eng17

7 Rosecroft Avenue
London
NW3 7QA

Basement Impact Assessment
for
William Tozer Associates

REVISIONS
Revisions are highlighted in the text

Rev	Date	By	Notes

Job No: ENG/210111
February 2021
Prepared by: Paul Cullen BA, BAI (Hons) CEng MIEI

INDEX

1. INTRODUCTION

This report has been prepared by eng17 Ltd, consulting civil & structural engineers, to accompany the planning submission by William Tozer Associates and is intended to satisfy the requirements of Camden Council Planning Department.

The purpose of this report is to ascertain the potential impact that the deepened area of the proposed garden structure may have on ground stability, the hydrogeology and the hydrology in the vicinity of the site. The assessments were carried out in general accordance with the Borough of Camden guidelines as requested by the client.

The following report outlines the structural methodology for the proposed garden structure with excavation, floor and timber frame superstructure construction. The proposals excavate the ground level adjacent to the rear boundary to increase the available ceiling level in the building, to accommodate a climbing / bouldering wall.

This report should be read in conjunction with the attached arboricultural report by Parsons Tree Care and the structural drawings.

2. SITE & DEVELOPMENT APPRAISAL

2.1 Existing Site and Context

Rosecroft Avenue is a residential street in the Hampstead area of London as shown in Figure 1. The property is not listed on the English Heritage website but the site does lie within the Camden Conservation Area.

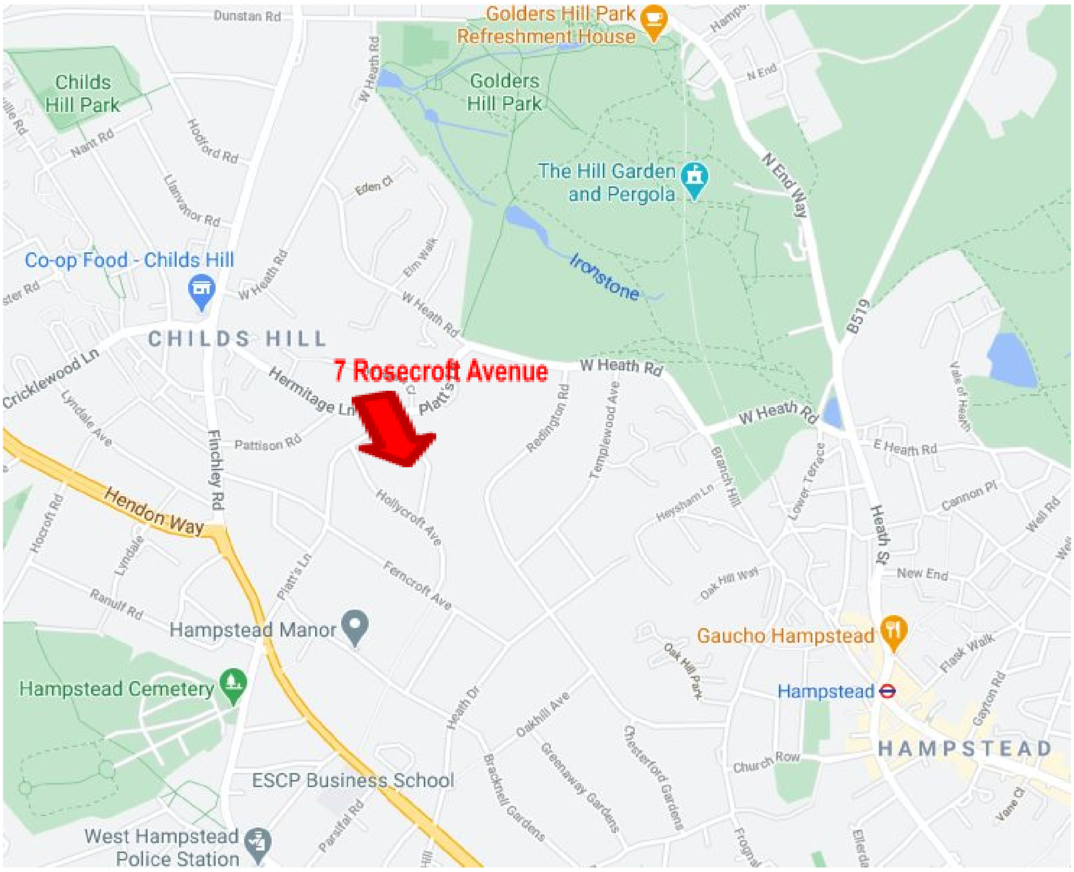


Figure 1a: Location Map

The existing structure is a single-storey timber shed within the rear garden of the property.

