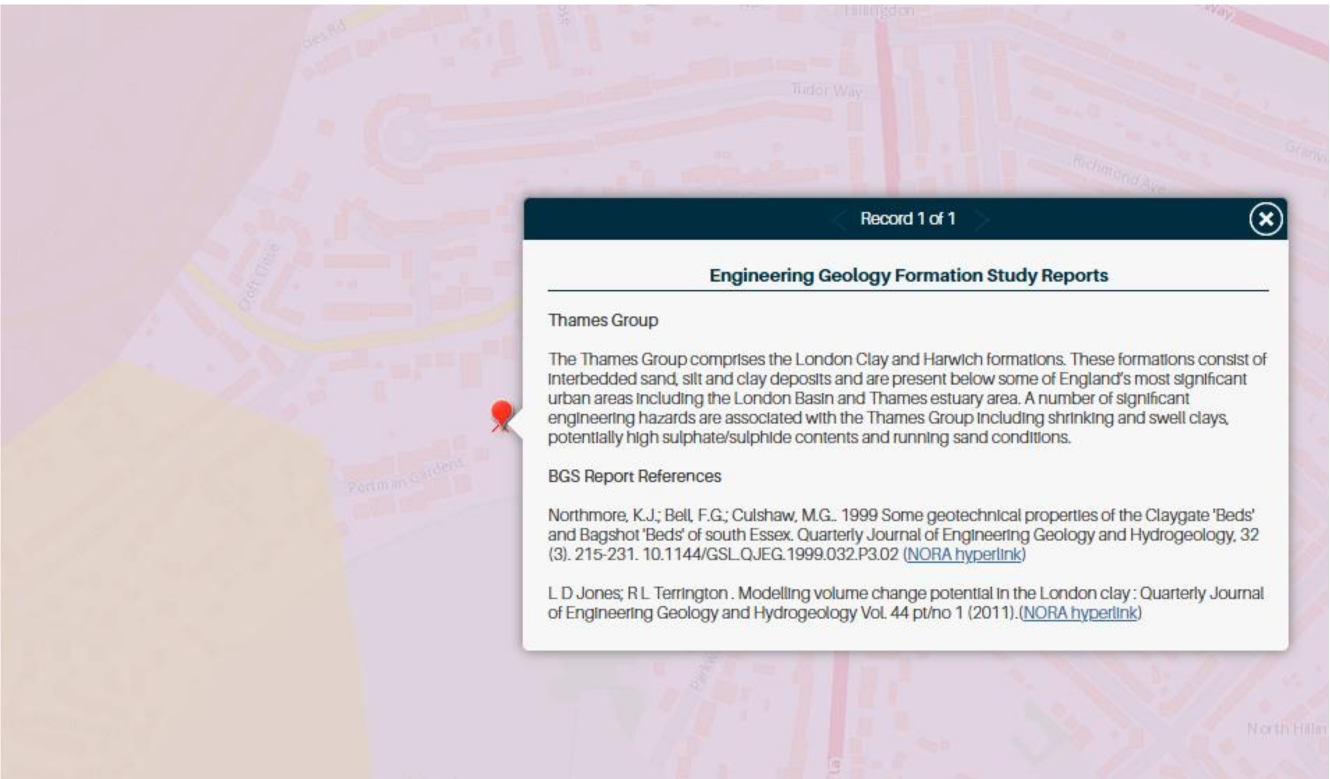


Figure 9: Image showing the geology details (Source – BGS maps)

Three boreholes have been identified surrounding the proposed site. Given the current scope of this report, the details from the British Geological Survey (BGS) and the nearby boreholes provide a reasonable representation of the expected soil strata at the proposed development area.

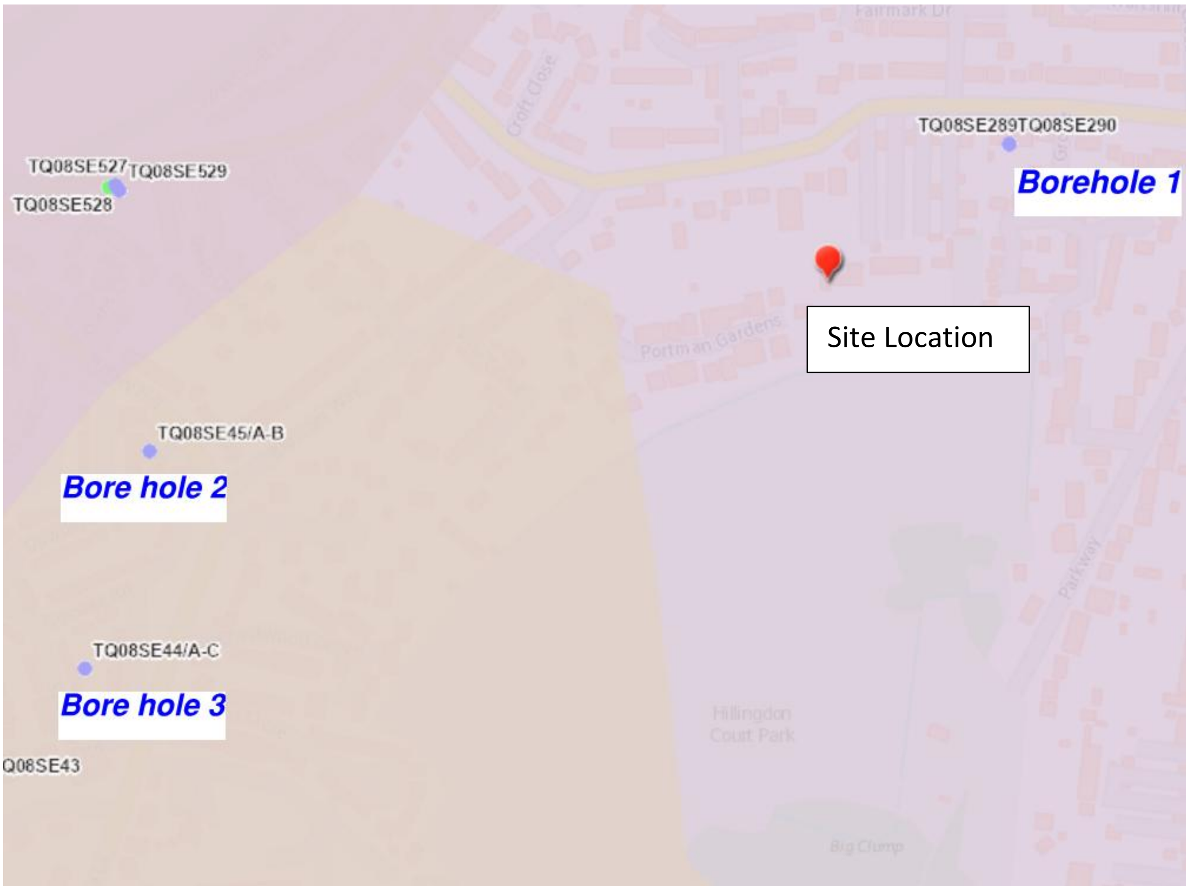


Figure 10: Image showing the surrounding bore holes (Source – BGS maps)

LOCATION : 21 Sweet Croft Lane, Hillingdon

BOREHOLE No. One

DATE OF BORING : 01.12.1988

Description of Strata	Strata Change: Legend	Strata Change: Depth M	Strata Change: O.D. Level M	Samples: Depth M	Samples: Type	SPT CPT N-Value	Water Level M
MADE GROUND Concrete over medium, brown orange CLAY with red brick, black coal ash, rootlets and flint fragments		1.00		1.00	J	(U100 blows) 5	
LONDON CLAY Stiff, light brown grey fissured CLAY with sand pockets and abundant selenite crystals.		2.00		2.00	U100	(35)	
				2.50	J		
- mottled yellow, slightly sandy		3.00		3.00	J	10	
- slightly silty, dark brown grey		4.00		4.00	U100	(25)	
				4.50	J		
- very stiff with dark grey sand pockets		5.00		5.00	J	20	
		6.00					
- claystone fragments				6.50	U100/J	(45)	
READING BEDS Very stiff, dark grey blue olive, sandy CLAY with abundant shell fragments.		7.00		7.00	J		
		8.00		8.00	B	34+	
Dense yellow, brown clayey SAND.		9.00					
				9.50	U100	(60)	
- yellow olive		10.00		10.00	B		

BOREHOLE DIAMETER : 150mm

LINING TUBES : 150mm to 1.50m

GROUND LEVEL :

REMARKS : Borehole drilled from existing ground level

- ▽ - Water strike
- ▼ - Water (standing level)
- W - Water Sample
- B/J - Bulk/Jar Sample
- S.P.T. - Standard Penetration Test
- C.P.T. - Cone Penetration Test
- (U) - Undisturbed Sample (38mm & 100mm)

Date. January 1989 | **BOREHOLE LOG** | **Report No.** S.1016(i)

Figure 11: Image showing the details of bore hole - 1

Civil Engineering Laboratory
Building Research Establishment

Figure: A
BOREHOLE No: B1
Sheet 1 of 1

Client: Property Services Agency / DCES
Investigation No: FGE / 2388 (Part 1)
Location: RAF UXBRIDGE (Site B: Honey Hill)
Project: Disposal of Land

BOREHOLE LOG

TQ08SE/45a

Date: 10th & 11th January 1985
Ground Level: (m. AOD)
Logged By: D. G. F.
Scale: 1:50
Equipment: Light Cable Percussion Boring Rig
Diameter: 200mm

Description of Strata	Depth (m)	O.D. Level (m)	Sample Depth (m)	Type	Test	Remarks
CLAY, Black organic, becoming brown, sandy and gravelly, gravel content comprises flint & brick fragment frequent roots (MADE GROUND)	GL		0.2-0.7	B		Water added in small amounts to assist boring
CLAY, Brown, lightly fissured, silty, occasional orange brown sandy pockets, frequent roots and root traces - Soft to Firm	0.7		0.7-1.2 1.2	U D	36	
CLAY, Brown, lightly mottled grey, heavily fissured, silty with occasional very thin layers of buff silt, fairly frequent roots and root traces - Very Stiff	1.5		0.7-1.2 2.0-2.5 2.5 2.8	B U D B	 37	No water entries observed, borehole dry throughout
CLAYSTONE, Brown calcareous claystone with occasional sandy pockets	2.9		3.0	D		
CLAY, Brown, mottled and streaked grey, heavily fissured, silty, with occasional thin layers and nodules of calcareously cemented buff sand, contains occasional sulphate and root traces - Very Stiff	3.1		3.5-4.0 4.0	U D	31	
CLAY, Brown, heavily fissured, silty, occasional sulphate traces - Stiff to Very Stiff	4.8		5.0-5.5 5.5	U D	27	
CLAY, Grey brown, heavily fissured silty with frequent thin bands and pockets of buff silt, occasional small nodules of brown calcareous claystone, frequent sulphate traces - Stiff to Very Stiff (WEATHERED LONDON CLAY)	6.0		6.5-7.0 7.0	U D	34	
CLAY, Grey, heavily fissured, silty, some fissures lightly dusted with light grey brown silt, infrequent sulphate traces - Stiff (LONDON CLAY)	7.5		8.0-8.5 8.5 9.5-10.0 10.0	U D U D	32 40	
End of borehole	10.0					

Figure 12: Image showing the details of bore hole - 2

TQ 08 SE/464a

Scott Wilson Kirkpatrick & Partners

Trial Pit Log

Trial Pit No. 1

067 840

Sheet 1 of 1

Site: RAF Uxbridge Communication Tower	Job No. 79043/96
Client: PSA	Method of Excavation: JCB 3CX
Contractor: PSA Term Contractor	Date: 10.7.1984
Weather: Dry and sunny	Ground Level

Sample	Depth (m)	Red. Level (mOD)	Description
	0.6		Stiff dark brown silty CLAY with fine to coarse gravel, brick fragments, ashes etc. (Topsoil and Fill)
	2.6		Stiff, occasionally firm to stiff, mottled blue grey and brown fissured silty CLAY with occasional thin layers of orange brown fine to medium sand. (Weathered London Clay)
	3.9		Firm to stiff mottled blue grey and brown fissured silty CLAY. (Weathered London Clay)

Very slow seepage below 2.6m.
Pit sides stable.
Plan area of pit 0.7m x 3.0m.

