

15 Woodside Rd, HA6 3QE

By CAD Engineering Solutions

Project No

CES_KK_93/2026

Date

March 2026

Document

Basement Impact Assessment

C1	2026.03.17	KK	MP
Rev	Date	Prepared	Checked

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1 Introduction

CAD Engineering Solutions has been appointed to prepare a Basement Impact Assessment (BIA) for the property located at 15 Woodside Road, Northwood, HA6 3QE, which is undergoing the construction of a new rear extension and basement beneath the area of the existing patio. The proposed works will include the partial underpinning of the existing rear wall; the existing house on the site will be retained.

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.

2 Proposed Development and Structural Methodology

- **Scope of Works:** The proposed scheme involves the construction of a new basement located beneath the footprint of the existing rear patio. The excavation will reach a depth of 3.0m to the Finished Floor Level (FFL).
- **Underpinning:** The construction will involve the partial underpinning of the existing rear wall of the host property.
- **Construction Sequencing:** To ensure ground stability, the basement retaining walls will be constructed using a sequential underpinning method. Reinforced concrete walls will be cast in strict, carefully managed sequences. This methodology is specifically chosen to ensure that the surrounding ground, particularly on adjacent plots, is not disturbed or undermined during the works.

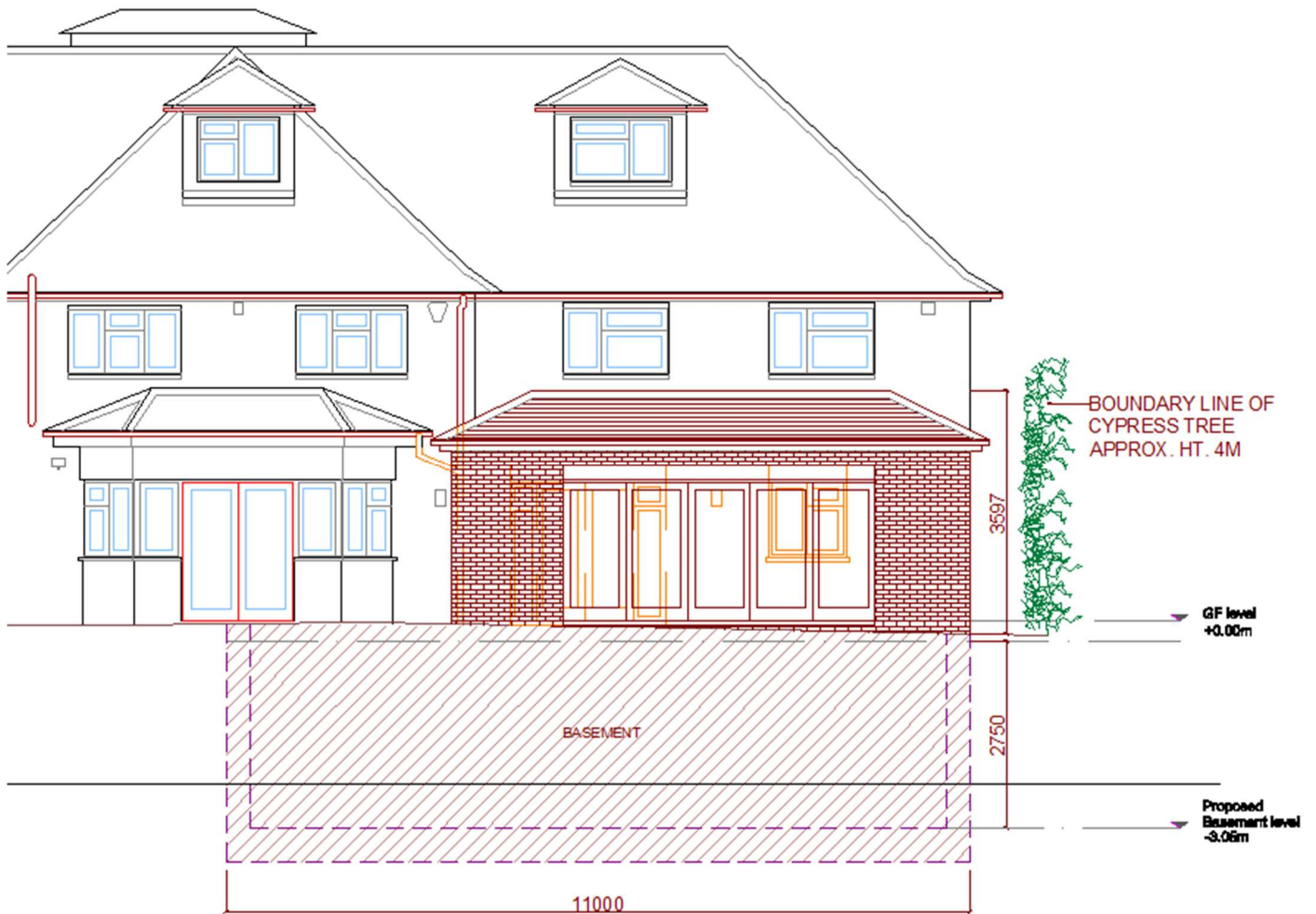


Figure 1. Proposed new extension rear elevation

3 Site Description

- **Proximity:** The closest neighbouring buildings are situated approximately 5.4m from the perimeter of the proposed basement excavation.
- **Assessment of Impact:** Given the relatively shallow basement depth of 3.0m to FFL, combined with the substantial 5.4m separation distance and the implementation of sequential concrete underpinning, the theoretical zone of influence from the excavation will not intersect with the foundations of the neighbouring properties. Therefore, the proposed works will not have any negative structural impact on adjacent buildings.



Figure 2. Distance between the proposed basement corner and the corner of the existing neighbours extension



Figure 3. Distance between the proposed basement corner and the corner of the existing neighbours building

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4 Site Geological and Hydrogeological Conditions

4.1 Site Geology

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

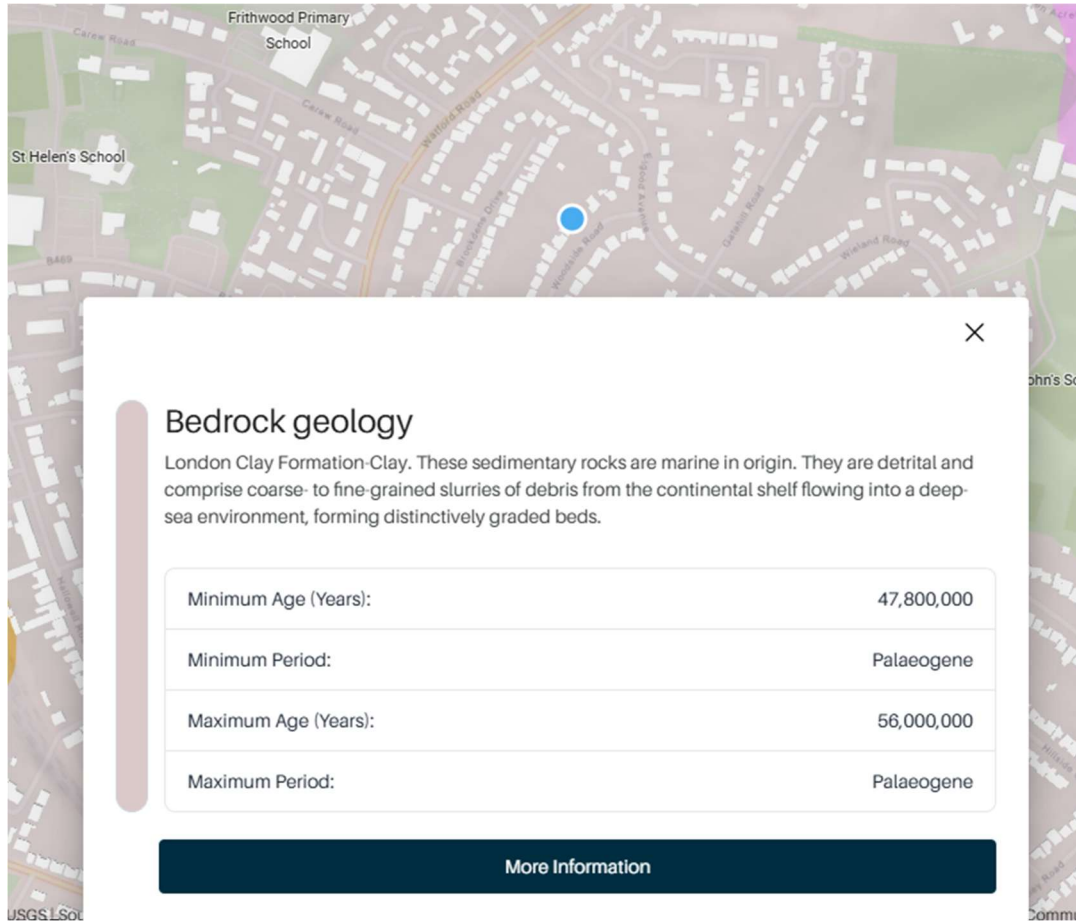


Figure 4. Image showing the bedrock geology details (Source – BGS maps)

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

ENVIRONMENT AGENCY

Form WR - 38	Ref: elfesselle_gate_end_northwood.doc	Agency No.
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BOREHOLE RECORD