Figure 1b: Satellite view

The historic bomb map shows that the site did not suffer damage caused by air raid strikes during the Second World War.

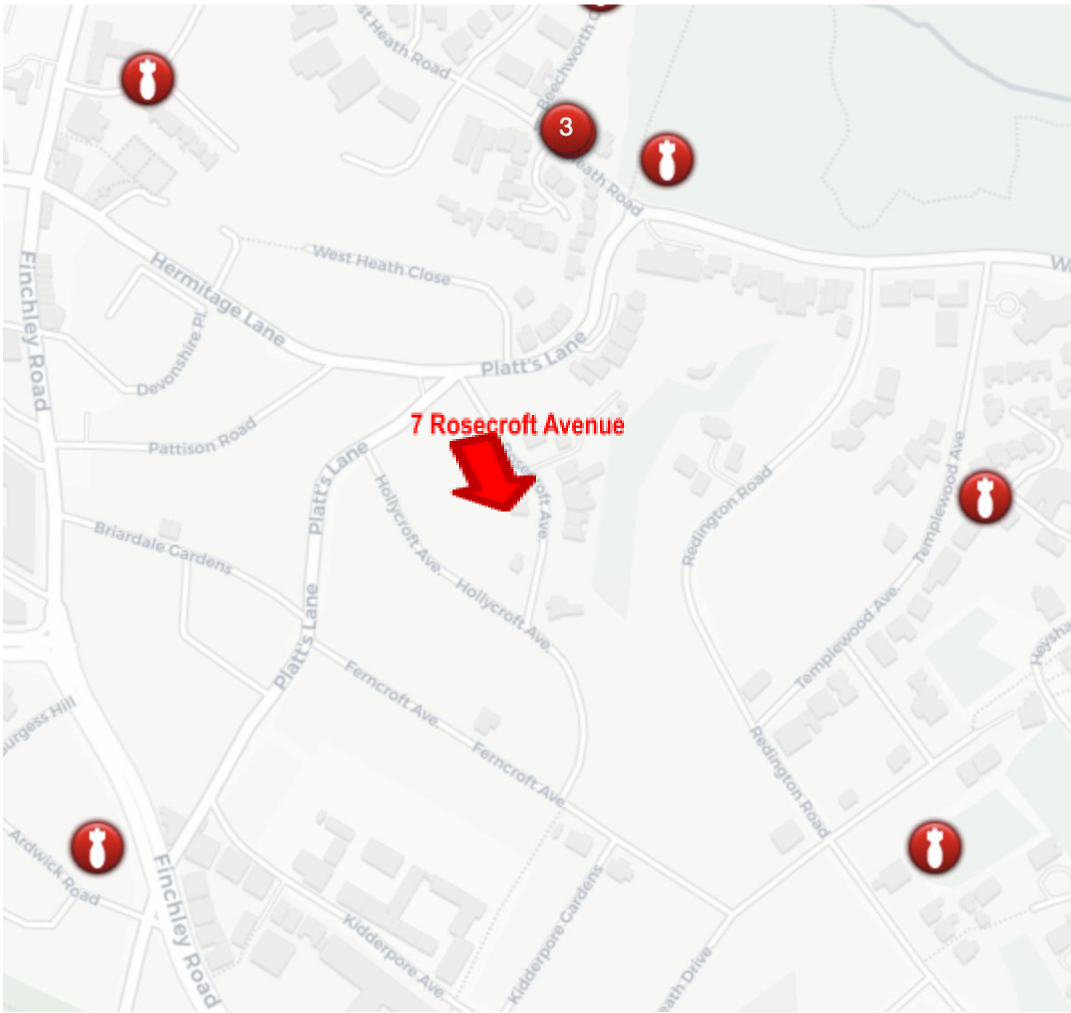


Figure 2: Bomb Map

A map from *The Lost Rivers of London* by Nicholas Barton shows headwater springs to tributaries of two of London's subterranean watercourses approximately 150m from the site; Brent Reservoir to the north and the Westbourne to the south, as indicated in Figure 3. However, the site is not directly affected and there are no known flooding issues with the subject property or the immediate environs.

Figure 3: Lost Rivers of London

The Tube Map in Figure 4 shows no tube lines passing within the vicinity of the site. The nearest tube line is the Northern Line running through Golders Hill Park, about 500m east of the site.

Figure 4 Tube Map

The British Geological Survey website has a map that illustrates the general soil profile in this area. The site is located on the boundary between London Clay Formation and Bagshot Formation bedrock. The superficial geology is London Clay Formation. This has been explored in more detail via trial pits with the assistance of Parsons Tree Care and is discussed in their report in Appendix C. Two boreholes have been carried out in close proximity to the site, both are included in Appendix D. One borehole is approximately 250m west of the site and the other is approximately 800m east of the site.