Figure 13: Image showing the details of bore hole - 3

2.2 – Site Hydrogeology

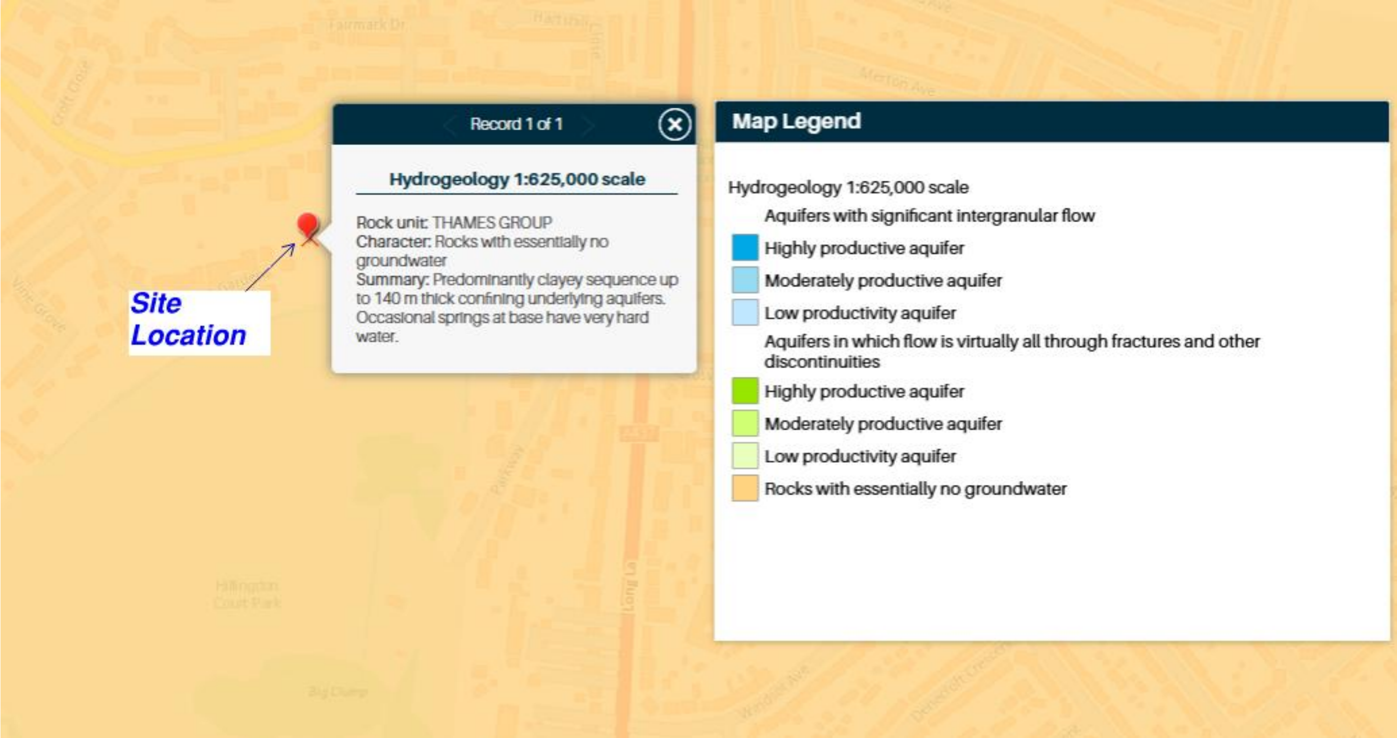


Figure 14: Image showing the site hydrogeology (Source – BGS Maps)

The British Geological Survey (BGS) maps indicate that the proposed site does not lie above an aquifer. However, a Additionally, groundwater levels identified in nearby boreholes are situated below the proposed basement height. Based on the above information site is less likely to encounter significant groundwater issues during construction, as the water table is at a lower elevation than the planned basement level.

Although no groundwater has been observed at basement height in nearby boreholes, we recommend conducting a detailed site investigation at the property to identify any potential groundwater issues.

The basement should be waterproofed, designed by a Chartered Surveyor in Structural Waterproofing (CSSW) to meet Grade 3 standards according to BS 8102:2009, suitable for habitable spaces.

For effective waterproofing, we advise considering combinations of Type A (Barrier Protection), Type B (Structurally Integral Protection), and Type C (Drained Protection) systems, as recommended by NHBC and Premier Guarantee. This approach will ensure robust protection against water ingress and enhance the longevity of the basement structure.

The Environment Agency's Flood Map for Planning (Rivers and Sea), shown below, indicates the site is in Flood Zone 1, and not at risk of flooding from rivers or the sea.

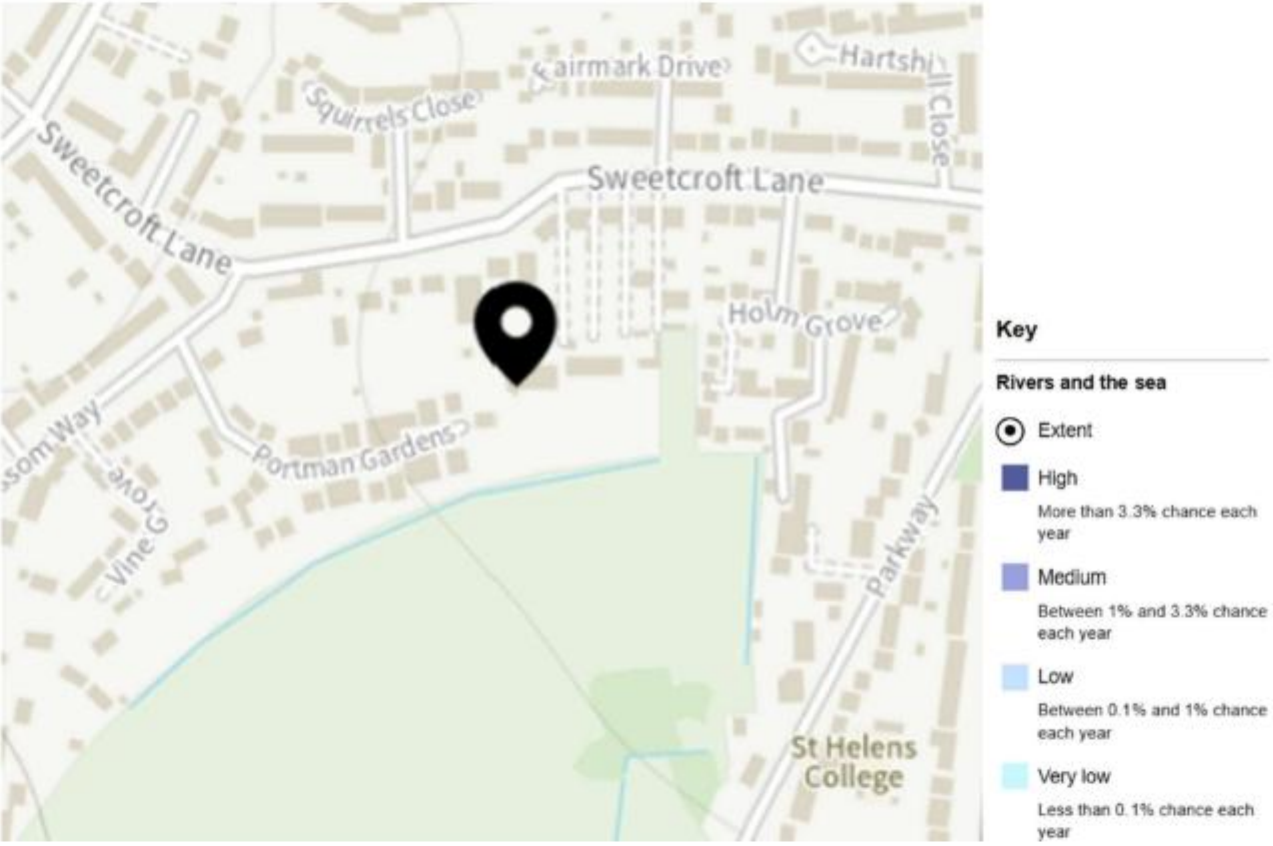


Figure 15: Image showing the site flood zone map

The proposed site is located in Flood Zone 1, which indicates a low probability of flooding, specifically less than a 0.1% annual chance of river or sea flooding. This classification suggests that the area is generally considered safe from significant flood risks. Since the development is less than 1 hectare and there is no critical drainage issues identified at the proposed site, a Flood Risk Assessment is not required.

While Flood Zone 1 has the lowest risk, it is important to note that flooding can still occur from other sources, such as surface water runoff or blocked drains. Therefore, developers should consider sustainable design and effective water management strategies to mitigate any potential risks associated with extreme weather events and climate change.

3.0 – Screening Stage

This stage identifies areas of concern that require further investigation, including a series of questions within a screening flowchart covering three categories: Geology and land stability, Hydrogeology, Surface flow and flooding.

3.1 – Geology and Land Stability

1. Does the existing site include slope, natural or man -made, greater than 7° (approximately 1 in 8)

No. The topography of the site is relatively level.

2. Will the proposed re-profiling of landscaping at site change slopes at the property boundary to greater than 7°?

No. There will be no re-profiling of the land within the site.

3. Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?

Currently, information about the neighbouring land is unknown. However, based on the surrounding topography, it is assumed that there are no significant slopes in the adjacent properties either.

4. Is the site within a wider hillside setting in which the general slope is greater than 7°?

The surrounding land does not have any significant sloped areas.

5. Is London Clay the shallowest stratum on the site?

The BGS maps shows that the shallowest strata are London Clay

6. Will any trees be felled as part of the proposed development and/ or are any works proposed within any tree protection zones where trees are to be retained?

No large trees will be felled as part of the development.

7. Is there a history of shrink / swell subsidence in the local area and/or evidence of such at the site?

No. There is no such evidence to the existing building or neighbouring properties.

8. Is the site within 100m of watercourse or a potential spring line?

No.

9. Is the site within an area of previously worked ground?

Unknown. Site Investigation Report to establish exact ground conditions for site.

10. Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering will be required during construction?

Site is not within an aquifer. Clay is present below the site.

11. Is the site within 5m of a public highway or pedestrian right of way?

The building is not located within 5 m of any public highway or pedestrian pathway

12. Will the proposed basement significantly increases the differential depth of foundations relative to neighbouring properties?

At two corners of the proposed basement, neighbouring buildings—specifically Nos. 51 and 43 Sweetcroft Lane—are located in close proximity. Assuming these properties do not have basements and their foundations are at shallow depth, we recommend constructing these corners of the basement in a sequence to eliminate any potential impact on the neighbouring foundations. Further details are provided in Section 5.0

13. Is the site over (or within the exclusion zone of) any tunnels?

No

3.2 – Hydrogeology

1.Is the site located directly above an aquifer?

Site is not within an aquifer. The site is underlain by clay, an unproductive stratum.

2. Will the proposed basement extend beneath the water table surface?

Unknown. According to the nearest boreholes, the water table is below the basement depth. It is recommended to conduct a site investigation to confirm.

3. Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas?

Unknown – Due to the reconstruction of the building, the impermeable areas may change.

4. As part of site drainage, will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via soakaways and/or SUDS)?

Unknown at this stage. There may be slight differences, but no significant variations are expected compared to the previous.

5. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond or spring line?

No

3.3 – Surface Flow and Flooding

1. Is the site within the catchment of the pond chains on Hampstead Heath?

No

2. As a part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off), be materially changed from the existing route?

Currently Unknown – Due to the construction of new building, the flow of water into the ground and the existing surface water drainage system may change. This needs to be confirmed by Drainage Strategy in further design stages.

3. Will the proposed basement result in a change in the proportion of hard surface water being received by adjacent properties or downstream watercourses?

Currently Unknown – Due to the new construction of the building, the impermeable areas may change.

4. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?

No. Collected surface water will be from building roofs and paving, as before. The quality of the water received downstream will therefore not change.

5. Is the site in an area known to be at risk from surface flooding or is it at risk from flooding because the proposed basement is below the static water level of a nearby surface water feature.

No. The site is in flood zone 1 and static water level is much below level of basement slab.

3.4 – Summary of Screening

The screening process identifies the following issues to be carried forward to scoping for further assessment:

- The two corners of the basement that are in close proximity to the neighbouring buildings should be cast in sequence to eliminate any potential impact on the adjacent foundations
- Change in water surface flow due to new construction.
- Change in the amount of impermeable surface due to new development

Other potential concerns identified in the Screening process were demonstrated to be not applicable or not significant when applied to the proposed development.

4.0 – Scoping Stage

The purpose of scoping is to assess in more detail the factors to be investigated in the impact assessment. Potential consequences are assessed for each of the identified potential impact factors.

4.1 – Geology and Land Stability

The construction of the basement will not impact the geology or stability of the ground, as the site is relatively level and the highways are located a safe distance from the proposed works. However, at two corners of the basement, the adjacent buildings at Nos. 43 and 51 Sweetcroft Lane are in close proximity. Therefore, these corners should be constructed in sequence as outlined in Section 5.0. Additionally, temporary supports must be installed as detailed in the Basement Construction Method Statement to ensure stability and prevent any issues during the construction process

4.2 – Hydrogeology

The exact water table at the proposed site is unknown and needs to be confirmed through a site investigation report. It is essential to design the basement to account for a full head of water or to establish the water table level based on the findings from the investigation.