Accession No. 46977
 A SITE DETAILS
 THAMES EA

256

TQ19/74
 TQ19SW/132

Borehole drilled for:	Mr Lawson		
Location:	Elfesselle, Gate End, Northwood, Middlesex		
N.G.R.:	TQ 101 913		
Ground Level (if known):	SURFACE		
Drilling Company:	W.B. & A.D. MORGAN LTD., PRESTEIGNE, POWYS. LD8 2UF		
Date of Drilling:	Commenced:	7/6/07	Completed: 20/6/07

B. CONSTRUCTION DETAILS

Borehole datum (if not ground level)		GROUNDLEVEL	
(Point from which all measurements of depth are taken e.g. flange, edge of chamber, etc.)			
Borehole drilled diameter.....	315 mm from	Surface to	43 m/depth
	200 mm from	43 to	75 m/depth
	mm from	to	m/depth
Casing material: u.P.V.C diameter and type (e.g. plain steel, plastic slotted)	103 mm from	Surface to	75 m/depth
Plain diameter	103 mm from	Surface to	45 m/depth
Slotted diameter	103 mm from	45 to	63 m/depth
Plain diameter	103 mm from	63 to	69 m/depth
Slotted diameter	103 mm from	69 to	75 m/depth
Grouting details:	Pressure grout 43m to surface		
Water struck at:	54 m (depth below datum - mbd)		
Rest water level on completion:	44 m (depth below datum - mbd)		
Estimated blowout yield:	360 Gallons per hour		

C. STRATA LOG

Description of Strata	Thickness (m)	Depth (m)
Red brick	1	1
Consolidated brown clay with cobbles	9	10
Consolidated grey clay	16	26
Brown clay with yellow/green sands	7	33
Soft brown/yellow/green sands	3	36
Hard black/brown flint	4	40
Medium white chalk & flints	35	75
Other Comments (e.g. gas encountered, saline water intercepted, etc.)		Lined borehole due to flint bands
Gravel Pack Quantity:	None	Temp Steel Casing: 2m x 340mm
Cement:	2,000kg	Depth and Diameter
Rig & Crew:	Halco V666, D. Morris, J. Scott	

Figure 5. Image showing the bore hole results (Source – BGS maps)

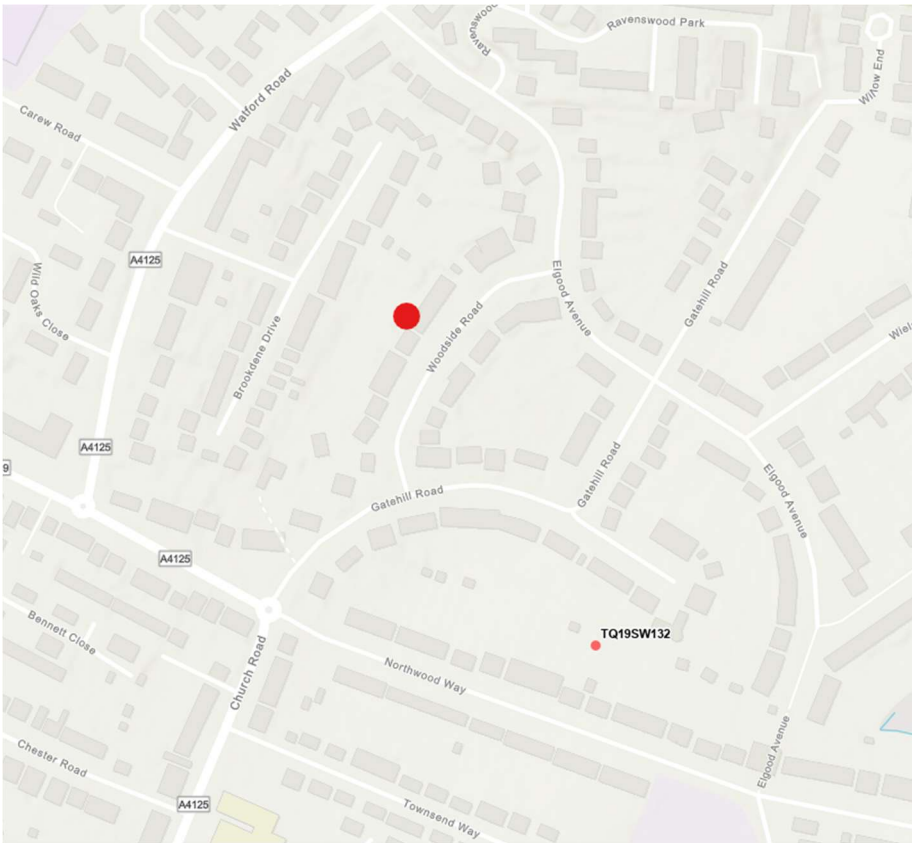


Figure 6. Image showing the location of the nearest borehole (approximately 250m from the proposed basement)

4.2 Site Hydrogeology

Although no groundwater has been observed at basement height in nearby borehole, 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, indicates the site is in Flood Zone 1, and not at risk of flooding from rivers or the sea.

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Surface water

[More about your surface water flood risk](#)

Yearly chance of flooding

Very low

Low

Medium

High

Yearly chance of flooding between 2040 and 2060

Very low

Low

Medium

High

Rivers and the sea

[More about your rivers and sea flood risk](#)

Yearly chance of flooding

Very low

Low

Medium

High

Yearly chance of flooding between 2036 and 2069

Very low

Low

Medium

High

Groundwater

[More about your groundwater flood risk](#)

We use groundwater flood alert areas to check the risk of flooding from groundwater.

This location is outside of a groundwater flood alert area.

▶ [What this means](#)

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.

5 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.

5.1 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 is 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?

At this stage, it is not possible to confirm whether neighbouring properties have experienced subsidence or have been underpinned. A site investigation and verification of neighbouring foundations will be required at the appropriate stage of the construction process to establish the actual ground conditions and any historic or existing structural interventions.

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?*

The closest neighbouring building, No. 13 Woodside Rd, is located approximately 5.4m away, which is safely outside the zone of influence. However, to ensure general excavation safety and to prevent any disturbance to the ground of the adjacent plot (as the basement wall is set back ~1.1m from the boundary), the proposed retaining walls will be constructed in a carefully managed sequence.

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

No

5.2 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?*

According to the nearest boreholes, the water table is below the basement depth.

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

Due to the extension 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

5.3 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.

Floor span: $6,5 / 2 = 3,25 \text{ m}$

Dead loads: $8 \times 3,25 = 26,00 \text{ kN/m}$

Live loads: $1,50 \times 3,25 = 4,88 \text{ kN/m}$

Partitions $0,50 \times 3,25 = 1,63 \text{ kN/m}$

Allowance for existing building's wall: 20 kN/m

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Allowance for the existing 1st floor and attic:

Floor span: $4 / 2 = 2,00 \text{ m}$

Dead loads: $0,7 \times 2 = 1,40 \text{ kN/m}$

Imposed $0,60 \times 2 = 1,20 \text{ kN/m}$

Partitions $0,30 \times 2 = 0,60 \text{ kN/m}$

Loft floor: $4 / 2 = 2,00 \text{ m}$

Dead loads: $0,5 \times 2 = 1,00 \text{ kN/m}$

Live load: $0,60 \times 2 = 1,20 \text{ kN/m}$

5.4 Summary of Screening

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

- The basement walls located near the boundary line should be cast in sequence to prevent any ground disturbance on the adjacent plot.
- 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.

6 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.

6.1 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. While the adjacent building at No. 13 Woodside Rd is situated at a safe distance of approximately 5.4m, the basement excavation is located approximately 1.1m from the boundary line. Therefore, to ensure overall stability, prevent any disturbance to the adjacent plot's ground, and safely manage the underpinning of the host property, the walls must be constructed in sequence. Additionally, temporary supports must be installed as detailed in the Basement Construction Method Statement to ensure complete safety during the construction process.

6.2 Hydrogeology

The water table is located below the proposed founding depth. The exact level 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.

6.3 Surface Flow

Currently, the 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.

6.4 Arboricultural Impact and Mitigation

The arboricultural impact of the proposed basement has been fully assessed. The proposed site layout allows for the healthy retention of all trees on the site and within nearby adjacent sites. The scheme does not require the removal of any trees, nor does it require any facilitation pruning to accommodate the construction.

- Impact on T1 (Eucalyptus, Category B1): The proposed basement will result in a minor 3% encroachment into the Root Protection Area (RPA) of T1. The area of excavation currently exists as a well-engineered patio with an assumed sub-base in excess of 300mm and an existing retaining wall. Due to these existing subterranean structures, it is highly unlikely that there are any significant roots from T1 in the area where the new basement will sit. T1 is a healthy tree that will tolerate this negligible root loss and recover quickly, especially given the large areas of open ground available on other sides of the RPA for compensatory root growth to develop.
- Impact on Other Trees: The proposed new structures are situated completely outside of the assessed RPAs of all of the other trees. Consequently, these trees pose no below-ground constraints on the new structure, and the basement will not impact them.
- Tree Protection Measures: The retained trees require protection in accordance with industry best practice and BS 5837: 2012 to ensure their longevity. Protective fencing must be erected prior to any works commencing in the vicinity of the trees. The existing hard surfacing provides adequate ground protection and must be retained in situ for the entirety of the site works. Furthermore, any new services or inspection chambers associated with the basement must be routed to avoid all RPAs of retained trees.