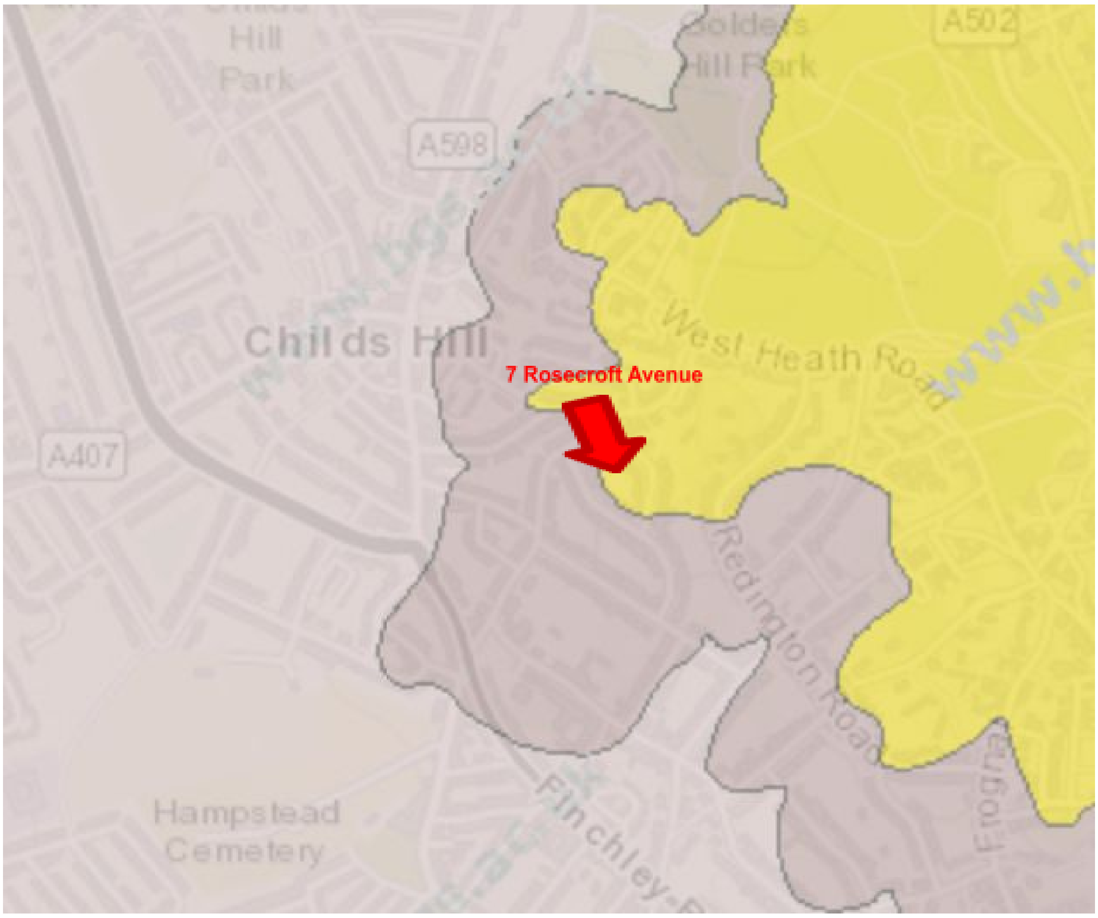


Figure 5: Extract from British Geological Survey Map (website)

2.2 Proposed Development

The proposed development involves deepening the rear garden area to provide a climbing wall and office space. The lowest point of the deepened area will be approximately 1.5m below the existing rear garden level.

This report will only address the issues arising from the deepening of the rear garden area.

2.3 Existing Ground Conditions

A site-specific site investigation has been carried out. The trial holes to a depth of 0.7m bgl indicate that the existing ground is brown sandy clay and pebbles. The existing ground conditions where the excavation is proposed are illustrated in Trial Pit 2 within the arboricultural report, appended to this report. Fibrous root system is present from previously removed vegetations such as ivy, however no structural roots were uncovered. Lime trees are growing beyond the rear boundary, in the garden of 15 Hollycroft Avenue, however, these roots haven't grown under the presumed 400mm wall foundation into the garden of 7 Rosecroft Avenue.

2.4 Site Hydrology

The proposed building will be deepened only in the climbing wall area. The construction of the deepened area is unlikely to have a detrimental effect on the hydrogeology of the site. No ground water was encountered in any of the trial holes at depths of up to 1.0m below ground level.

There is a combined sewer indicated on the attached Thames Water Sewer Map running along the centre of Rosecroft Avenue. There is a 125mm HPPE water supply main also running along Rosecroft Avenue, shown on the attached Thames Water Map. Their positions are outside the site of the proposed excavation.