4.3 – Surface Flow

Currently, the revised surface flow conditions are unknown. While it is anticipated that there will not be any significant difference in surface flow compared to the previous building, the construction of a new building and basement may alter surface flow patterns. This potential change needs to be confirmed through a Drainage Strategy in the further design stages to ensure effective management of surface water runoff and compliance with local drainage regulations.

5.0 – Basement Construction Method Statement

The proposed basement has a footprint of ~20.5 m in length and ~19 m in width, with a depth of ~3.2 m below the existing ground level. The basement walls will be set back ~1.2 m from the boundary with No. 43 and ~0.4 m from the boundary with No. 51. Two corners of the proposed basement are in close proximity to the existing adjacent foundations. To prevent any impact on these foundations, we recommend constructing the basement sequentially at these two corners, as outlined below.

Adequate temporary propping will be provided throughout the basement excavation to prevent any issues arising during construction that could affect the adjacent structures.

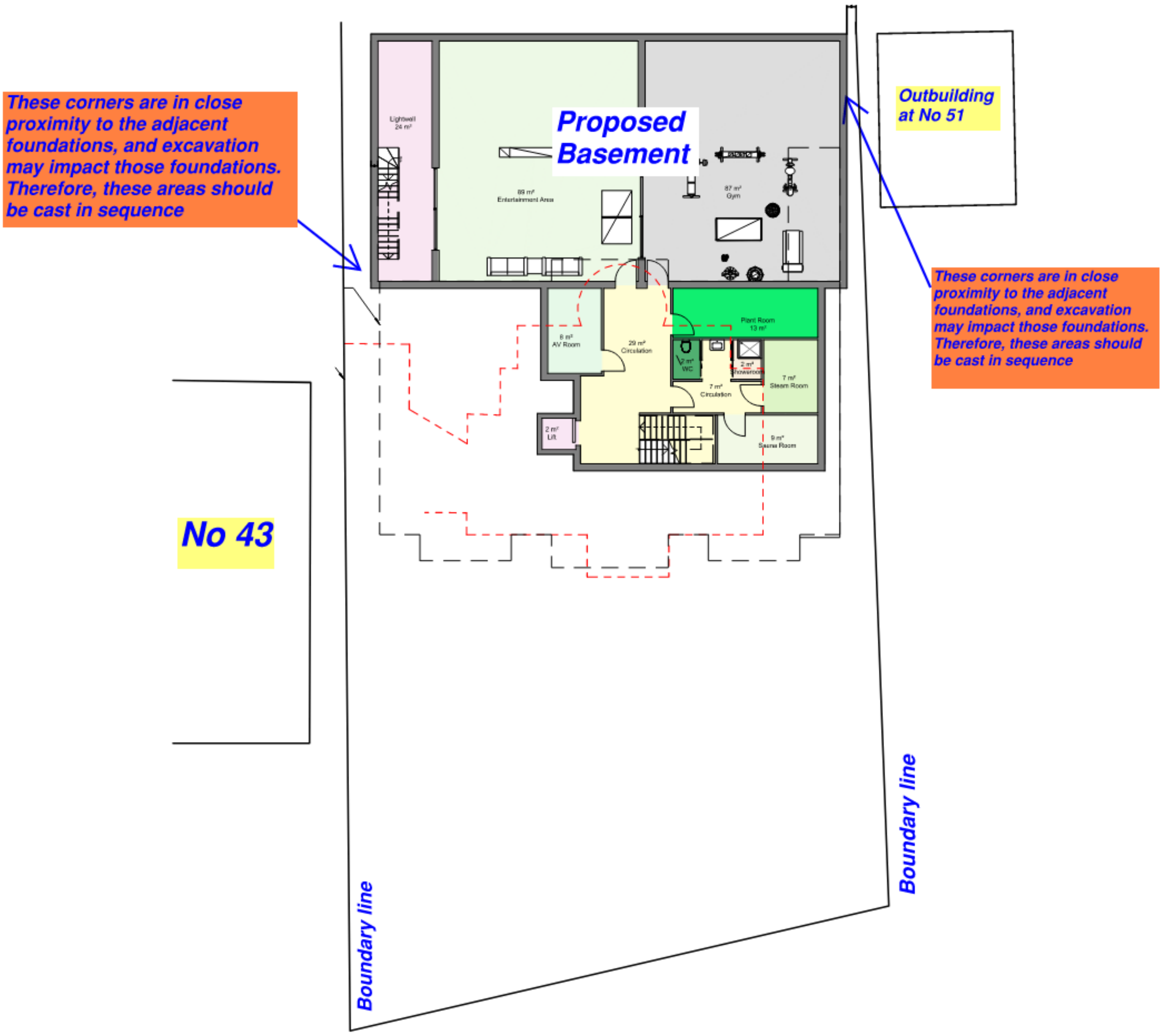


Figure 16: Image showing the proposed basement

5.1 – Sequence of works

The below works should be performed in the sequence shown in the temporary works drawings. The following steps should be repeated for each pair of sections in the sequence provided. The basement retaining walls will be 250 mm thick reinforced concrete (RC) walls. The base will consist of a 1500 mm toe and a 1000 mm heel, with a 300 mm thick RC base wall and a 200 mm thick basement slab. Refer to annexure for basement wall calculations.

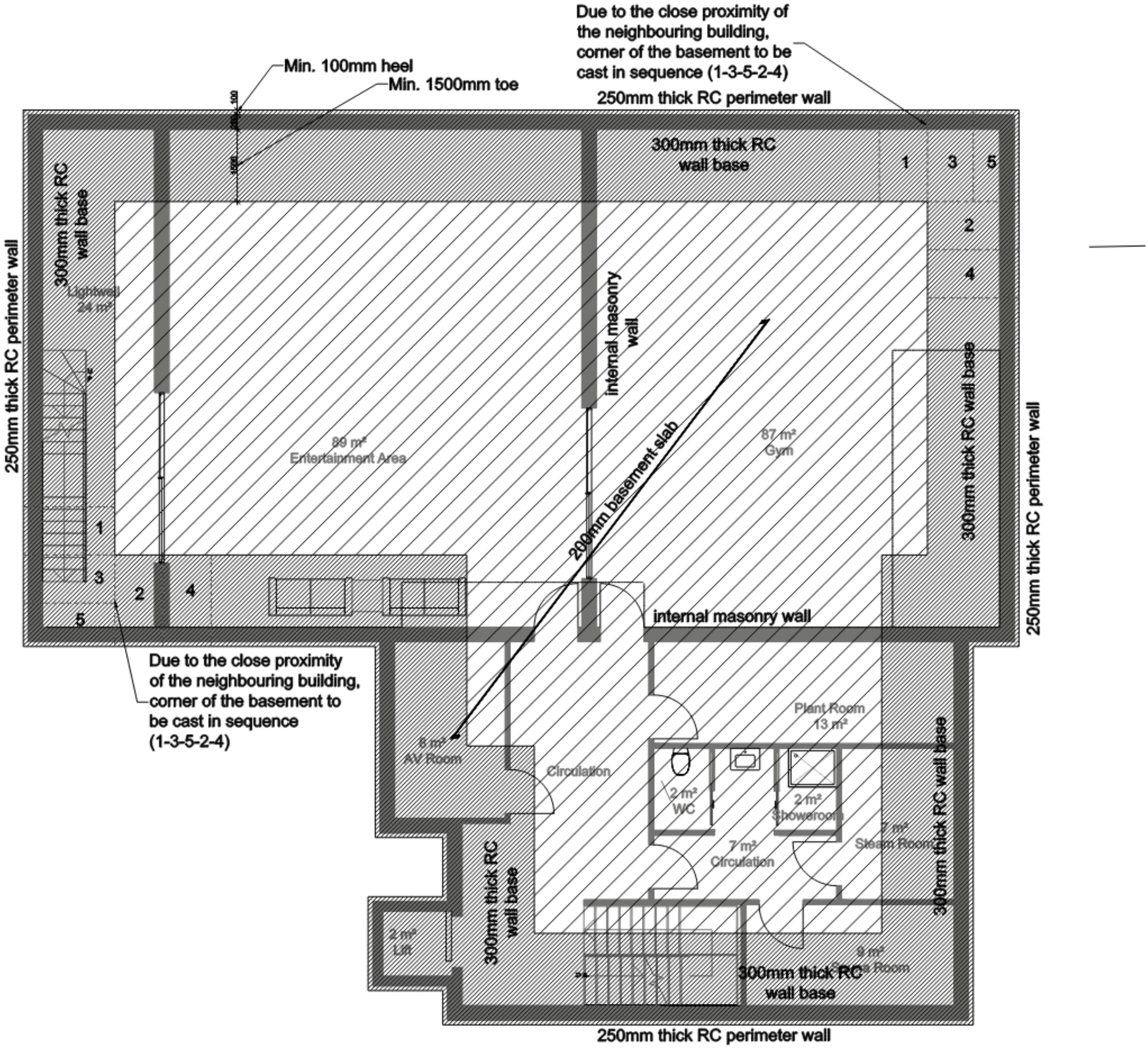


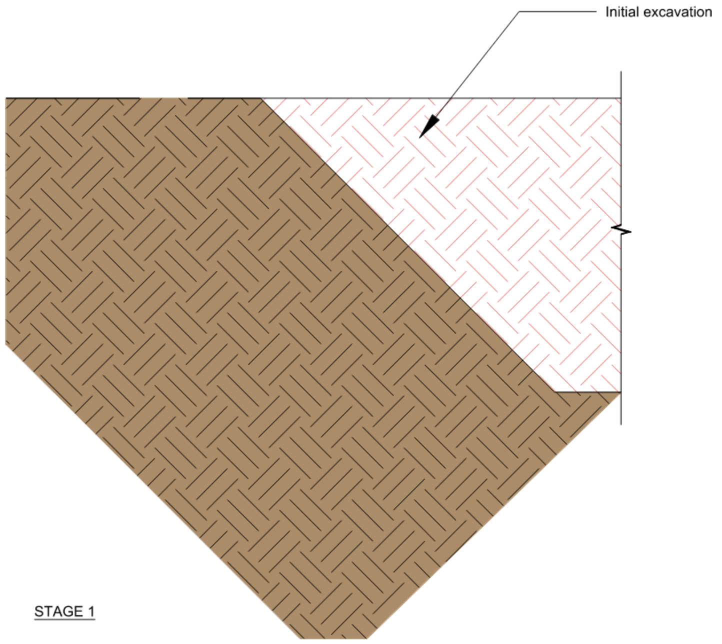
Figure 17: Basement Structure Plan

Stage 0

- Existing site conditions. Ground levels and wall thickness to be confirmed by contractor.
- The plywood hoarding will be fully secure with a lockable door for site access only.
- Spoil will be transported from the face of the excavation by wheelbarrow.

Stage 1

- Perform initial excavation for the proposed basement.