7 Basement Construction Method Statement

7.1 General recommendations

The proposed basement has a footprint of ~7.1 m in length and ~11.3 m in width, with a depth of ~3.3 m below the existing ground level. The basement walls will be set back ~1.1 m from the boundary with No. 13. To completely prevent any ground disturbance to the adjacent plot and to guarantee overall safety during the excavation, we recommend constructing the basement sequentially, as outlined below.

7.2 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 340mm thick reinforced concrete (RC) walls. The base will consist of a 300 mm thick basement slab with a 150 mm heel. Refer to annexure for basement wall calculations.

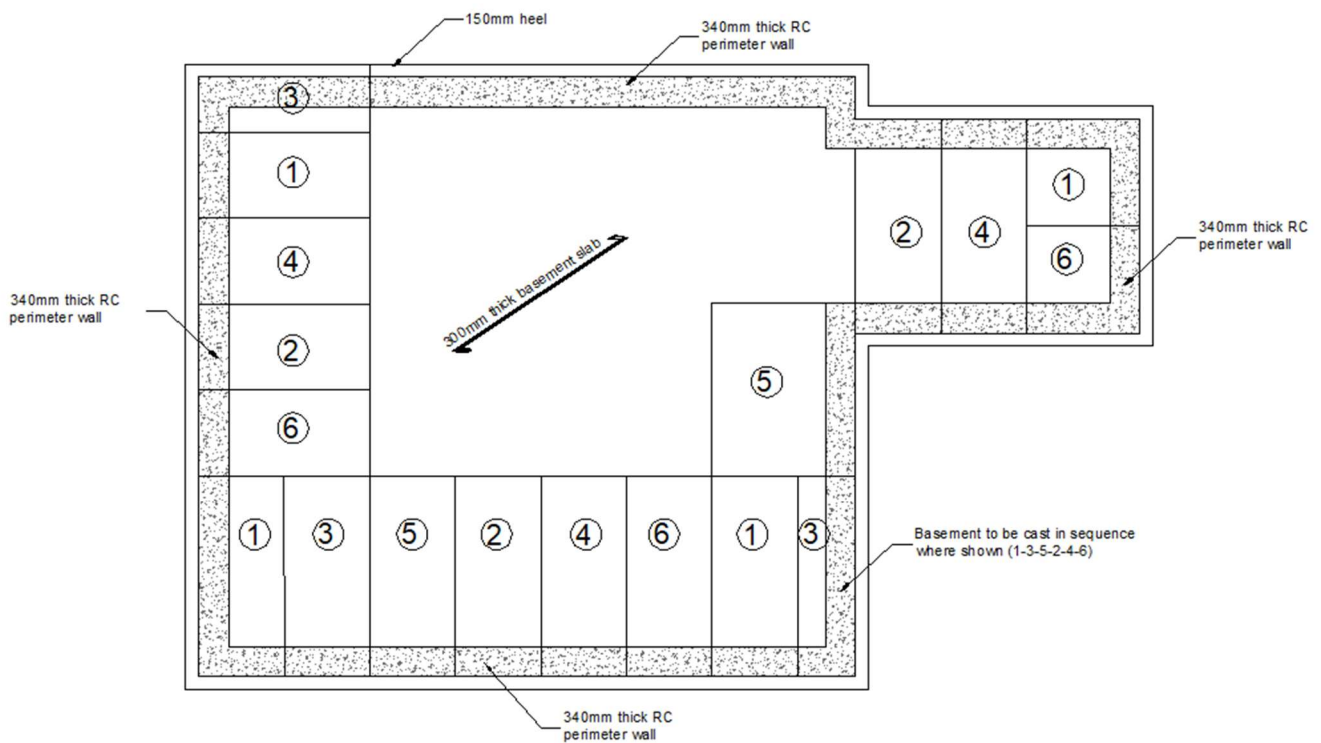
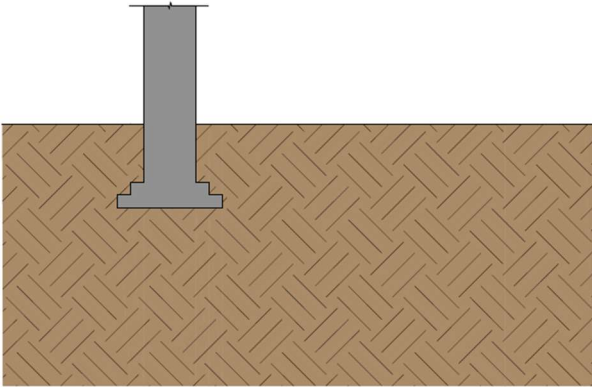


Figure 7. Basement Structure Plan

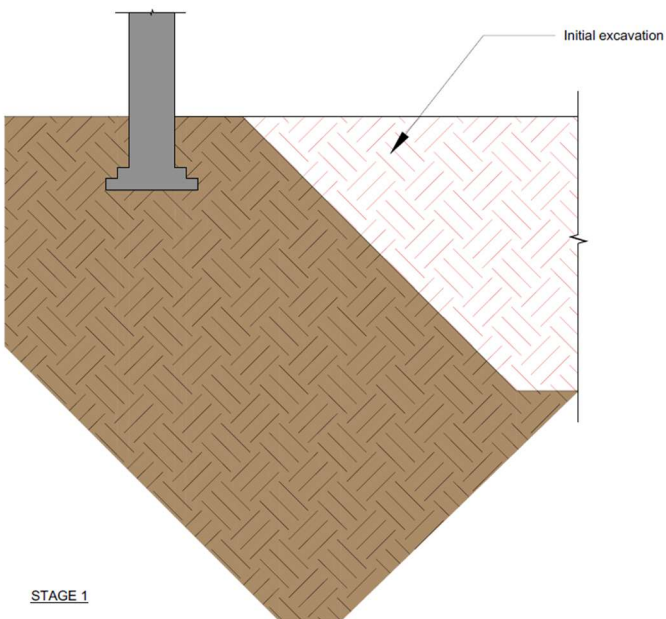
Stage 0



STAGE 0

Stage 1

- Perform initial excavation for the proposed basement.



STAGE 1

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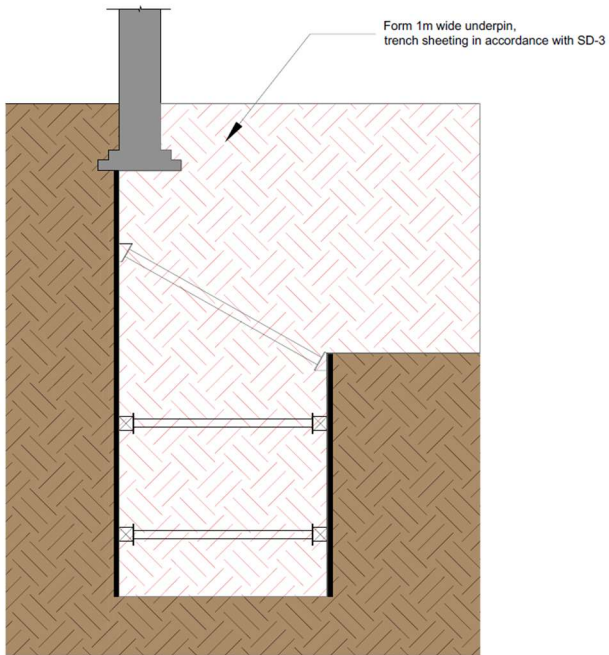
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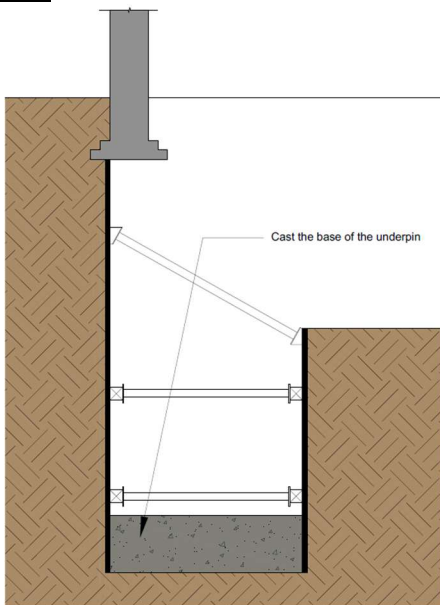
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Stage 2