Figure 6a: Thames Water Asset Location Sewer Map

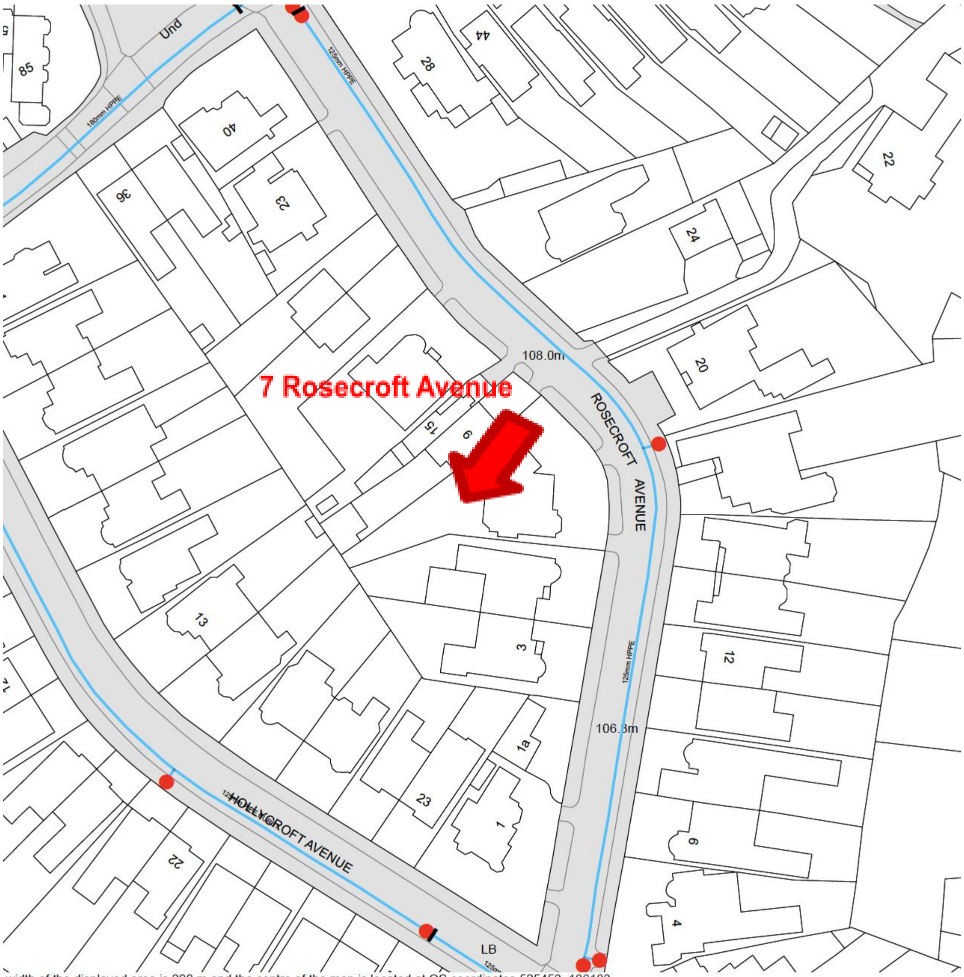


Figure 6b: Thames Water Asset Location Water Map

The site is outside the fluvial/tidal flood zone of the River Thames and its tributaries. The long term flood risk is deemed very low due to both surface water