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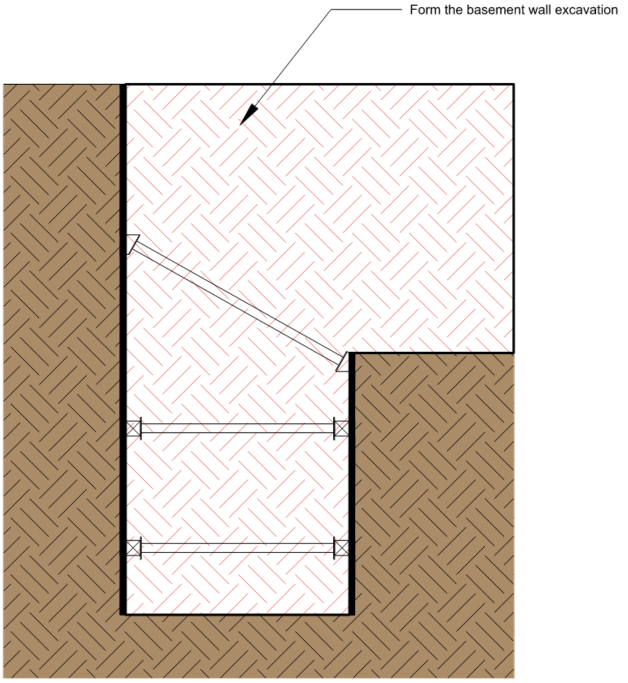
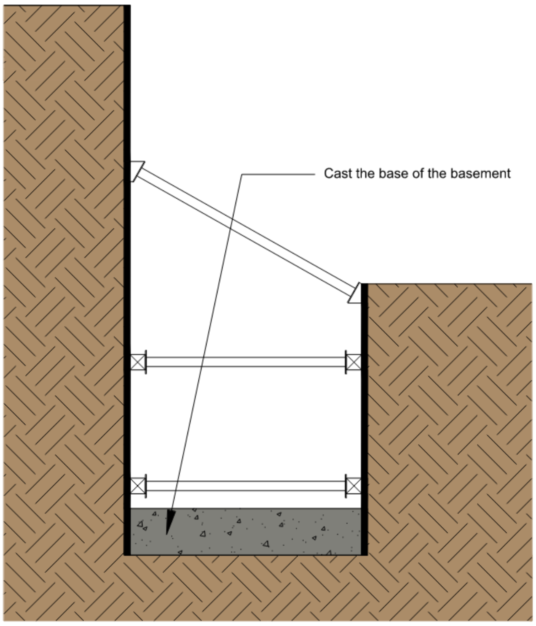
Stage 2

- Form excavations along the boundary of the basement wall and install temporary excavation supports



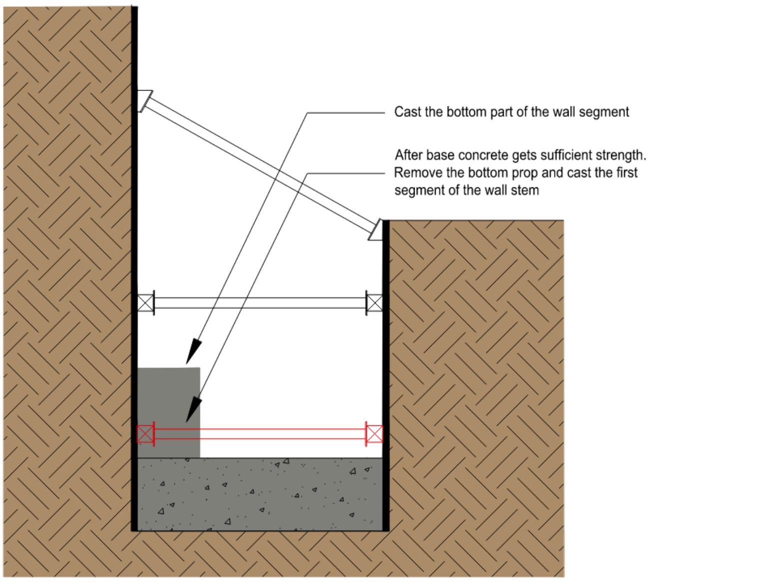
Stage 3

- Install the base reinforcement and L-shaped pull-out bars connecting particular parts of the foundation. Cast the base of the excavation.



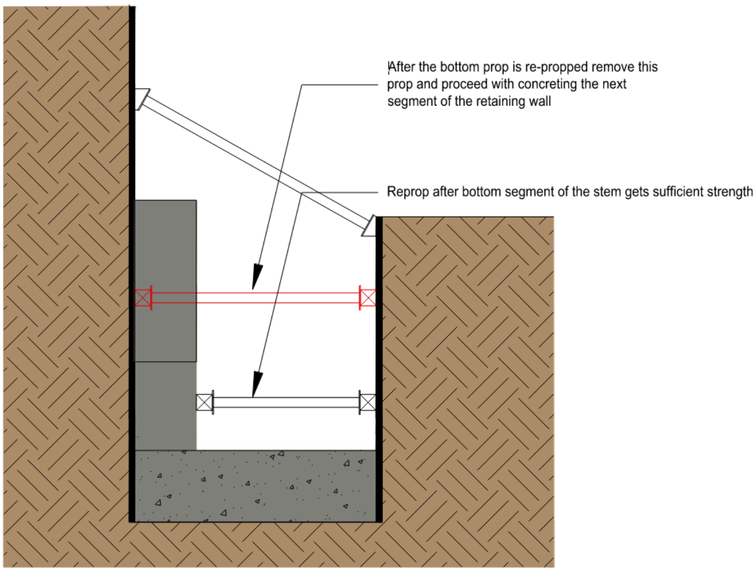
Stage 4

- Install the stem reinforcement and L-shaped pull-out bars to connect specific parts of the foundation. Remove the bottom prop once the base concrete has gained sufficient strength, then cast the first part of the stem.



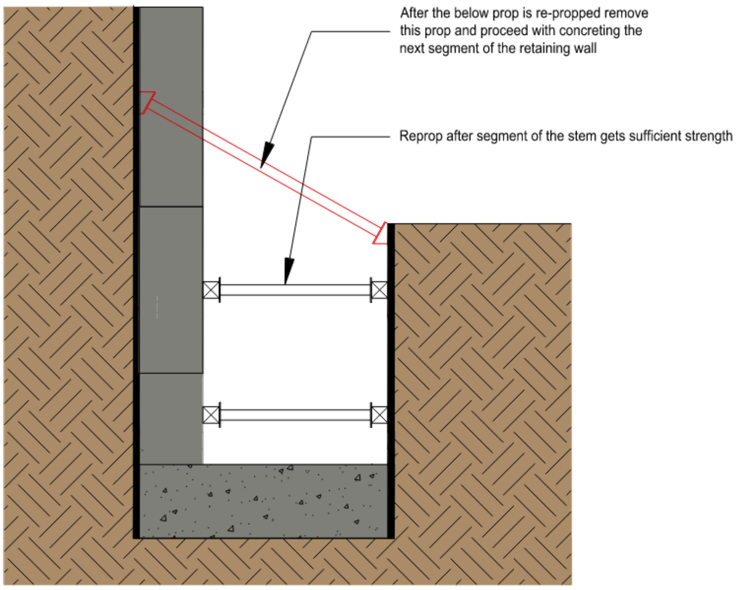
Stage 5

- Reinstate the prop once the bottom segment of the stem has attained sufficient strength. After reinstating the bottom prop, remove the subsequent prop and proceed with the concreting of the next segment of the retaining wall.



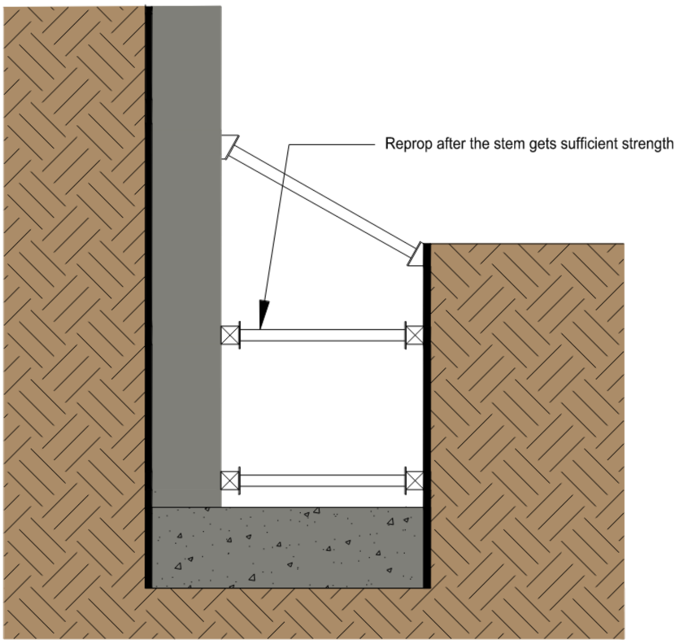
Stage 6

- Reinstate prop after the segment of the stem has gained sufficient strength. Once the prop is reinstated remove the next prop and proceed with concreting the next segment of the retaining wall



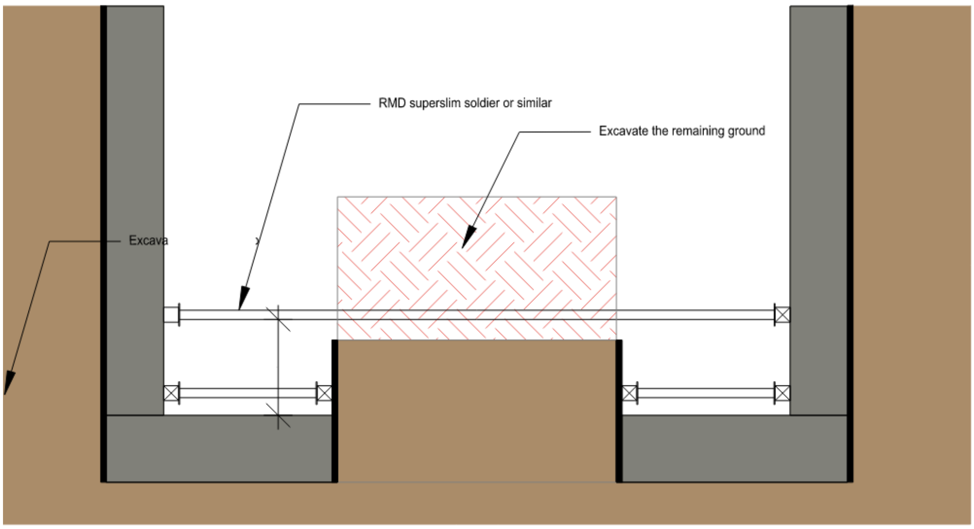
Stage 7

- Remove Shuttering after concrete gained sufficient strength



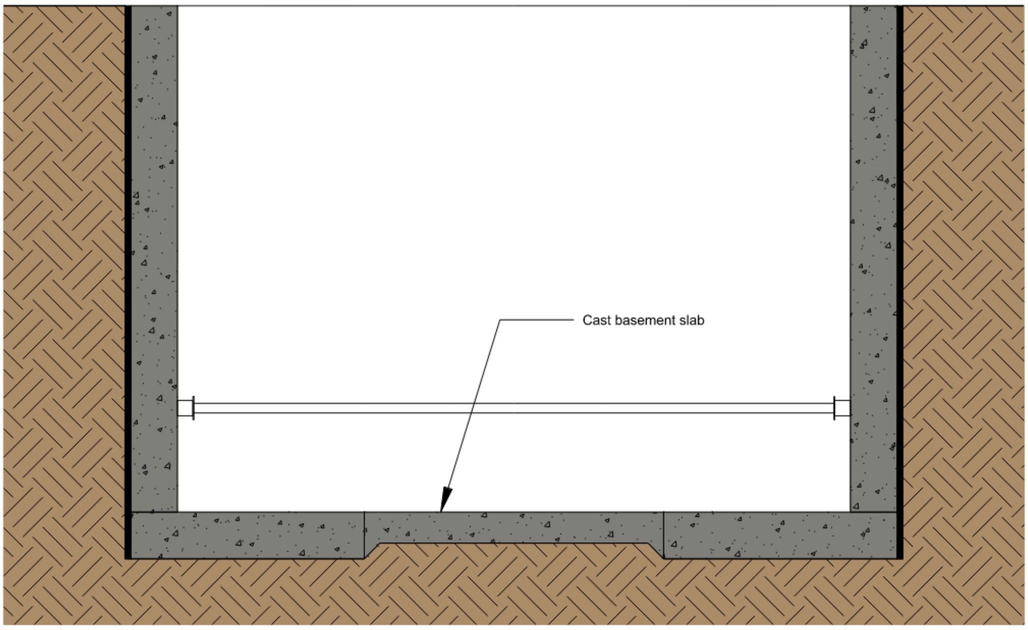
Stage 8

- Perform next part of the foundation on the opposite side of the basement following the same steps. Excavate the remaining ground. Install RMS superslim soldier or similar support.



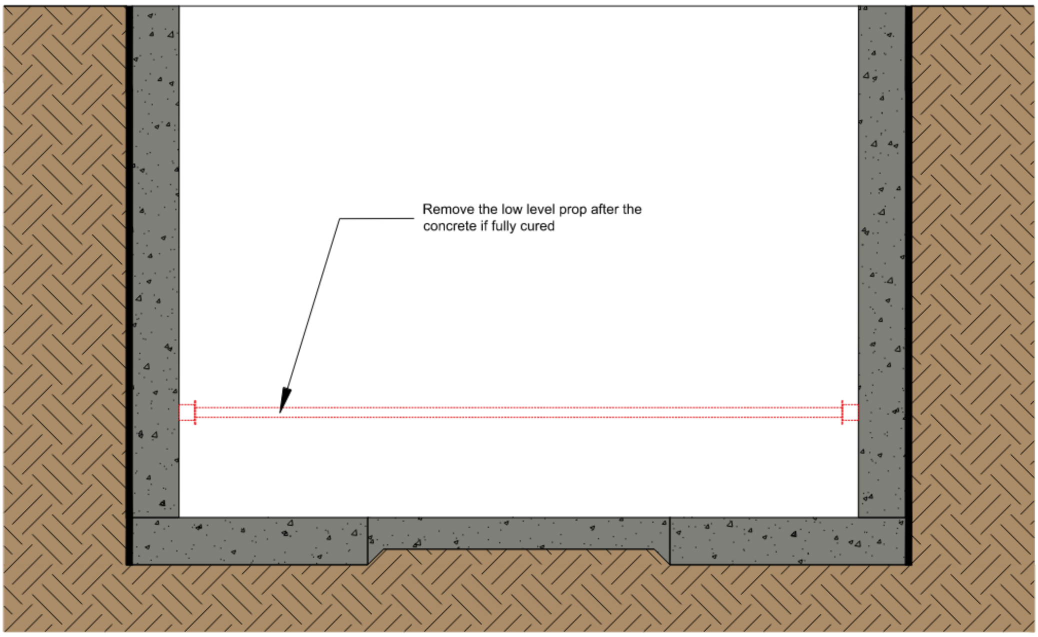
Stage 9

- Install the basement slab reinforcement and L-shaped pull-out bars connecting particular parts of the slab. Cast the basement slab.