Stage 3



STAGE 3

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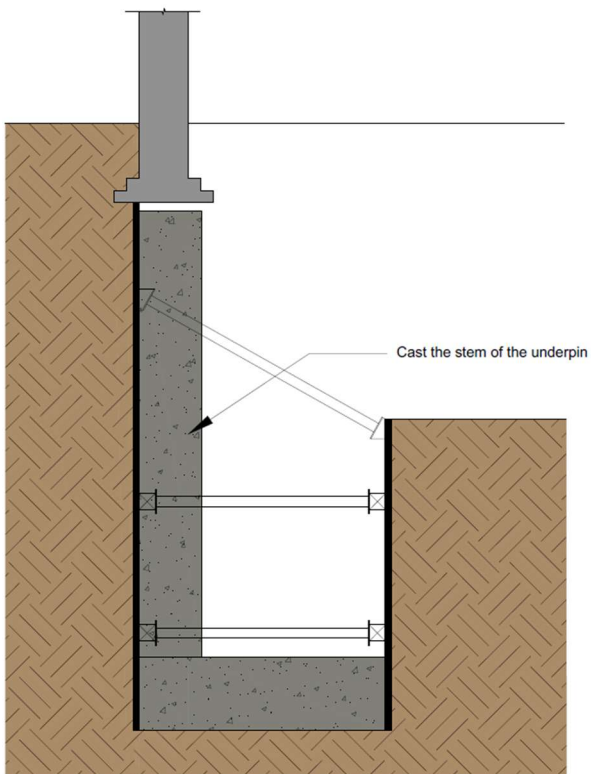
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Stage 4



STAGE 4

Stage 5

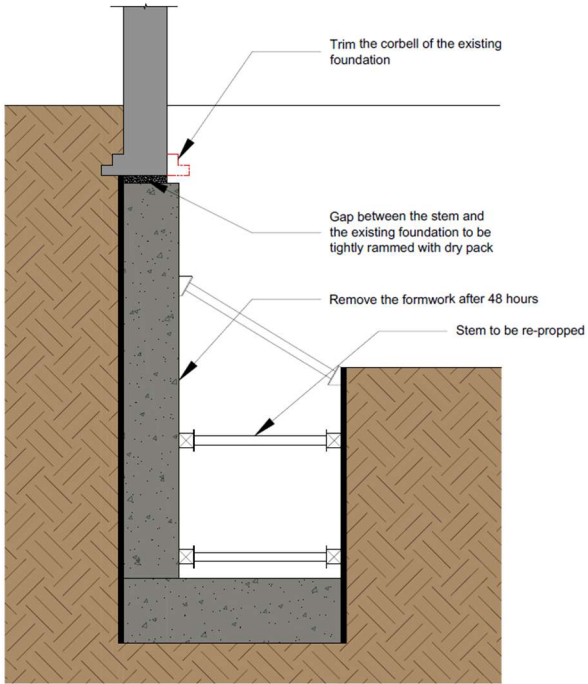
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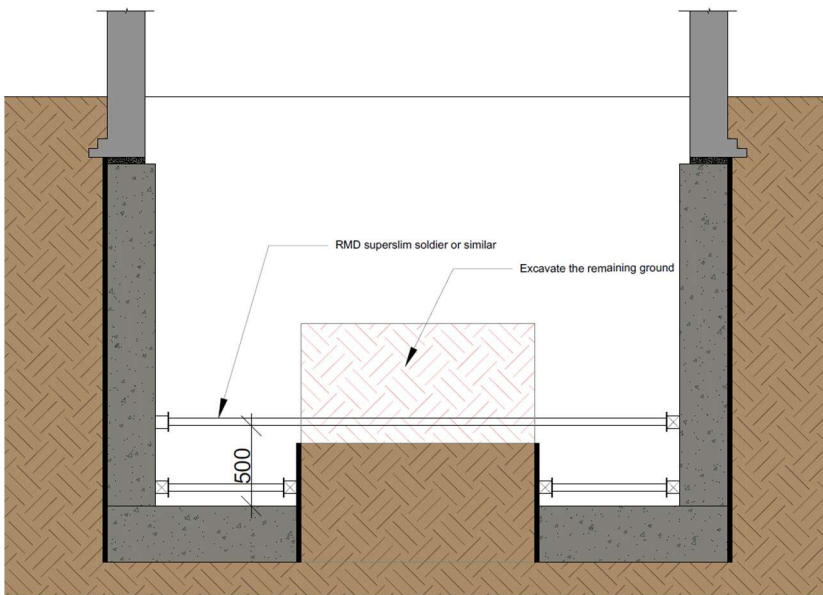
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STAGE 5

Stage 6



STAGE 6

Stage 7

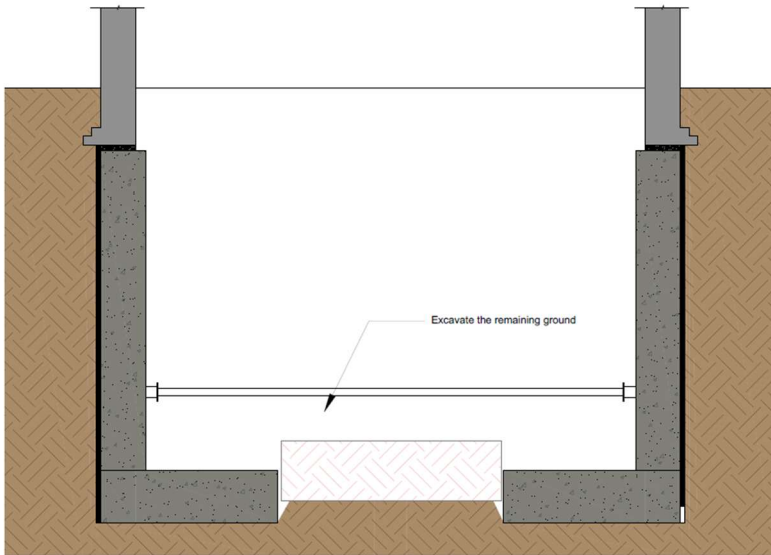
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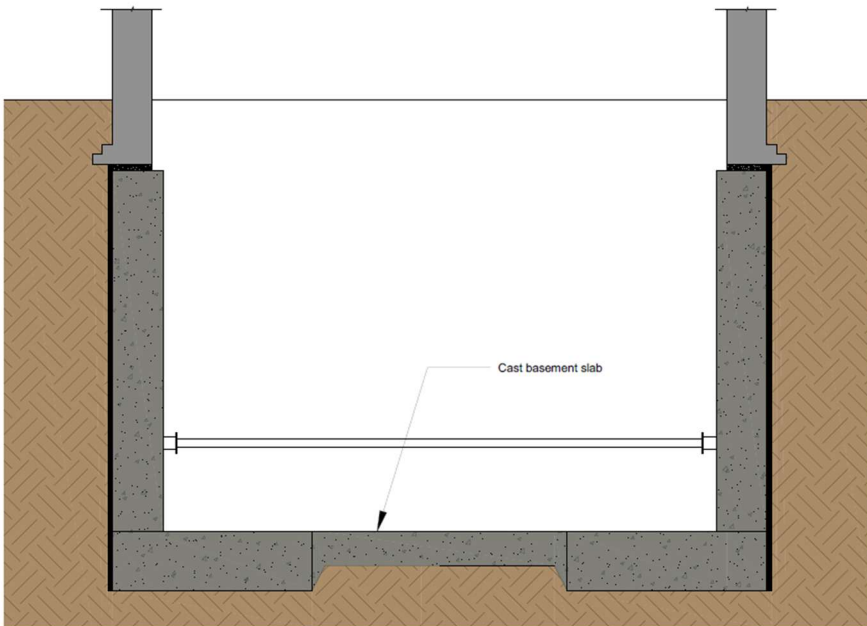


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STAGE 7

Stage 8



STAGE 8

Stage 9

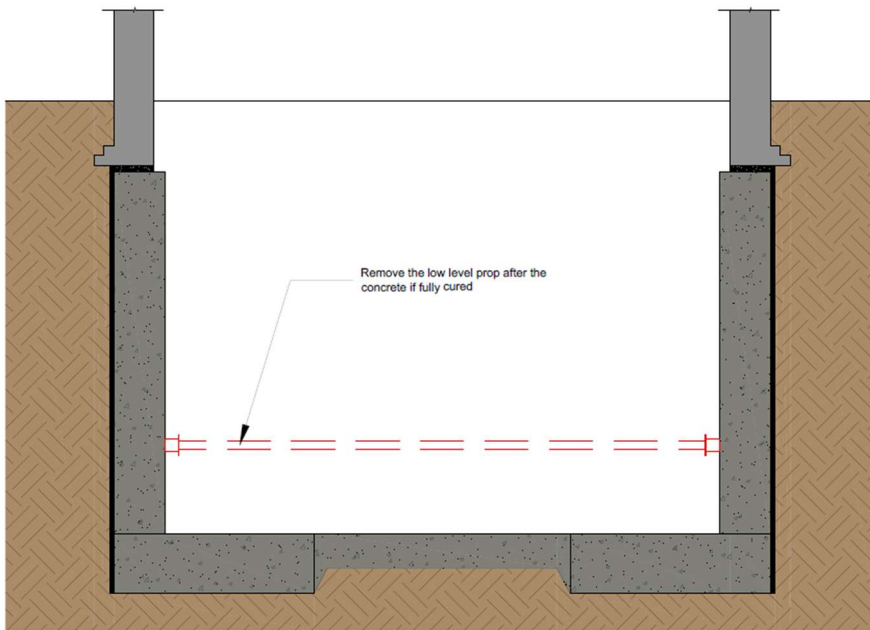
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STAGE 9

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 **0.915**

Description	Unit	Provided	Required	Utilisation	Result
Bearing pressure	kN/m ²	100	37.72	0.377	PASS