and rivers and the sea flooding, as per the flood map extracted from the planning website. The flood map for planning website indicates a low probability of flooding and that no flood risk assessment is required.

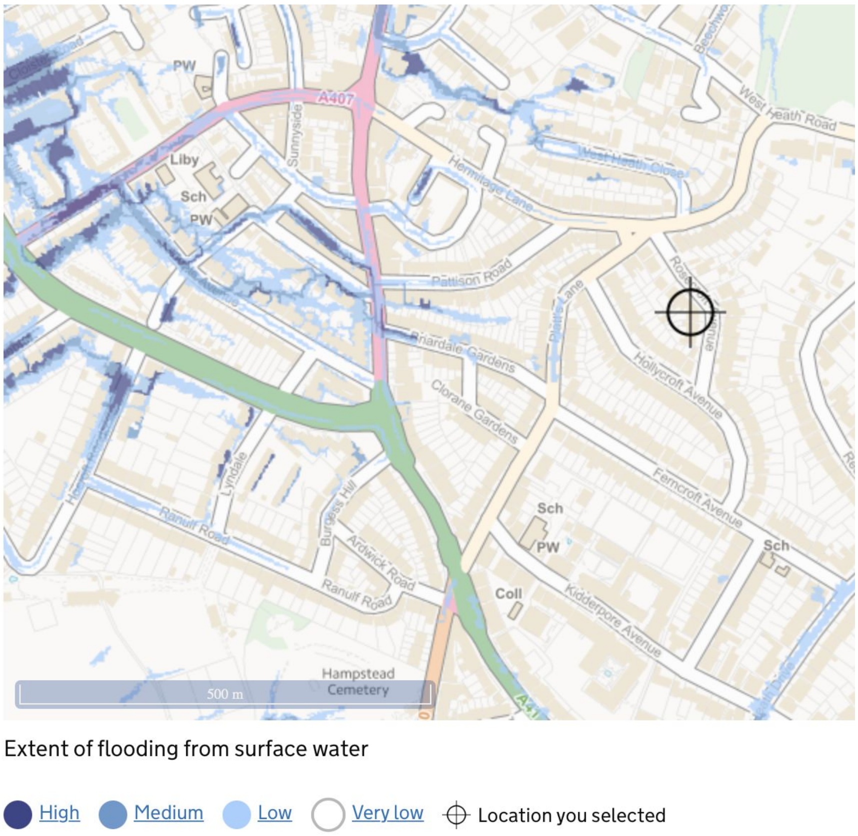
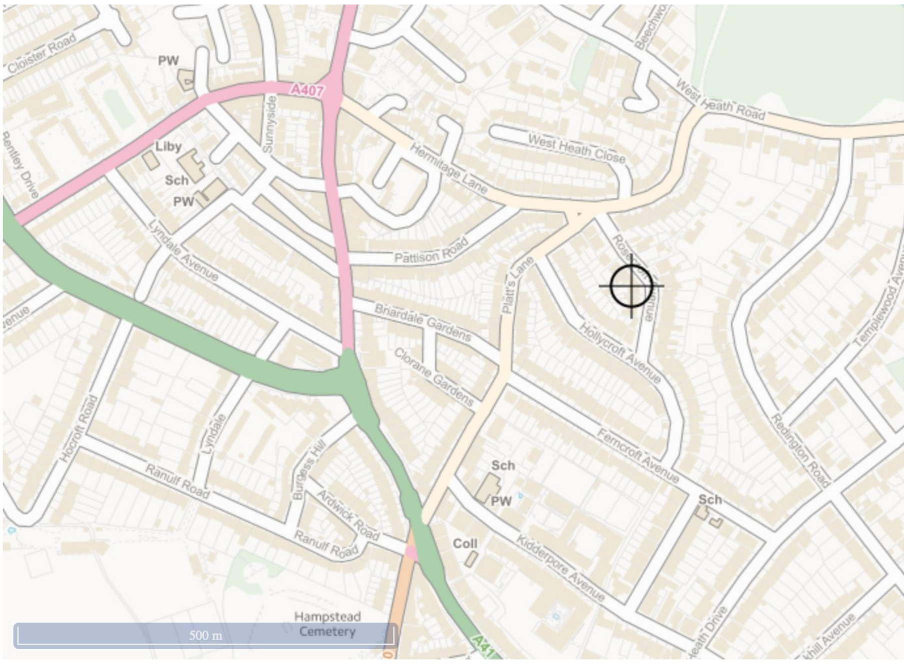