Stage 10

- Remove all props after the concrete is fully cured



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6.0 –Basement Impact Assessment

6.1 – Impact on the adjacent building structures

Based on the initial assessment, two corners of the proposed basement, adjacent to No. 43 and the outbuilding at No. 51, are in close proximity to the basement footprint. Assuming these buildings do not have basements and their foundations are at shallow depth, we recommend conducting the basement excavation at these corners in sequence, as outlined above.

The basement walls have been designed to withstand the stresses generated by the adjacent foundations. Sequencing the excavation in these areas will help prevent any potential impact on the neighbouring foundations caused by the basement construction.

By carrying out the excavation in a controlled, sequential manner in the affected areas near the buildings in close proximity, we will ensure that no adverse effects occur on the adjacent building foundations during the construction of the new basement. Additionally, the use of temporary supports and propping during excavation will ensure the safety and integrity of the existing buildings throughout the basement construction process. Overall, this methodical sequencing will help safeguard the adjacent foundations from any adverse effects associated with the new basement works.

7.0 – Conclusion

The existing building will be fully demolished, and a new three-storey building with a basement will be constructed. The proposed basement is located well away from existing highways and pedestrian pavements; therefore, its construction is not expected to have any impact on public roads or pedestrian access.

However, at two corners of the basement, the proposed structure is in close proximity to the existing foundations of No. 43 and the outbuilding at No. 51. In these areas, the basement excavation will affect the neighbouring foundations. It is assumed that these adjacent buildings have shallow foundations and no basements. To mitigate any potential issues to the existing adjacent foundations resulting from the basement excavation, we recommend constructing those corners of the basement in a sequenced manner, as outlined in above.

By following the above sequential construction method, we can potentially eliminate any adverse effects on nearby existing foundations caused by the basement construction

Contact

- In case of any further clarification, please contact:

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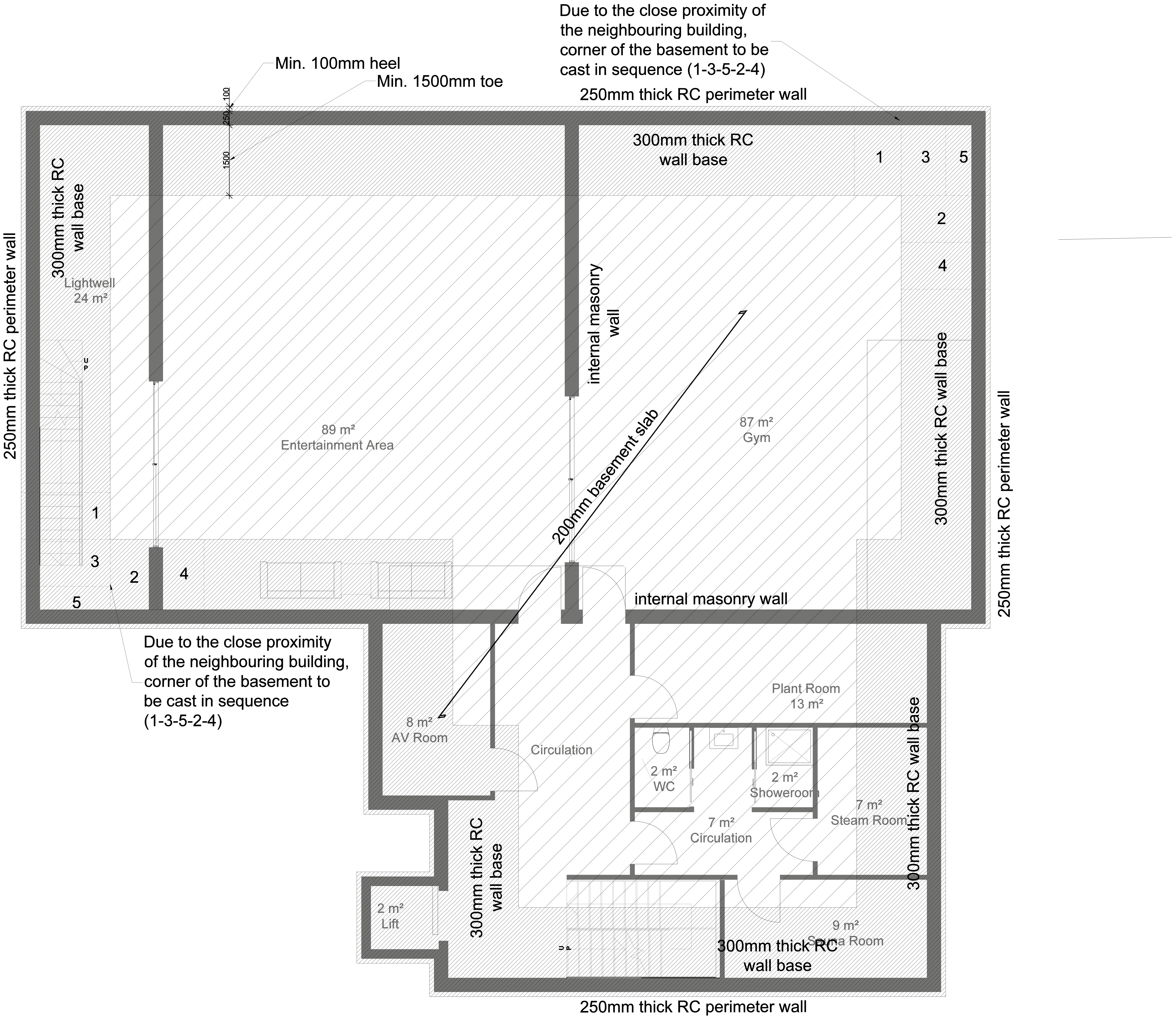
Director, Structural Engineer
e: martin@cadengineeringsolutions.co.uk

Annexure A – Structural General Arrangement drawings of Basement and Ground Floor

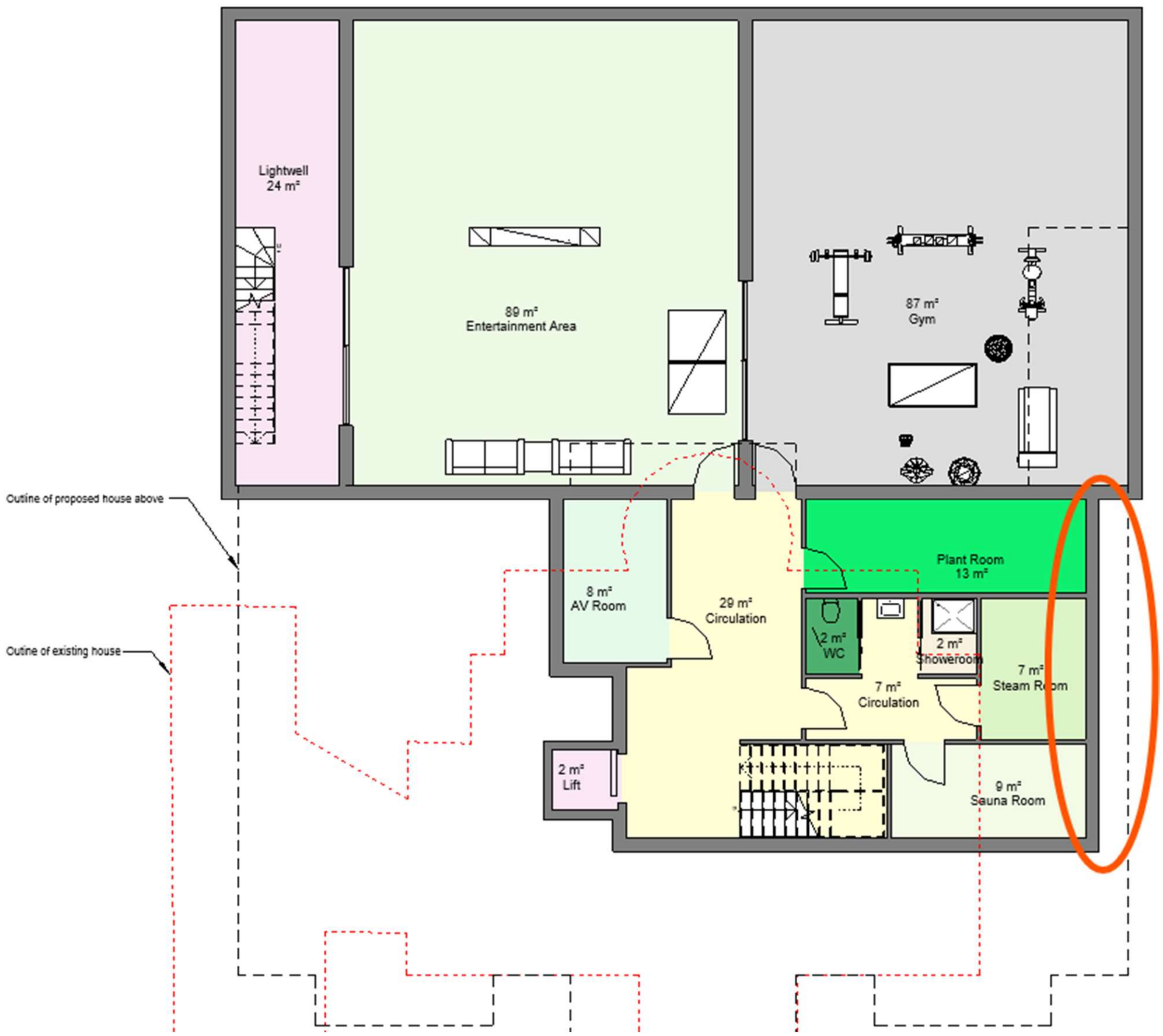


Basement Structure Plan

<u>Ground Floor Structure Plan</u>

Annexure B – Calculations

For calculation purposes, only the wall with the highest load and greatest eccentricity was analysed.