Design - Combination 1.35G + 1.5Q + 1.5Qr

Description	Unit	Provided	Required	Utilisation	Result
Pure axial capacity	kN/m	5919.52	114.03	0.019	PASS
Bending capacity	kNm/m	105.18	-30.68	0.292	PASS
Shear axial capacity	kN/m	142.35	48.62	0.342	PASS
Foot. bending reinf.	mm ² /m	2011	1839	0.915	PASS
Foot. shear capacity	kN/m	162.41	101.95	0.628	PASS

Basement wall details

Stem height	$h_{\text{stem}} = 3000$ mm
Thickness of stem	$t_{\text{stem}} = 340$ mm
Stem area	$A_{\text{stem}} = h_{\text{stem}} \times t_{\text{stem}} = 1020000$ mm ²
Density	$\gamma_{\text{stem}} = 25$ kN/m ³
Fixity at base of the wall	Pinned
Angle to rear face of basement wall	$\alpha = 90$
Retained soil height	$h_{\text{ret}} = 3000$ mm

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Backfill soil angle

$\beta = 0$

Strip footing details

Footing depth

$h_{\text{footing}} = 300 \text{ mm}$

Toe length

$l_{\text{toe}} = 3000 \text{ mm}$

Heel length

$l_{\text{heel}} = 150 \text{ mm}$

Total length

$l_{\text{total}} = 3490 \text{ mm}$

Footing area

$A_{\text{footing}} = l_{\text{total}} \times h_{\text{footing}} = 1047000 \text{ mm}^2$

Density

$\gamma_{\text{footing}} = 25 \text{ kN/m}^3$

Footing rotation

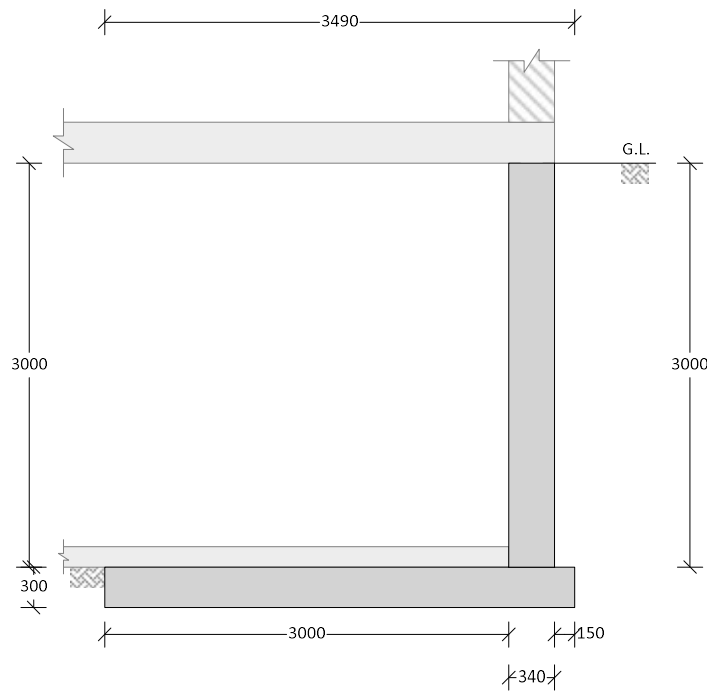
Prevented

Total height

$h_{\text{total}} = h_{\text{stem}} + h_{\text{footing}} = 3300 \text{ mm}$

Effective soil height

$h_{\text{eff}} = h_{\text{ret}} + h_{\text{footing}} = 3300 \text{ mm}$



Loading details

Axial permanent load on top of wall

$P_G = 48.4 \text{ kN/m}$

Permanent surcharge load

$p_{G,\text{sur}} = 5 \text{ kN/m}^2$

Permanent surcharge load on footing toe

$p_{G,\text{sur,toe}} = 2 \text{ kN/m}^2$

Axial imposed load on top of wall

$P_Q = 9.51 \text{ kN/m}$

Moment imposed load on top of wall

$M_Q = 1.62 \text{ kNm/m}$

Imposed surcharge load

$p_{Q,\text{sur}} = 2 \text{ kN/m}^2$

Imposed surcharge load on footing toe

$p_{Q,\text{sur,toe}} = 2 \text{ kN/m}^2$

Retained soil properties

Soil type

Medium dense well graded sand

Moist soil height

$h_{\text{moist}} = 3000 \text{ mm}$

Saturated soil height

$h_{\text{sat}} = 0 \text{ mm}$

Moist density

$\gamma_{\text{mr}} = 20 \text{ kN/m}^3$

Saturated density

$\gamma_{\text{sr}} = 22 \text{ kN/m}^3$

Characteristic effective shear resistance angle

$\phi'_{r,k} = 30 \text{ deg}$

Characteristic wall friction angle

$\delta_{r,k} = 0 \text{ deg}$

Base soil properties

Soil type
Soil density
Characteristic cohesion
Characteristic effective shear resistance angle

Medium dense well graded sand
 $\gamma_b = 20 \text{ kN/m}^3$
 $c'_{b,k} = 0 \text{ kN/m}^2$
 $\phi'_{b,k} = 30 \text{ deg}$

Using Coulomb theory

At rest pressure coefficient

$$K_0 = 1 - \sin(\phi'_{r,k}) = 0.5$$

Lateral pressure

Permanent surcharge pressure
Imposed surcharge pressure
Permanent toe surcharge pressure
Imposed toe surcharge pressure
Soil pressure at top of retained soil
Soil pressure at footing

$$p_{Q,\text{Surch.Press}} = K_0 \times p_{Q,\text{sur}} = 1 \text{ kN/m}^2$$

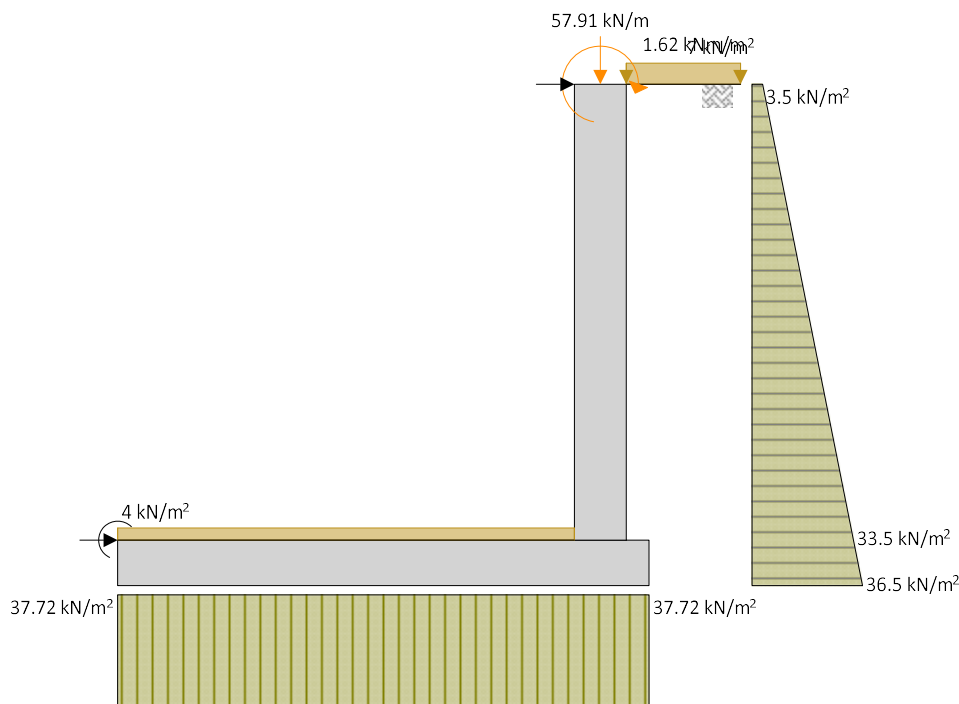
$$p_{G,\text{Surch.Press}} = K_0 \times p_{G,\text{sur}} = 2.5 \text{ kN/m}^2$$

$$p_{Q,\text{Surch.Toe.Press}} = K_0 \times p_{Q,\text{sur,toe}} = 1 \text{ kN/m}^2$$

$$p_{G,\text{Surch.Toe.Press}} = K_0 \times p_{G,\text{sur,toe}} = 1 \text{ kN/m}^2$$

$$p_{\text{Soil.Top}} = 0 \text{ kN/m}^2$$

$$p_{\text{Soil.Footing}} = K_0 \times (h_{\text{ret}} + h_{\text{footing}}) \times \gamma_{\text{mr}} = 33 \text{ kN/m}^2$$



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

Axial $R_{V,\text{StemBase}} = 83.41 \text{ kN/m}$
Shear $R_{H,\text{StemBase}} = 35.79 \text{ kN/m}$
Moment $R_{M,\text{StemBase}} = 0 \text{ kNm/m}$