Figure 7a: Risk of flooding from surface water



Extent of flooding from rivers or the sea

High Medium Low Very low Location you selected

Figure 7b: Risk of flooding from rivers or the sea

3 PROPOSED STRUCTURE

3.1 Foundations in General

There is approximately 0.5m of soil and stones over 2.5m of brown, sandy clay, over 1.5m of very stiff London Clay. The basement slab and walls will be founded within the London Clay, which will provide a suitable bearing stratum for the support of the proposed garden structure. The arboricultural report proposes pad foundations to avoid damaging the existing tree roots. However, to mitigate the effects of heave, piled foundations are proposed. The design and sequence of temporary works for the basement will be driven by the following factors:

- The stability of the top soil and underlying clay
- Maintaining the stability of adjacent out buildings (9 Rosecroft Avenue) and landscaping
- Accommodating the existing public sewer serving 7 Rosecroft Avenue

3.2 Deepened Area Construction

The formation level for the deepened area will be within the London Clay formation.

The new structure will comprise reinforced concrete retaining walls and floor slab in the deepened area and a suspended RC slab at the higher office level. The entire floorplate will be piled. The intention is to use screw piles to reduce the carbon footprint of concrete piles and protect the tree roots. The office floor load will be fully supported by the piles, adding no surcharge to the existing ground at formation level, thereby protecting the existing tree roots.

The excavations are predicted to remain dry during construction but an allowance for minor pumping will be made in the event that any pockets of groundwater are encountered.

3.3 Party Wall Matters

The proposed development falls within the scope of the Party Wall etc Act 1996. Procedures under the Act and will be dealt with in full by the Employer's Party Wall Surveyor. The Party Wall Surveyor will prepare and serve necessary Notices under the provisions of the Act and agree Party Wall Awards in the event of disputes. The Contractor will be required to provide the Party Wall Surveyor with appropriate drawings, method statements and other relevant information covering the works that are notifiable under the Act. The resolution of matter under the Act and provision of the Party Wall Awards will protect the interests of all owners.

4 CONSTRUCTION SEQUENCE/METHODOLOGY

All of the works, particularly the sub-structure are to be carried out in a manner that minimises any noise and vibration that may affect the neighbouring properties. The engineer will make regular site visits during the construction.

Outline construction sequence and temporary works assumed in the design as described below will be superseded by the Contractor’s proposals. The Contractor will be required to submit full proposals, method statements and calculations to the engineer for review prior to the start of any works on site.