Ground floor loads:

RC Floor				6,2	/2	=	3,10 m
Dead loads:	8,2	x	3,1 =	25,42	kN/m		
Partitions:	0,50	x	3,1 =	1,55	kN/m		
Live load:	1,50	x	3,1 =	4,65	kN/m		

Eccentricity of the ground floor wall (c/c): 650mm

Superstructure loads:

HCP First Floor:	5,05	/2	=	2,53 m

Dead loads:	5	x	2,525	=	12,63 kN/m
Partitions:	0,50	x	2,525	=	1,26 kN/m
Live load:	1,50	x	2,525	=	3,79 kN/m

HCP 2nd Floor: 5,05 /2 = 2,53 m

Dead loads:	5	x	2,525	=	12,63 kN/m
Partitions:	0,50	x	2,525	=	1,26 kN/m
Live load:	1,50	x	2,525	=	3,79 kN/m

Pitched roof: 5,2 /2 = 2,6 m

Dead loads:	0,85	x	2,6	=	2,21 kN/m
Imposed:	0,6	x	2,6	=	1,56 kN/m
Snow:	0,30	x	2,6	=	0,78 kN/m
Wind (H):	0,6	x	2,6	=	1,56 kN/m

Cavity wall height: 6,5 m

Ext. brickwork:	1,80	x		6,5	=	11,70 kN/m
Insulation:	0,05	x		6,5	=	0,33 kN/m
Int. blockwork:	1,40	x		6,5	=	9,10 kN/m
Int. layers:	0,10	x		6,5	=	0,65 kN/m
			TOTAL			21,78 kN/m

Total DL: 12.63+12.63+2.21+21.78 = 49.25

Total LL: 1.26+3.79+1.26+3.79+1.56+0.78+1.56=14

RETAINING BASEMENT WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

Tedds calculation version 1.0.00

Design summary

Overall design status PASS

Overall design utilisation 0.920

Description	Unit	Provided	Required	Utilisation	Result
Bearing pressure	kN/m^2	80	73.63	0.920	PASS

Design - Combination 1.35G + 1.5Q + 1.5Qr

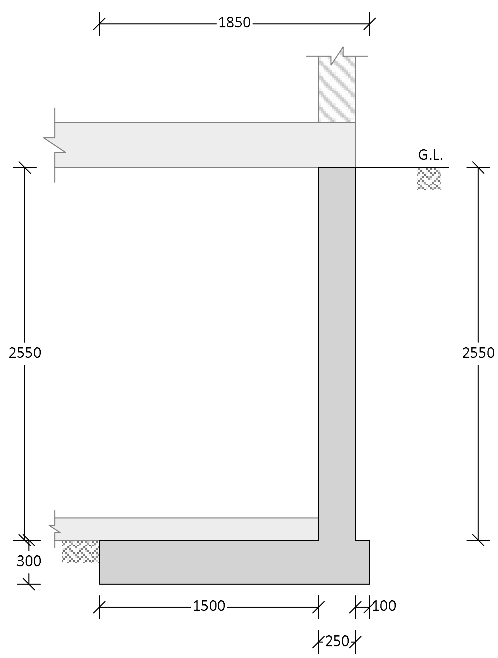
Description	Unit	Provided	Required	Utilisation	Result
Pure axial capacity	kN/m	5200.59	152.62	0.029	PASS
Bending capacity	kNm/m	103.98	-57.55	0.553	PASS
Shear axial capacity	kN/m	122.86	67.19	0.547	PASS
Foot. bending reinf.	mm^2/m	1571	713	0.454	PASS
Foot. shear capacity	kN/m	151.86	118.72	0.782	PASS

Basement wall details

Stem height	h_{stem} = **2550** mm
Thickness of stem	t_{stem} = **250** mm
Stem area	$A_{stem} = h_{stem} \times t_{stem}$ = **637500** mm^2
Density	γ_{stem} = **25** kN/m^3
Fixity at base of the wall	Fixed
Angle to rear face of basement wall	α = **90**
Retained soil height	h_{ret} = **2550** mm
Backfill soil angle	β = **0**

Strip footing details

Footing depth	$h_{footing}$ = **300** mm
Toe length	l_{toe} = **1500** mm
Heel length	l_{heel} = **100** mm
Total length	l_{total} = **1850** mm
Footing area	$A_{footing} = l_{total} \times h_{footing}$ = **555000** mm^2
Density	$\gamma_{footing}$ = **25** kN/m^3
Footing rotation	Prevented
Total height	$h_{total} = h_{stem} + h_{footing}$ = **2850** mm
Effective soil height	$h_{eff} = h_{ret} + h_{footing}$ = **2850** mm



Loading details

Axial permanent load on top of wall	P_G = **74.67** kN/m
Moment permanent load on top of wall	M_G = **32.01** kNm/m
Permanent surcharge load	$p_{G,sur}$ = **2.5** kN/m^2
Permanent surcharge load on footing toe	$p_{G,sur,toe}$ = **2.5** kN/m^2
Axial imposed load on top of wall	P_Q = **20.2** kN/m
Moment imposed load on top of wall	M_Q = **9.1** kNm/m

Imposed surcharge load | $p_{Q,sur}$ = **2** kN/m^2
Imposed surcharge load on footing toe | $p_{Q,sur,toe}$ = **2** kN/m^2

Retained soil properties

Soil type	Medium dense gravel
Moist soil height	h_{moist} = **2550** mm
Saturated soil height	h_{sat} = **0** mm
Moist density	γ_{mr} = **17** kN/m^3
Saturated density	γ_{sr} = **21** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **15** deg

Base soil properties

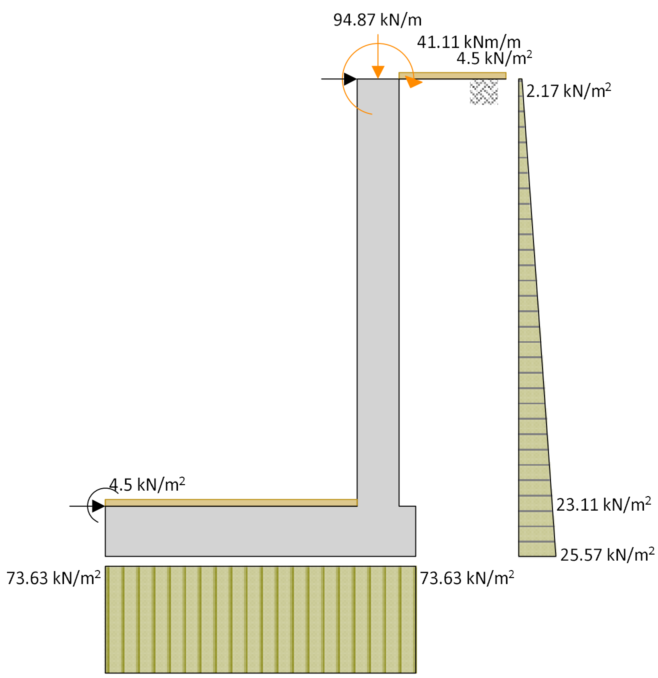
Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic cohesion	$c'_{b.k}$ = **30** kN/m^2
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg

Using Coulomb theory

At rest pressure coefficient	$K_0 = 1 - \sin(\phi'_{r.k})$ = **0.5**

Lateral pressure

Permanent surcharge pressure	$p_{Q.Surch.Press} = K_0 \times \cos(\delta_{r.d}) \times p_{Q,sur}$ = **1** kN/m^2
Imposed surcharge pressure	$p_{G.Surch.Press} = K_0 \times \cos(\delta_{r.d}) \times p_{G,sur}$ = **1.2** kN/m^2
Permanent toe surcharge pressure	$p_{Q.Surch.Toe.Press} = K_0 \times \cos(\delta_{r.d}) \times p_{Q,sur,toe}$ = **1** kN/m^2
Imposed toe surcharge pressure	$p_{G.Surch.Toe.Press} = K_0 \times \cos(\delta_{r.d}) \times p_{G,sur,toe}$ = **1.2** kN/m^2
Soil pressure at top of retained soil	$p_{Soil.Top}$ = **0** kN/m^2
Soil pressure at footing	$p_{Soil.Footing} = K_0 \times \cos(\delta_{r.d}) \times (h_{ret} + h_{footing}) \times \gamma_{mr}$ = **23.4** kN/m^2



Reactions at base of stem (from 2D analysis model)

Axial	$R_{V.StemBase}$ = **110.81** kN/m
Shear	$R_{H.StemBase}$ = **49** kN/m
Moment	$R_{M.StemBase}$ = **31.4** kNm/m

Bearing pressure check

Vertical forces

Stem	$F_{V.Stem} = A_{stem} \times \gamma_{stem} =$ **15.94** kN/m
Footing	$F_{V.Footing} = A_{footing} \times \gamma_{footing} =$ **13.88** kN/m
Retained soil	$F_{V.Soil} = h_{moist} \times l_{heel} \times \gamma_{mr} =$ **4.34** kN/m
Surcharge load	$F_{V.Surch} = p_{G,sur} \times l_{heel} + p_{Q,sur} \times l_{heel} =$ **0.45** kN/m
Toe surcharge load	$F_{V.Surch.Toe} = p_{G,sur,toe} \times l_{toe} + p_{Q,sur,toe} \times l_{toe} =$ **6.75** kN/m
Applied axial loads	$F_{V.Applied} = P_G + P_Q =$ **94.87** kN/m
Total	$F_{V.Total} = F_{V.Stem} + F_{V.Footing} + F_{V.Soil} + F_{V.Surch} + F_{V.Surch.Toe} + F_{V.Applied} =$ **136.22** kN/m

Horizontal forces

Stem	$F_{H.Stem} = R_{H.StemBase} =$ **49** kN/m
Retained soil	$F_{H.Soil} = (p_{Soil.WallBase} + 0.5 \times (p_{Soil.Footing} - p_{Soil.WallBase})) \times h_{footing} =$ **6.65** kN/m
Distance	$D_{FH.Soil} = h_{footing} \times (2 / 3 \times p_{Soil.WallBase} + p_{Soil.Footing} / 3) / (p_{Soil.WallBase} + p_{Soil.Footing}) =$ **0.147** m
Surcharge load	$F_{H.Surch} = (p_{G.Surch.Press} + p_{Q.Surch.Press}) \times h_{footing} =$ **0.65** kN/m
Prop	$F_{H.Prop} = F_{H.Stem} + F_{H.Soil} + F_{H.Surch} =$ **56.31** kN/m
Total	$F_{H.Total} = F_{H.Stem} + F_{H.Soil} + F_{H.Surch} - F_{H.Prop} =$ **0** kN/m

Moments

Stem	$M_{Stem} = R_{V.StemBase} \times (l_{toe} + t_{stem} / 2) - R_{H.StemBase} \times h_{footing} - R_{M.StemBase} =$ **133.96** kNm/m
Footing	$M_{Footing} = F_{V.Footing} \times l_{total} / 2 =$ **12.83** kNm/m
Retained soil	$M_{Soil} = F_{V.Soil} \times (l_{toe} + t_{stem} + l_{heel} / 2) - F_{H.Soil} \times D_{FH.Soil} =$ **6.82** kNm/m
Surcharge load	$M_{Surch} = F_{V.Surch} \times (l_{toe} + t_{stem} + l_{heel} / 2) - F_{H.Surch} \times h_{footing} / 2 =$ **0.71** kNm/m
Toe surcharge load	$M_{SurchToe} = F_{V.Surch.Toe} \times l_{toe} / 2 =$ **5.06** kNm/m
Horizontal base prop	$M_{H.Prop} = F_{H.Prop} \times h_{footing} =$ **16.89** kNm/m
Moment resisted by base prop	$M_{Rest.Prop} = l_{total} / 2 \times F_{V.Total} - (M_{Stem} + M_{Footing} + M_{Soil} + M_{Surch} + M_{SurchToe} + M_{H.Prop}) =$ **-50.29** kNm/m
Total	$M_{Total} = M_{Stem} + M_{Footing} + M_{Soil} + M_{Surch} + M_{SurchToe} + M_{H.Prop} + M_{Rest.Prop} =$ **126** kNm/m

Bearing pressure check

Distance to reaction	$\bar{x} = M_{Total} / F_{V.Total} =$ **925** mm
Eccentricity of reaction	$e = \bar{x} - l_{total} / 2 =$ **0** mm
Loaded length of base	$l_{load} = l_{total} =$ **1850** mm
Bearing pressure at toe	$q_{toe} = F_{V.Total} / l_{total} \times (1 - 6 \times e / l_{total}) =$ **73.6** kN/m^2
Bearing pressure at heel	$q_{heel} = F_{V.Total} / l_{total} \times (1 + 6 \times e / l_{total}) =$ **73.6** kN/m^2
Factor of safety	$FoS_{bearing} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.086**

PASS - Allowable bearing capacity exceeds maximum applied bearing pressure

RETAINING BASEMENT WALL DESIGN

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

Tedds calculation version 1.0.00

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

Concrete strength class	C32/40
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	$f_{ck} =$ **32** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **40** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = **3.0** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} \times$ AAF = **33346** N/mm^2
Compressive shortening strain - Table 3.1	$\varepsilon_{c3} =$ **0.0018**

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**
Coefficient k_1	k_1 = **0.40**
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Coefficient k_3	k_3 = **0.40**
Coefficient k_4	$k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**
Partial factor for concrete -Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **21.3** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	f_{yd} = **435** N/mm^2
Nominal cover to wall front reinforcement	$c_{nom.front}$ = **50** mm
Nominal cover to wall rear reinforcement	$c_{nom.rear}$ = **75** mm
Nominal cover to footing top reinforcement	$c_{nom.foot.top}$ = **50** mm
Nominal cover to footing bottom reinforcement	$c_{nom.foot.bot}$ = **75** mm

Wall vertical reinforcement details

Reinforcement provided	2 Layers of 16 mm ϕ bars at 200 mm c/c
Bar diameter	$\phi_{Stem.V}$ = **16** mm
Area of reinforcement provided	$A_{s.prov.Stem.V} = 2 \times \pi \times \phi_{Stem.V}^2 / (4 \times s_{Stem.V})$ = **2011** mm^2/m
Maximum allowable spacing - cl.9.6.2(3)	$s_{max.Stem.V} = \min(3 \times t_{stem}, 400mm)$ = **400** mm
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - cl.9.6.2	$A_{s.min.Stem.V} = 0.002 \times t_{stem}$ = **500** mm^2/m
	PASS - Reinforcement provided exceeds minimum reinforcement required