Bearing pressure check

Vertical forces

Stem $F_{V,\text{Stem}} = A_{\text{stem}} \times \gamma_{\text{stem}} = 25.5 \text{ kN/m}$
Footing $F_{V,\text{Footing}} = A_{\text{footing}} \times \gamma_{\text{footing}} = 26.18 \text{ kN/m}$
Retained soil $F_{V,\text{Soil}} = h_{\text{moist}} \times l_{\text{heel}} \times \gamma_{\text{mr}} = 9 \text{ kN/m}$
Surcharge load $F_{V,\text{Surch}} = p_{G,\text{sur}} \times l_{\text{heel}} + p_{Q,\text{sur}} \times l_{\text{heel}} = 1.05 \text{ kN/m}$
Toe surcharge load $F_{V,\text{Surch.Toe}} = p_{G,\text{sur,toe}} \times l_{\text{toe}} + p_{Q,\text{sur,toe}} \times l_{\text{toe}} = 12 \text{ kN/m}$
Applied axial loads $F_{V,\text{Applied}} = P_G + P_Q = 57.91 \text{ kN/m}$

Total	$F_{V.Total} = F_{V.Stem} + F_{V.Footing} + F_{V.Soil} + F_{V.Surch} + F_{V.Surch.Toe} + F_{V.Applied} =$ 131.64 kN/m
Horizontal forces	
Stem	$F_{H.Stem} = R_{H.StemBase} =$ 35.79 kN/m
Retained soil	$F_{H.Soil} = (p_{Soil.WallBase} + 0.5 \times (p_{Soil.Footing} - p_{Soil.WallBase})) \times h_{footing} =$ 9.45 kN/m
Distance	$D_{FH.Soil} = h_{footing} \times (2 / 3 \times p_{Soil.WallBase} + p_{Soil.Footing} / 3) / (p_{Soil.WallBase} + p_{Soil.Footing}) =$ 0.148 m
Surcharge load	$F_{H.Surch} = (p_{G.Surch.Press} + p_{Q.Surch.Press}) \times h_{footing} =$ 1.05 kN/m
Prop	$F_{H.Prop} = F_{H.Stem} + F_{H.Soil} + F_{H.Surch} =$ 46.29 kN/m
Total	$F_{H.Total} = F_{H.Stem} + F_{H.Soil} + F_{H.Surch} - F_{H.Prop} =$ 0 kN/m
Moments	
Stem	$M_{Stem} = R_{V.StemBase} \times (l_{toe} + t_{stem} / 2) - R_{H.StemBase} \times h_{footing} - R_{M.StemBase} =$ 253.67 kNm/m
Footing	$M_{Footing} = F_{V.Footing} \times l_{total} / 2 =$ 45.68 kNm/m
Retained soil	$M_{Soil} = F_{V.Soil} \times (l_{toe} + t_{stem} + l_{heel} / 2) - F_{H.Soil} \times D_{FH.Soil} =$ 29.34 kNm/m
Surcharge load	$M_{Surch} = F_{V.Surch} \times (l_{toe} + t_{stem} + l_{heel} / 2) - F_{H.Surch} \times h_{footing} / 2 =$ 3.43 kNm/m
Toe surcharge load	$M_{SurchToe} = F_{V.Surch.Toe} \times l_{toe} / 2 =$ 18 kNm/m
Horizontal base prop	$M_{H.Prop} = F_{H.Prop} \times h_{footing} =$ 13.89 kNm/m
Moment resisted by base prop	$M_{Rest.Prop} = l_{total} / 2 \times F_{V.Total} - (M_{Stem} + M_{Footing} + M_{Soil} + M_{Surch} + M_{SurchToe} + M_{H.Prop}) =$ -134.3 kNm/m
Total	$M_{Total} = M_{Stem} + M_{Footing} + M_{Soil} + M_{Surch} + M_{SurchToe} + M_{H.Prop} + M_{Rest.Prop} =$ 229.7 kNm/m

Bearing pressure check

Distance to reaction	$\bar{x} = M_{Total} / F_{V.Total} =$ 1745 mm
Eccentricity of reaction	$e = \bar{x} - l_{total} / 2 =$ 0 mm
Loaded length of base	$l_{load} = l_{total} =$ 3490 mm
Bearing pressure at toe	$q_{toe} = F_{V.Total} / l_{total} \times (1 - 6 \times e / l_{total}) =$ 37.7 kN/m²
Bearing pressure at heel	$q_{heel} = F_{V.Total} / l_{total} \times (1 + 6 \times e / l_{total}) =$ 37.7 kN/m²
Factor of safety	$FoS_{bearing} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ 2.651

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

Teds calculation version 1.0.00

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

Concrete strength class	C28/35
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = 1.0
Characteristic compressive cylinder strength	$f_{ck} =$ 28 N/mm²
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 =$ 36 N/mm²
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} =$ 2.8 N/mm²
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} \times \text{AAF} =$ 32308 N/mm²
Compressive shortening strain - Table 3.1	$\epsilon_{c3} =$ 0.0018
Ultimate strain - Table 3.1	$\epsilon_{cu2} =$ 0.0035
Shortening strain - Table 3.1	$\epsilon_{cu3} =$ 0.0035
Effective compression zone height factor	$\lambda =$ 0.80
Effective strength factor	$\eta =$ 1.00

Coefficient k_1	$k_1 = 0.40$
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \epsilon_{cu2}) = 1.00$
Coefficient k_3	$k_3 = 0.40$
Coefficient k_4	$k_4 = 1.0 \times (0.6 + 0.0014 / \epsilon_{cu2}) = 1.00$
Partial factor for concrete - Table 2.1N	$\gamma_C = 1.50$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 15.9 \text{ N/mm}^2$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{ccw} = 1.00$
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C = 18.7 \text{ N/mm}^2$
Maximum aggregate size	$h_{agg} = 20 \text{ mm}$

Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = 500 \text{ N/mm}^2$
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = 1.15$
Design yield strength of reinforcement	$f_{yd} = 435 \text{ N/mm}^2$
Nominal cover to wall front reinforcement	$C_{nom.front} = 50 \text{ mm}$
Nominal cover to wall rear reinforcement	$C_{nom.rear} = 50 \text{ mm}$
Nominal cover to footing top reinforcement	$C_{nom.foot.top} = 35 \text{ mm}$
Nominal cover to footing bottom reinforcement	$C_{nom.foot.bot} = 75 \text{ mm}$

Wall vertical reinforcement details

Reinforcement provided	2 Layers of 10 mm ϕ bars at 100 mm c/c
Bar diameter	$\phi_{Stem.V} = 10 \text{ mm}$
Area of reinforcement provided	$A_{s.prov.Stem.V} = 2 \times \pi \times \phi_{Stem.V}^2 / (4 \times s_{Stem.V}) = 1571 \text{ mm}^2/\text{m}$
Maximum allowable spacing - cl.9.6.2(3)	$s_{max.Stem.V} = \min(3 \times t_{stem}, 400\text{mm}) = 400 \text{ mm}$
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - cl.9.6.2	$A_{s.min.Stem.V} = 0.002 \times t_{stem} = 680 \text{ mm}^2/\text{m}$

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	$\phi_{Stem.H} = 10 \text{ mm}$
Area of reinforcement provided	$A_{s.prov.Stem.H} = 2 \times \pi \times \phi_{Stem.H}^2 / (4 \times s_{Stem.H}) = 785 \text{ mm}^2/\text{m}$
Maximum allowable spacing - cl.9.6.3	$s_{max.Stem.H} = \min(3 \times t_{stem}, 400\text{mm}) = 400 \text{ mm}$
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - cl.9.6.3	$A_{s.min.Stem.H} = \max(0.25 \times A_{s.prov.Stem.V}, 0.001 \times t_{stem}) = 393 \text{ mm}^2/\text{m}$

PASS - Reinforcement provided exceeds minimum reinforcement required

Footing main reinforcement details

Reinforcement provided	16 mm ϕ bars at 100 mm c/c
Bar diameter	$\phi_{Foot.long} = 16 \text{ mm}$
Area of reinforcement provided	$A_{s.prov.Foot.long} = \pi \times \phi_{Foot.long}^2 / (4 \times s_{Foot.long}) = 2011 \text{ mm}^2/\text{m}$
Maximum allowable spacing - 9.3.1.1(3)	$s_{max.Foot.long} = \min(3 \times h_{footing}, 400\text{mm}) = 400 \text{ mm}$
	PASS - Maximum allowable spacing exceeds reinforcement spacing
Min.area required per metre length - exp.9.1N	$A_{s.min.Foot.long} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times \max(d_{foot.top}, d_{foot.bot}) = 370 \text{ mm}^2/\text{m}$

PASS - Reinforcement provided exceeds minimum reinforcement required

Footing distribution reinforcement details

Reinforcement provided	10 mm ϕ bars at 100 mm c/c
Bar diameter	$\phi_{Foot.trans} = 10 \text{ mm}$

Area of reinforcement provided

Maximum allowable spacing - 9.3.1.1(3)

Min.area required per metre length - exp.9.1N

$$A_{s,prov.Foot.tran} = \pi \times \phi_{Foot.tran}^2 / (4 \times s_{Foot.tran}) = 785 \text{ mm}^2/\text{m}$$

$$s_{max.Foot.tran} = \min(3.5 \times h_{footing}, 450\text{mm}) = 450 \text{ mm}$$

PASS - Maximum allowable spacing exceeds reinforcement spacing

$$A_{s,min.Foot.tran} = 0.2 \times A_{s,prov.Foot.long} = 402 \text{ mm}^2/\text{m}$$

PASS - Reinforcement provided exceeds minimum reinforcement required

DA1 6.10 load combinations (ULSD)