The Contractor will be responsible for the design, erection and maintenance of all temporary works in accordance with all relevant British Standards. The Contractor is to provide adequate temporary works and supervision to ensure that the stability of the existing structure, excavations and surrounding structures are maintained at all times.

4.1 Assumed Sequence of Construction

The anticipated constructed sequence is shown on the sketches in Appendix A. The main stages of the work area are as outlined below:

The borehole logs taken from neighbouring properties confirm that the site-specific ground conditions comprise 0.5m of soil and stones over 2.5m of brown, sandy clay, over London Clay. See Appendix D.

The borehole sunk at 378 Finchley Road (No. TQ28NE421) encountered ground water at 1.20m below ground level. Any water will be closely monitored as London Clay can easily host groundwater (rising in level depending on the period of the year) or at least perched water.

The existing boundary wall foundations comprise concrete footings as per the attached recorded trial pits in Appendix C.

The proposed building is approximately 30m from the nearest buildings on Rosecroft Avenue and Hollycroft Avenue. Therefore the proposed excavations will not adversely affect the existing structures.

The new structure will be designed for a maximum ground bearing pressure of 100Kn/m2.
The soil conditions are considered suitable for safe excavation and underpinning work.

Construction Sequence

1. Hoarding will be erected around the site to protect the work area from the residents of 7 Rosecroft Avenue.

2. Demolition of existing shed to be carried out by contractor in a safe manor, following RAMS approved by the engineer.

3. Access to the dig area will be via the passageway to the side of 7 Rosecroft Avenue. Spoil will be removed via wheelbarrows through the passageway to a skip located at the front.

4. The existing perimeter walls will be underpinned in bays not exceeding 1000mm in accordance with eng17's specification, as shown on the attached sequence of works drawings.

5. The excavation and underpinning will be temporarily horizontally propped across the site as shown on the drawings. If necessary, localized pumping will be used to remove ground water during excavations and underpinning, although it must be noted that groundwater was not encountered in the trial holes during site investigations.

6. When the central soil mound (berm) is to be excavated and removed a new line of temporary horizontal propping will be provided laterally across the full width of the site using RMD props or similar.

7. The new concrete basement slab will be cast on completion of the underpinning works. Once this slab is cured the lower level temporary props can be removed.

8. The timber frame walls and roof can be constructed.

Some of the issues that affect the sequence of works on this project are:

- Forming sensible access onto the site to minimise disruption to the neighbouring residents
- Heave issues from the London Clay
- Providing a safe working environment
- Sustainability

Generally, the scale of the works is such that relatively straightforward methods can be used for the basement construction.

4.2 Noise, Dust and Vibration

The Contractor shall undertake the works in such a way as to minimise noise, dust and vibration when working close to adjacent buildings in order to protect the amenities of the nearby occupiers. The demolition of existing structure shall be carried out, where possible, to minimise vibration to the adjacent properties and associated construction noise. All demolitions and excavations will be undertaken in a carefully controlled sequence, taking into account the requirement to minimise vibration and noise.

4.3 Impact on Local Environment

The new lower ground floor and basement structure will be designed and built with detailed consideration to the impact of the local environment, including screw piles where possible, and entirely timber frame superstructure. The external landscaping will be designed sufficiently to allow there to be little change to surface water flows. Similarly we would expect there to be little effect on the local flora. The trees on the property will be protected by suspended flooring and piles.

4.4 Impact on Adjacent Structures and Services

Impact on Adjacent Structure 9 Rosecroft Avenue outbuilding

The 9 Rosecroft Avenue outbuilding is a timber clad shed with corrugated iron roof, to the north of the subject building.

The proposed underpinning/retaining wall details for 7 Rosecroft Avenue will be designed to support the vertical and surcharge loadings generated from the building at 9 Rosecroft Avenue. The excavation and construction for the new basement will be carried out in a safe sequence with lateral props provided to maintain lateral stability of the adjacent ground and structures. A method statement for the works will be completed by the Contractor for party wall purposes. We do not envisage any significant damage will develop as a consequence of the proposed work.