Wall horizontal reinforcement details

Reinforcement provided	2 Layers of 10 mm ϕ bars at 200 mm c/c
Bar diameter	$\phi_{Stem.H}$ = **10** mm
Area of reinforcement provided	$A_{s.prov.Stem.H} = 2 \times \pi \times \phi_{Stem.H}^2 / (4 \times s_{Stem.H})$ = **785** mm^2/m
Maximum allowable spacing - cl.9.6.3	$s_{max.Stem.H} = \min(3 \times t_{stem}, 400mm)$ = **400** mm
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - cl.9.6.3	$A_{s.min.Stem.H} = \max(0.25 \times A_{s.prov.Stem.V}, 0.001 \times t_{stem})$ = **503** mm^2/m
	PASS - Reinforcement provided exceeds minimum reinforcement required

Footing main reinforcement details

Reinforcement provided	20 mm ϕ bars at 200 mm c/c
Bar diameter	$\phi_{Foot.long}$ = **20** mm
Area of reinforcement provided	$A_{s.prov.Foot.long} = \pi \times \phi_{Foot.long}^2 / (4 \times s_{Foot.long})$ = **1571** mm^2/m
Maximum allowable spacing - 9.3.1.1(3)	$s_{max.Foot.long} = \min(3 \times h_{footing}, 400mm)$ = **400** mm
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - exp.9.1N	$A_{s.min.Foot.long} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times \max(d_{foot.top}, d_{foot.bot})$ = **377** mm^2/m
	PASS - Reinforcement provided exceeds minimum reinforcement required

Footing distribution reinforcement details

Reinforcement provided	10 mm ϕ bars at 200 mm c/c
Bar diameter	$\phi_{Foot.tran}$ = **10** mm

Area of reinforcement provided $A_{s.prov.Foot.tran} = \pi \times \phi_{Foot.tran}^2 / (4 \times s_{Foot.tran})$ = **393** mm²/m

Maximum allowable spacing - 9.3.1.1(3) $s_{max.Foot.tran} = \min(3.5 \times h_{footing}, 450mm)$ = **450** mm

PASS - Maximum allowable spacing exceeds reinforcement spacing

Min.area required per metre length - exp.9.1N $A_{s.min.Foot.tran} = 0.2 \times A_{s.prov.Foot.long}$ = **314** mm²/m

PASS - Reinforcement provided exceeds minimum reinforcement required

DA1 6.10 load combinations (ULSD)

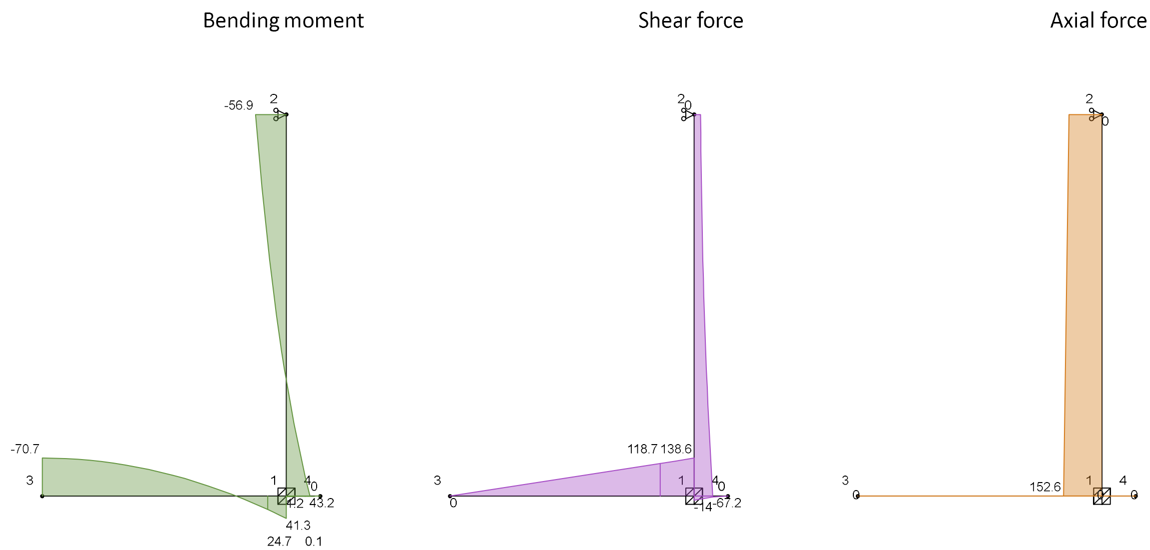
1.35G + 1.5Q + 1.5Qr (0.782)

1.00G + 1.5W (0.470)

1.35G + 1.5Q + 1.5Qr + $\psi_W \times$1.5W (0.782)

1.35G + 1.5Q + $\psi_W \times$1.5W + $\psi_S \times$1.5S (0.782)

Critical ULSD combination results: 1.35G + 1.5Q + 1.5Qr



Check pure axial capacity

Ultimate axial force $N_{Ed.max}$ = **152.62** kN/m

Strain with uniform compression $\varepsilon_0 = \varepsilon_{c3}$ = **0.00175**

Stress in reinforcement $\sigma_0 = \min(\varepsilon_0 \times E_s, f_{yd})$ = **350** N/mm²

Pure axial design capacity $N_{Rd0} = A_{s.prov.Stem.V} \times \sigma_0 + (t_{stem} - A_{s.prov.Stem.V}) \times f_{cd}$ = **5200.59** kN/m

$N_{Ed.max} / N_{Rd0}$ = **0.029**

PASS - Design axial capacity exceeds ultimate axial force

Slenderness limit - (cl.5.8.3.1)

Effective length factor (Fig. 5.7) f = **0.7**

Unsupported length l_u = **2550** mm

Effective length $l_0 = f \times l_u$ = **1785** mm

Radius of gyration $i = t_{stem} / \sqrt{(12)}$ = **7.2** cm

Slenderness ratio (5.8.3.2(1)) $\lambda_{slender} = l_0 / i$ = **24.7**

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

Smaller factored end moment $M_{1.end}$ = **-56.9** kNm/m

Larger factored end moment $M_{2.end}$ = **43.2** kNm/m

Ecc. due to geometric imperfections $e_i = l_0 / 400$ = **4.5** mm

Minimum end moment $M_{01} = \min(\text{abs}(M_{1.end}), \text{abs}(M_{2.end})) + e_i \times N_{Ed.max}$ = **43.9** kNm/m

Maximum end moment $M_{02} = \max(\text{abs}(M_{1.end}), \text{abs}(M_{2.end})) + e_i \times N_{Ed.max}$ = **57.5** kNm/m

Slenderness limit for buckling (cl. 5.8.3.1)

Area of concrete $A_c = t_{stem} - A_{s.prov.Stem.V}$ = **247989** mm²/m

Factor A A = **0.7**

Mechanical reinforcement ratio $\omega = A_{s.prov.Stem.V} \times f_{yd} / (A_c \times f_{cd})$ = **0.194**

Factor B $B = \sqrt{(1 + 2 \times \omega)}$ = **1.178**

Moment ratio $r_m = -1.0 \times M_{01} / M_{02}$ = **-0.762**

Factor C $C = 1.7 - r_m$ = **2.462**

Relative normal force $n = N_{Ed.max} / (A_c \times f_{cd})$ = **0.034**

Slenderness limit $\lambda_{lim} = 20 \times A \times B \times C / \sqrt{(n)}$ = **220.5**

$\lambda_{slender} / \lambda_{lim}$ = **0.112**

Actual slenderness ratio is less than limit, slenderness effects may be neglected

Wall design moment (positive)

Stem moment M_{stem} = **43.2** kNm/m

Wall design moment (conservative) $M_{Ed} = \max(M_{stem} + e_i \times N_{Ed.max}, N_{Ed} \times \max(t_{stem} / 30, 20mm))$ = **43.9** kNm/m

Check bending capacity when F is N_{Ed} at bottom of storey

Design axial force N_{Ed} = **152.6** kN/m

Design bending moment M_{Ed} = **43.9** kNm/m (rear face in tension)

Position of neutral axis (by iteration) z = **49.2** mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3)) $F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0mm), t_{stem})$ = **714.4** kN/m

Moment of resistance $M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2)$ = **75.2** kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement $A_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm^2/m

Depth to tension face reinforcement $d_t = t_{stem} - c_{nom.rear} - \phi_{Stem.V} / 2$ = **167** mm

Strain in tension face reinforcement $\varepsilon_s = \varepsilon_{cu3} \times (1 - d_t / z)$ = **-0.00837**

Stress in tension face reinforcement $\sigma = \max(\varepsilon_s \times E_s, - f_{yd})$ = **-434.78** N/mm^2

Force in tension face reinforcement $F_{st} = A_s \times \sigma$ = **-437.09** kN/m

Tension face reinf. moment of resistance $M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t)$ = **18.36** kNm/m

Area of compression face bars $A'_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm^2/m

Depth to compression face reinforcement $d' = c_{nom.front} + \phi_{Stem.V} / 2$ = **58** mm

Strain in compression face reinforcement $\varepsilon'_s = \varepsilon_{cu3} \times (1 - d' / z)$ = **-0.00062**

Stress in compression face reinforcement $\sigma' = \max(\varepsilon'_s \times E_s, - f_{yd})$ = **-124.46** N/mm^2

Force in compression face reinforcement $F_{sc} = A'_s \times \sigma'$ = **-125.12** kN/m

Comp. face reinf. moment of resistance $M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d')$ = **-8.38** kNm/m

Total resistance of section

Resultant concrete/steel force $F = F_c + F_{st} + F_{sc}$ = **152.15** kN/m

PASS - F is within half of one percent of N_{Ed}

Moment of resistance $M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc}$ = **85.2** kNm/m

$abs(M_{Ed}) / M_{Rd}$ = **0.515**

PASS - Design moment capacity exceeds ultimate bending moment

Check bending capacity when F is N_{Ed} at top of storey

Design axial force N_{Ed} = **131.1** kN/m

Design bending moment M_{Ed} = **43.9** kNm/m (rear face in tension)

Position of neutral axis (by iteration) z = **48.6** mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3)) $F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0mm), t_{stem})$ = **704.5** kN/m

Moment of resistance $M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2)$ = **74.4** kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement $A_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm^2/m

Depth to tension face reinforcement $d_t = t_{stem} - c_{nom.rear} - \phi_{Stem.V} / 2$ = **167** mm

Strain in tension face reinforcement $\varepsilon_s = \varepsilon_{cu3} \times (1 - d_t / z)$ = **-0.00854**

Stress in tension face reinforcement $\sigma = \max(\varepsilon_s \times E_s, - f_{yd})$ = **-434.78** N/mm^2

Force in tension face reinforcement	$F_{st} = A_s \times \sigma$ = **-437.09** kN/m
Tension face reinf. moment of resistance	$M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t)$ = **18.36** kNm/m
Area of compression face bars	$A'_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm²/m
Depth to compression face reinforcement	$d' = c_{nom.front} + \phi_{Stem.V} / 2$ = **58** mm
Strain in compression face reinforcement	$\varepsilon'_s = \varepsilon_{cu3} \times (1 - d' / z)$ = **-0.00068**
Stress in compression face reinforcement	$\sigma' = \max(\varepsilon'_s \times E_s, - f_{yd})$ = **-136.07** N/mm²
Force in compression face reinforcement	$F_{sc} = A'_s \times \sigma'$ = **-136.79** kN/m
Comp. face reinf. moment of resistance	$M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d')$ = **-9.17** kNm/m

Total resistance of section

Resultant concrete/steel force	$F = F_c + F_{st} + F_{sc}$ = **130.57** kN/m
	PASS - F is within half of one percent of N_{Ed}
Moment of resistance	$M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc}$ = **83.57** kNm/m
	$\text{abs}(M_{Ed}) / M_{Rd}$ = **0.525**
	PASS - Design moment capacity exceeds ultimate bending moment

Wall design moment (negative)

Stem moment	M_{stem} = **-56.9** kNm/m
Wall design moment (conservative)	$M_{Ed} = \min(M_{stem} - e_i \times N_{Ed.max}, -N_{Ed} \times \max(t_{stem} / 30, 20mm))$ = **-57.5** kNm/m