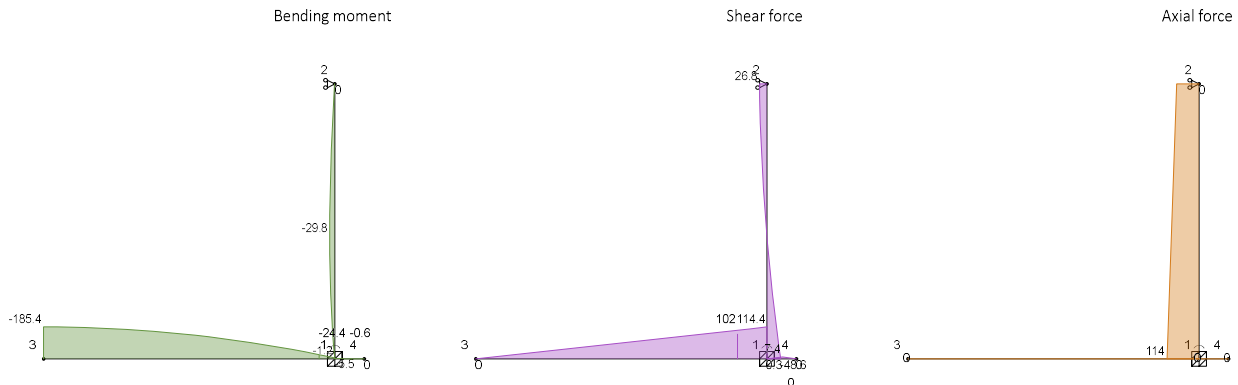
$$1.35G + 1.5Q + 1.5Q_r (0.915)$$

$$1.00G + 1.5W (0.585)$$

$$1.35G + 1.5Q + 1.5Q_r + \psi_w \times 1.5W (0.915)$$

$$1.35G + 1.5Q + \psi_w \times 1.5W + \psi_s \times 1.5S (0.915)$$

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



Check pure axial capacity

Ultimate axial force

Strain with uniform compression

Stress in reinforcement

Pure axial design capacity

$$N_{Ed,max} = 114.03 \text{ kN/m}$$

$$\epsilon_0 = \epsilon_{c3} = 0.00175$$

$$\sigma_0 = \min(\epsilon_0 \times E_s, f_{yd}) = 350 \text{ N/mm}^2$$

$$N_{Rd0} = A_{s,prov.Stem.V} \times \sigma_0 + (t_{stem} - A_{s,prov.Stem.V}) \times f_{cd} = 5919.52 \text{ kN/m}$$

$$N_{Ed,max} / N_{Rd0} = 0.019$$

PASS - Design axial capacity exceeds ultimate axial force

Slenderness limit - (cl.5.8.3.1)

Effective length factor (Fig. 5.7)

Unsupported length

Effective length

Radius of gyration

Slenderness ratio (5.8.3.2(1))

$$f = 1$$

$$l_u = 3000 \text{ mm}$$

$$l_0 = f \times l_u = 3000 \text{ mm}$$

$$i = t_{stem} / \sqrt{12} = 9.8 \text{ cm}$$

$$\lambda_{slender} = l_0 / i = 30.6$$

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

Smaller factored end moment

Larger factored end moment

Ecc. due to geometric imperfections

Minimum end moment

Maximum end moment

$$M_{1,end} = -2.4 \text{ kNm/m}$$

$$M_{2,end} = 0.0 \text{ kNm/m}$$

$$e_i = l_0 / 400 = 7.5 \text{ mm}$$

$$M_{01} = \min(\text{abs}(M_{1,end}), \text{abs}(M_{2,end})) + e_i \times N_{Ed,max} = 0.9 \text{ kNm/m}$$

$$M_{02} = \max(\text{abs}(M_{1,end}), \text{abs}(M_{2,end})) + e_i \times N_{Ed,max} = 3.3 \text{ kNm/m}$$

Slenderness limit for buckling (cl. 5.8.3.1)

Area of concrete

Factor A

Mechanical reinforcement ratio

$$A_c = t_{stem} - A_{s,prov.Stem.V} = 338429 \text{ mm}^2/\text{m}$$

$$A = 0.7$$

$$\omega = A_{s,prov.Stem.V} \times f_{yd} / (A_c \times f_{cd}) = 0.127$$

Factor B	$B = \sqrt{1 + 2 \times \omega} = 1.120$
Moment ratio	$r_m = M_{01} / M_{02} = 0.261$
Factor C	$C = 1.7 - r_m = 1.439$
Relative normal force	$n = N_{Ed,max} / (A_c \times f_{cd}) = 0.021$
Slenderness limit	$\lambda_{lim} = 20 \times A \times B \times C / \sqrt{n} = 154.9$
	$\lambda_{slender} / \lambda_{lim} = 0.197$

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

Wall design moment (negative)

Stem moment	$M_{stem} = -29.8$ kNm/m
Wall design moment (conservative)	$M_{Ed} = \min(M_{stem} - e_i \times N_{Ed,max}, -N_{Ed} \times \max(t_{stem} / 30, 20\text{mm})) = -30.7$ kNm/m

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

Design axial force	$N_{Ed} = 114$ kN/m
Design bending moment	$M_{Ed} = -30.7$ kNm/m (front face in tension)
Position of neutral axis (by iteration)	$z = 45.2$ mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3))	$F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0\text{mm}), t_{stem}) = 574.3$ kN/m
Moment of resistance	$M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2) = 87.2$ kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement	$A_s = A_{s,prov.Stem.V} / 2 = 785.4$ mm ² /m
Depth to tension face reinforcement	$d_t = t_{stem} - C_{nom.front} - \phi_{Stem.H} - \phi_{Stem.V} / 2 = 275$ mm
Strain in tension face reinforcement	$\epsilon_s = \epsilon_{cu3} \times (1 - d_t / z) = -0.01777$
Stress in tension face reinforcement	$\sigma = \max(\epsilon_s \times E_s, -f_{yd}) = -434.78$ N/mm ²
Force in tension face reinforcement	$F_{st} = A_s \times \sigma = -341.48$ kN/m
Tension face reinf. moment of resistance	$M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t) = 35.86$ kNm/m
Area of compression face bars	$A'_s = A_{s,prov.Stem.V} / 2 = 785.4$ mm ² /m
Depth to compression face reinforcement	$d' = C_{nom.rear} + \phi_{Stem.V} / 2 = 55$ mm
Strain in compression face reinforcement	$\epsilon'_s = \epsilon_{cu3} \times (1 - d' / z) = -0.00075$
Stress in compression face reinforcement	$\sigma' = \max(\epsilon'_s \times E_s, -f_{yd}) = -150.87$ N/mm ²
Force in compression face reinforcement	$F_{sc} = A'_s \times \sigma' = -118.49$ kN/m
Comp. face reinf. moment of resistance	$M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d') = -13.63$ kNm/m

Total resistance of section

Resultant concrete/steel force	$F = F_c + F_{st} + F_{sc} = 114.38$ kN/m
	PASS - F is within half of one percent of N_{Ed}
Moment of resistance	$M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc} = 109.47$ kNm/m
	$\text{abs}(M_{Ed}) / M_{Rd} = 0.28$
	PASS - Design moment capacity exceeds ultimate bending moment

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

Design axial force	$N_{Ed} = 79.6$ kN/m
Design bending moment	$M_{Ed} = -30.7$ kNm/m (front face in tension)
Position of neutral axis (by iteration)	$z = 44$ mm

Moment of resistance of concrete

Concrete compression force (3.1.7(3))	$F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0\text{mm}), t_{stem}) = 558.5$ kN/m
Moment of resistance	$M_{Rdc} = F_c \times (t_{stem} / 2 - \min(\lambda_{sb} \times z, t_{stem}) / 2) = 85.1$ kNm/m