Check bending capacity when F is N_{Ed} at bottom of storey

Design axial force	N_{Ed} = **152.6** kN/m
Design bending moment	M_{Ed} = **-57.5** kNm/m (front face in tension)
Position of neutral axis (by iteration)	z = **59.6** mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3))	$F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0mm), t_{stem})$ = **865.2** kN/m
Moment of resistance	$M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2)$ = **87.5** kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement	$A_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm²/m
Depth to tension face reinforcement	$d_t = t_{stem} - c_{nom.front} - \phi_{Stem.V} / 2$ = **192** mm
Strain in tension face reinforcement	$\varepsilon_s = \varepsilon_{cu3} \times (1 - d_t / z)$ = **-0.00777**
Stress in tension face reinforcement	$\sigma = \max(\varepsilon_s \times E_s, - f_{yd})$ = **-434.78** N/mm²
Force in tension face reinforcement	$F_{st} = A_s \times \sigma$ = **-437.09** kN/m
Tension face reinf. moment of resistance	$M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t)$ = **29.29** kNm/m
Area of compression face bars	$A'_s = A_{s.prov.Stem.V} / 2$ = **1005.3** mm²/m
Depth to compression face reinforcement	$d' = c_{nom.rear} + \phi_{Stem.V} / 2$ = **83** mm
Strain in compression face reinforcement	$\varepsilon'_s = \varepsilon_{cu3} \times (1 - d' / z)$ = **-0.00137**
Stress in compression face reinforcement	$\sigma' = \max(\varepsilon'_s \times E_s, - f_{yd})$ = **-274.1** N/mm²
Force in compression face reinforcement	$F_{sc} = A'_s \times \sigma'$ = **-275.56** kN/m
Comp. face reinf. moment of resistance	$M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d')$ = **-11.57** kNm/m

Total resistance of section

Resultant concrete/steel force	$F = F_c + F_{st} + F_{sc}$ = **152.59** kN/m
	PASS - F is within half of one percent of N_{Ed}
Moment of resistance	$M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc}$ = **105.22** kNm/m
	$\text{abs}(M_{Ed}) / M_{Rd}$ = **0.547**
	PASS - Design moment capacity exceeds ultimate bending moment

Check bending capacity when F is N_{Ed} at top of storey

Design axial force	N_{Ed} = **131.1** kN/m
Design bending moment	M_{Ed} = **-57.5** kNm/m (front face in tension)
Position of neutral axis (by iteration)	z = **59** mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3))	$F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0mm), t_{stem})$ = **855.3** kN/m

Moment of resistance $M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2) =$ **86.7** kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement $A_s = A_{s.prov.Stem.V} / 2 =$ **1005.3** mm^2/m

Depth to tension face reinforcement $d_t = t_{stem} - c_{nom.front} - \phi_{Stem.V} / 2 =$ **192** mm

Strain in tension face reinforcement $\varepsilon_s = \varepsilon_{cu3} \times (1 - d_t / z) =$ **-0.0079**

Stress in tension face reinforcement $\sigma = \max(\varepsilon_s \times E_s, - f_{yd}) =$ **-434.78** N/mm^2

Force in tension face reinforcement $F_{st} = A_s \times \sigma =$ **-437.09** kN/m

Tension face reinf. moment of resistance $M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t) =$ **29.29** kNm/m

Area of compression face bars $A'_s = A_{s.prov.Stem.V} / 2 =$ **1005.3** mm^2/m

Depth to compression face reinforcement $d' = c_{nom.rear} + \phi_{Stem.V} / 2 =$ **83** mm

Strain in compression face reinforcement $\varepsilon'_s = \varepsilon_{cu3} \times (1 - d' / z) =$ **-0.00143**

Stress in compression face reinforcement $\sigma' = \max(\varepsilon'_s \times E_s, - f_{yd}) =$ **-285.4** N/mm^2

Force in compression face reinforcement $F_{sc} = A'_s \times \sigma' =$ **-286.91** kN/m

Comp. face reinf. moment of resistance $M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d') =$ **-12.05** kNm/m

Total resistance of section

Resultant concrete/steel force $F = F_c + F_{st} + F_{sc} =$ **131.32** kN/m

PASS - F is within half of one percent of N_{Ed}

Moment of resistance $M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc} =$ **103.98** kNm/m

$abs(M_{Ed}) / M_{Rd} =$ **0.553**

PASS - Design moment capacity exceeds ultimate bending moment

Check shear capacity of wall (cl.6.2.2)

Design shear force $V =$ **67.19** kN/m

Depth to tension steel $d_v = \min(t_{stem} - c_{nom.front} - \phi_{Stem.V} / 2, t_{stem} - c_{nom.rear} - \phi_{Stem.V} / 2) =$ **167** mm

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

$k_v = \min(1 + \sqrt{(200mm / d_v)}, 2) =$ **2.000**

Longitudinal reinforcement ratio $\rho_l = \min(A_{s.prov.Stem.V} / (2 \times d_v), 0.02) =$ **0.006**

$v_{min} = 0.035\ N/mm^2 \times k_v^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5} =$ **0.560** N/mm^2

$k_{1.v} =$ **0.15**

Comp stress in concrete from axial loading $\sigma_{cp} = N_{Ed.max} / (t_{stem} - A_{s.prov.Stem.V}) =$ **0.615** N/mm^2

Design shear resistance (exp.6.2a & 6.2b) $V_{Rd.c} = (\max(C_{Rd,c} \times k_v \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) + k_{1.v} \times \sigma_{cp}) \times d_v =$ **122.9** kN/m

$V / V_{Rd.c} =$ **0.547**

PASS - Design concrete shear capacity exceeds ultimate shear force

Check footing in flexure - Section 6.1

Design bending mnt, 0 mm from wall face $M_{Ed.foot} =$ **24.7** kNm/m

Depth to tension reinforcement $d_{foot.bot} = h_{footing} - c_{nom.foot.bot} - \phi_{Foot.long} / 2 =$ **215** mm

$K_{foot} = abs(M_{Ed.foot}) / (d_{foot.bot}^2 \times f_{ck}) =$ **0.017**

$K'_{foot} = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1)/(2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z_{foot} = \min(0.5 + 0.5 \times (1 - 2 \times K_{foot} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d_{foot.bot} =$ **204** mm

Depth of neutral axis $x_{foot} = 2.5 \times (d_{foot.bot} - z_{foot}) =$ **27** mm

Area of tension reinforcement required $A_{s.des} = abs(M_{Ed.foot}) / (f_{yd} \times z_{foot}) =$ **278** mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{foot.bot} =$ **338** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s.max} = 0.04 \times h_{footing} =$ **12000** mm^2/m

Area of tension reinforcement provided $A_{s.prov.Foot.long} =$ **1571** mm^2/m

$abs(A_{s.req}) / A_{s.prov.Foot.long} =$ **0.215**

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

Check footing in flexure - Section 6.1

Design bending mnt, 1500 mm from wall face — $M_{Ed.foot}$ = **-70.7** kNm/m

Depth to tension reinforcement — $d_{foot.top} = h_{footing} - c_{nom.foot.top} - \phi_{Foot.long} / 2$ = **240** mm

$K_{foot} = abs(M_{Ed.foot}) / (d_{foot.top}^2 \times f_{ck})$ = **0.038**

$K'_{foot} = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1)/(2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z_{foot} = \min(0.5 + 0.5 \times (1 - 2 \times K_{foot} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d_{foot.top}$ = **228** mm

Depth of neutral axis — $x_{foot} = 2.5 \times (d_{foot.top} - z_{foot})$ = **30** mm

Area of tension reinforcement required — $A_{s.des} = abs(M_{Ed.foot}) / (f_{yd} \times z_{foot})$ = **713** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{foot.top}$ = **377** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s.max} = 0.04 \times h_{footing}$ = **12000** mm²/m

Area of tension reinforcement provided — $A_{s.prov.Foot.long}$ = **1571** mm²/m

$abs(A_{s.req}) / A_{s.prov.Foot.long}$ = **0.454**

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

Check shear capacity of footing (cl.6.2.2)

Design shear force — V_{foot} = **118.72** kN/m

Depth to tension steel — $d_{v.foot} = d_{foot.top}$ = **240** mm

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

$k_{v.foot} = \min(1 + \sqrt{(200mm / d_{v.foot})}, 2)$ = **1.913**

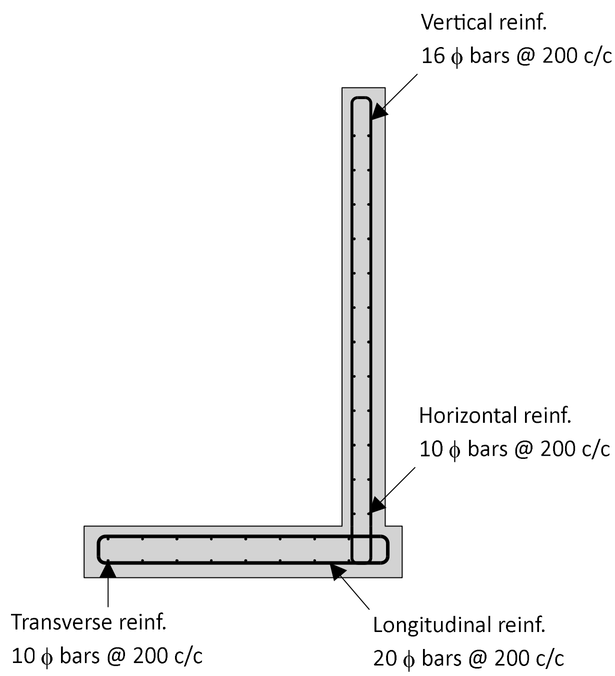
Main reinforcement ratio — $\rho_{l.foot} = \min(A_{s.prov.Foot.long} / d_{v.foot}, 0.02)$ = **0.007**

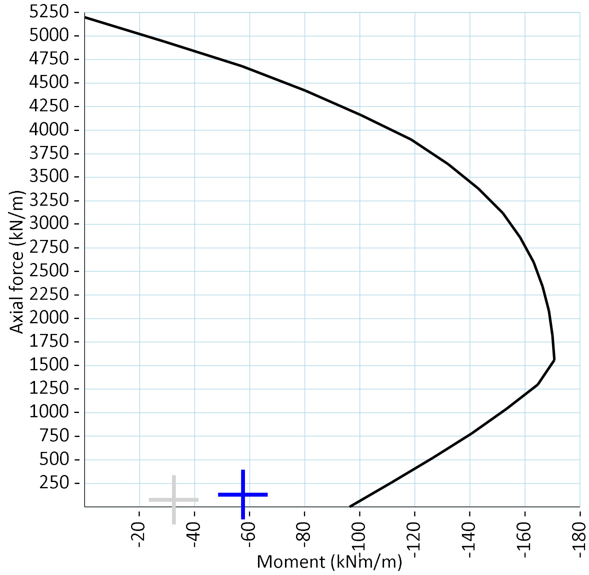
$v_{min.foot} = 0.035\ N/mm^2 \times k_{v.foot}^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5}$ = **0.524** N/mm²

Design shear resistance (exp.6.2a & 6.2b) — $V_{Rd.c.foot} = \max(C_{Rd,c} \times k_{v.foot} \times (100\ N^2/mm^4 \times \rho_{l.foot} \times f_{ck})^{1/3}, v_{min.foot}) \times d_{v.foot}$ = **151.9** kN/m

$V_{foot} / V_{Rd.c.foot}$ = **0.782**

PASS - Design concrete shear capacity exceeds ultimate shear force

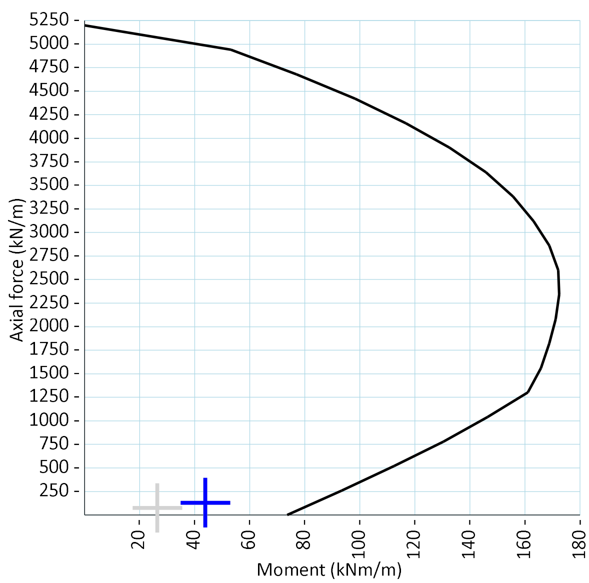




Note: Crit. combination hightlighted: 1.35G + 1.5Q + 1.5Qr

N_{Ed} at top of storey

Interaction diagram - Negative moments



Note: Crit. combination hightlighted: 1.35G + 1.5Q + 1.5Qr

N_{Ed} at top of storey

Interaction diagram - Positive moments