Moment of resistance of reinforcement

Area of tension face reinforcement	$A_s = A_{s,prov.Stem.V} / 2 = 785.4$ mm ² /m
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Depth to tension face reinforcement	$d_t = t_{stem} - C_{nom.front} - \phi_{stem.H} - \phi_{stem.V} / 2 = 275 \text{ mm}$
Strain in tension face reinforcement	$\epsilon_s = \epsilon_{cu3} \times (1 - d_t / z) = -0.01837$
Stress in tension face reinforcement	$\sigma = \max(\epsilon_s \times E_s, -f_{yd}) = -434.78 \text{ N/mm}^2$
Force in tension face reinforcement	$F_{st} = A_s \times \sigma = -341.48 \text{ kN/m}$
Tension face reinf. moment of resistance	$M_{Rdst} = F_{st} \times (t_{stem} / 2 - d_t) = 35.86 \text{ kNm/m}$
Area of compression face bars	$A'_s = A_{s.prov.stem.V} / 2 = 785.4 \text{ mm}^2/\text{m}$
Depth to compression face reinforcement	$d' = C_{nom.rear} + \phi_{stem.V} / 2 = 55 \text{ mm}$
Strain in compression face reinforcement	$\epsilon'_s = \epsilon_{cu3} \times (1 - d' / z) = -0.00087$
Stress in compression face reinforcement	$\sigma' = \max(\epsilon'_s \times E_s, -f_{yd}) = -174.94 \text{ N/mm}^2$
Force in compression face reinforcement	$F_{sc} = A'_s \times \sigma' = -137.4 \text{ kN/m}$
Comp. face reinf. moment of resistance	$M_{Rdsc} = F_{sc} \times (t_{stem} / 2 - d') = -15.8 \text{ kNm/m}$
Total resistance of section	
Resultant concrete/steel force	$F = F_c + F_{st} + F_{sc} = 79.67 \text{ kN/m}$ PASS - F is within half of one percent of N_{Ed}
Moment of resistance	$M_{Rd} = M_{Rdc} + M_{Rdst} + M_{Rdsc} = 105.18 \text{ kNm/m}$ $\text{abs}(M_{Ed}) / M_{Rd} = 0.292$ PASS - Design moment capacity exceeds ultimate bending moment
Check shear capacity of wall (cl.6.2.2)	
Design shear force	$V = 48.62 \text{ kN/m}$
Depth to tension steel	$d_v = \min(t_{stem} - C_{nom.front} - \phi_{stem.H} - \phi_{stem.V} / 2, t_{stem} - C_{nom.rear} - \phi_{stem.V} / 2) = 275 \text{ mm}$
	$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
	$k_v = \min(1 + \sqrt{(200\text{mm} / d_v)}, 2) = 1.853$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{s.prov.stem.V} / (2 \times d_v), 0.02) = 0.003$
	$V_{min} = 0.035 \text{ N/mm}^2 \times k_v^{3/2} \times (f_{ck} / 1\text{N/mm}^2)^{0.5} = 0.467 \text{ N/mm}^2$
	$k_{1,v} = 0.15$
Comp stress in concrete from axial loading	$\sigma_{cp} = N_{Ed,max} / (t_{stem} - A_{s.prov.stem.V}) = 0.337 \text{ N/mm}^2$
Design shear resistance (exp.6.2a & 6.2b)	$V_{Rd,c} = (\max(C_{Rd,c} \times k_v \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, V_{min}) + k_{1,v} \times \sigma_{cp}) \times d_v = 142.3 \text{ kN/m}$ $V / V_{Rd,c} = 0.342$ PASS - Design concrete shear capacity exceeds ultimate shear force
Check footing in flexure - Section 6.1	
Design bending mnt, 3000 mm from wall face	$M_{Ed.foot} = -185.4 \text{ kNm/m}$
Depth to tension reinforcement	$d_{foot.top} = h_{footing} - C_{nom.foot.top} - \phi_{Foot.long} / 2 = 257 \text{ mm}$
	$K_{foot} = \text{abs}(M_{Ed.foot}) / (d_{foot.top}^2 \times f_{ck}) = 0.100$
	$K'_{foot} = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$ $K' > K$ - No compression reinforcement is required
Lever arm	$Z_{foot} = \min(0.5 + 0.5 \times (1 - 2 \times K_{foot} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d_{foot.top} = 232 \text{ mm}$
Depth of neutral axis	$x_{foot} = 2.5 \times (d_{foot.top} - Z_{foot}) = 63 \text{ mm}$
Area of tension reinforcement required	$A_{s.des} = \text{abs}(M_{Ed.foot}) / (f_{yd} \times Z_{foot}) = 1839 \text{ mm}^2/\text{m}$
Minimum area of reinforcement - exp.9.1N	$A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{foot.top} = 370 \text{ mm}^2/\text{m}$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s.max} = 0.04 \times h_{footing} = 12000 \text{ mm}^2/\text{m}$
Area of tension reinforcement provided	$A_{s.prov.Foot.long} = 2011 \text{ mm}^2/\text{m}$ $\text{abs}(A_{s.req}) / A_{s.prov.Foot.long} = 0.915$ PASS - Area of reinforcement provided is greater than area of reinforcement required

Check shear capacity of footing (cl.6.2.2)

Design shear force
 Depth to tension steel

Main reinforcement ratio

Design shear resistance (exp.6.2a & 6.2b)

$$V_{foot} = 101.95 \text{ kN/m}$$

$$d_{v,foot} = d_{foot,top} = 257 \text{ mm}$$

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

$$k_{v,foot} = \min(1 + \sqrt{(200\text{mm} / d_{v,foot})}, 2) = 1.882$$

$$\rho_{l,foot} = \min(A_{s,prov.Foot.long} / d_{v,foot}, 0.02) = 0.008$$

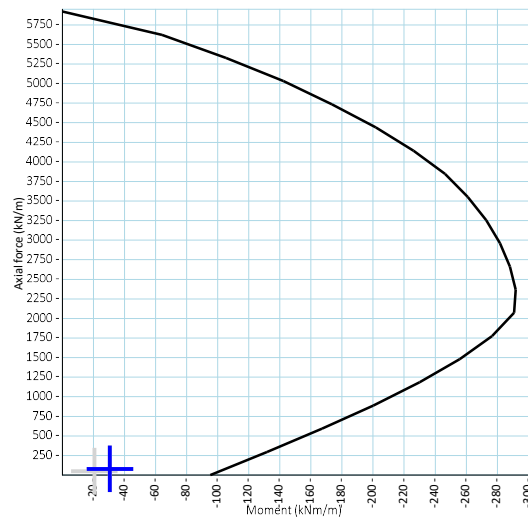
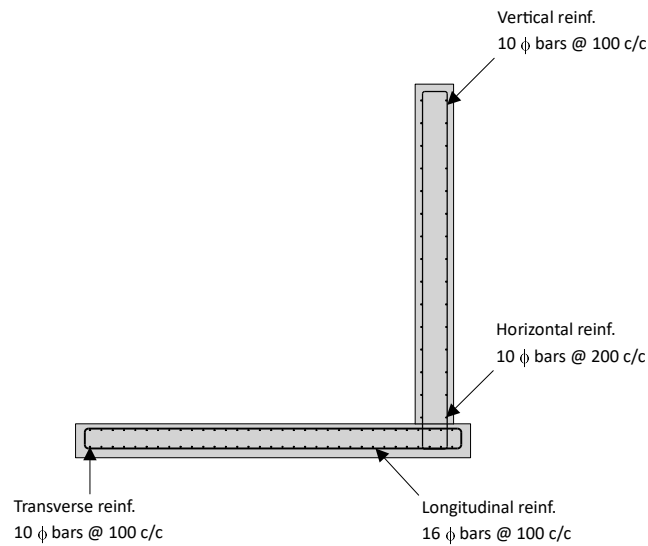
$$V_{min,foot} = 0.035 \text{ N/mm}^2 \times k_{v,foot}^{3/2} \times (f_{ck} / 1\text{N/mm}^2)^{0.5} = 0.478 \text{ N/mm}^2$$

$$V_{Rd,c,foot} = \max(C_{Rd,c} \times k_{v,foot} \times (100 \text{ N}^2/\text{mm}^4 \times \rho_{l,foot} \times f_{ck})^{1/3}, V_{min,foot}) \times d_{v,foot}$$

$$= 162.4 \text{ kN/m}$$

$$V_{foot} / V_{Rd,c,foot} = 0.628$$

PASS - Design concrete shear capacity exceeds ultimate shear force



Note: Crit. combination highlighted: 1.35G + 1.5Q + 1.5Gr
 N_d at top of storey
Interaction diagram - Negative moments

8 Basement Impact Assessment

8.1 Impact on the adjacent building structures

Based on the assessment, the closest neighbouring building at No. 13 Woodside Rd is situated approximately 5.4m from the perimeter of the proposed basement. Given the relatively shallow excavation depth of 3.0m to the Finished Floor Level (FFL), the theoretical zone of influence from the excavation does not intersect with the foundations of the neighbouring property. Therefore, the proposed works will not have any negative structural impact on the adjacent building.

However, the basement retaining wall is set back approximately 1.1m from the boundary line with No. 13. To ensure the highest level of safety and to strictly prevent any ground disturbance or relaxation on the neighbouring plot, the excavation and construction of the basement retaining walls will be conducted in a controlled, sequential manner. The use of temporary supports and propping during this sequential excavation will further ensure ground stability and the overall safety of the works.

9. Site Monitoring Requirements

To ensure the structural integrity of the host building and to verify that adjacent properties remain entirely unaffected during the works, a comprehensive monitoring regime must be implemented by the principal contractor:

- **Pre-Condition Survey:** A thorough dilapidation/condition survey of the host property's rear elevation and the boundary line with No. 13 Woodside Rd must be conducted and documented prior to any excavation commencing.
- **Monitoring Targets:** Reflective monitoring targets or survey studs must be installed on the host building's existing rear wall above the underpinning zone, and at strategic points along the boundary.
- **Monitoring Frequency:** Optical levelling and verticality checks must be taken before excavation begins to establish a baseline. Monitoring must then be recorded on a weekly basis during the active excavation and sequential underpinning phases. Once the basement slab and structural walls are fully cured, monitoring should continue on a monthly basis for 3 months.
- **Action Plan and Trigger Levels:** The contractor must establish acceptable movement trigger levels (e.g., Amber warning level at 3mm, Red stop level at 5mm). Should monitoring indicate movement approaching or exceeding the trigger levels, works must be halted immediately, temporary propping checked and reinforced, and the Structural Engineer must be consulted before works can resume.

Contact

- In case of any further clarification, please contact:

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