Impact on Rear Garden of 15 Hollycroft Avenue

The garden at 7 Rosecroft Avenue slopes down to the rear and steps c. 900mm further down to the adjoining garden at the rear boundary on Hollycroft Avenue. The proposed excavation reduces the ground level by approximately 1500mm so, c. 600mm below the Hollycroft rear garden level.

Impact on the Existing Sewers

The existing Thames Water sewer running along Rosecroft Avenue will remain in operation at all times and will not be affected by the works. Similarly, the mains water supply located in the roadway will not be affected.

Impact on the Public Footpath & Highway

The works will not unduly impact the pavement and highway. Construction activities will be confined to within the site, as there is ample space within the rear garden for off-street site set-up.

Impact on Buried Services on the Site

We do not expect any impact on services as they enter the site. However, all reasonable precautions will be carried out using CAT scanning, initial hand digging, and reference to utility maps.

5 BIA Specifics

The following section covers specifically the points required by Camden Council BIA guidance. The information is primarily contained elsewhere within this report and is referenced here for convenience.

- *Whether the geology is capable of supporting the loads and construction techniques to be imposed.*

  The calculations below indicate that the ground bearing pressures resulting from the work are well within the limits recommended by the intrusive ground investigation.

- *The impact of the subterranean development, and associated construction and temporary works, on the structural integrity and natural ability for movement of existing and surrounding structures, utilities, infrastructure and man-made cavities, such as tunnels.*

  The anticipated ground movement and associated damage is predicted to be of 'very slight' or locally 'slight' in accordance with the Burland Category of CIRIA C580.

- *Whether the development will initiate slope instability which may threaten its neighbours.*

  Local topography means that slope stability is not an issue for this site.

- *The impact of the subterranean development on drainage, sewage, surface water on ground water, flows and levels.*

  There will be a very slight increase in the in-foul discharge volumes. Groundwater issues are covered in section 2.4, which confirms no impact.

- *How many geological, hydrological and structural concerns have been satisfactorily addressed?*

  This is covered in section 2, 3 and 4 as well as the calculations in Appendix B.

- *The engineering details of the scheme, including proposals for the excavation and construction.*

  Indicative preliminary sketches of the structure and proposed sequence of works are shown in Section Appendix A.

- *The impact of the proposed subterranean development on the structural stability of the existing and adjoining buildings, especially listed buildings.*

  See section 4.4.

- *The impact of the proposed subterranean development on existing and proposed trees.*

  An arboricultural report is included in Appendix C. A trench was excavated along the rear boundary with Hollycroft Avenue and no significant roots were found in this area where the excavation is proposed. No adverse impact is expected if permanent and temporary works are followed as per engineer's drawings. The floor slabs are designed as two way spanning, onto pile capping beams, so the load will be transferred through the piles and won't adversely affect the tree roots.

- *The sequence for the temporary works, which mitigates the effects on neighbours, and the details and design of the preferred method of Temporary Works.*

  An indicative preliminary works scheme and sequence of works has been indicated in the drawings in Appendix A to be read with the anticipated construction sequence given in section 4 above, and calculations in Appendix B. The final design will be the responsibility of the Contractor but will be subject to review, comment and approval by the Structural Engineer to ensure its adequacy and suitability for the works proposed.

6 SUSTAINABILITY

The proposed development has been designed in line with the iStructE One Planet Living initiative for sustainable design:

Zero waste in materials: Bolted connections between timber beams used rather than welds to encourage zero waste when the building reaches the end of its life.
Zero carbon energy: Screw piles preferable to deep concrete trench foundations. Superstructure in timber rather than steel frame / masonry.
Culture & community: Flexible design incorporating dual-use of tiltable climbing wall / open floor space.
Land and Nature: Piles incorporated to reduce spoil waste from deep excavation for pad foundations and limiting the volume of material directed to landfill.
Materials and Products / Land and Nature: Piling to reduce the impact to the existing trees in close proximity to the proposed excavation.

APPENDICES

A Sketches of Proposals, Suggested Sequencing and Temporary Works

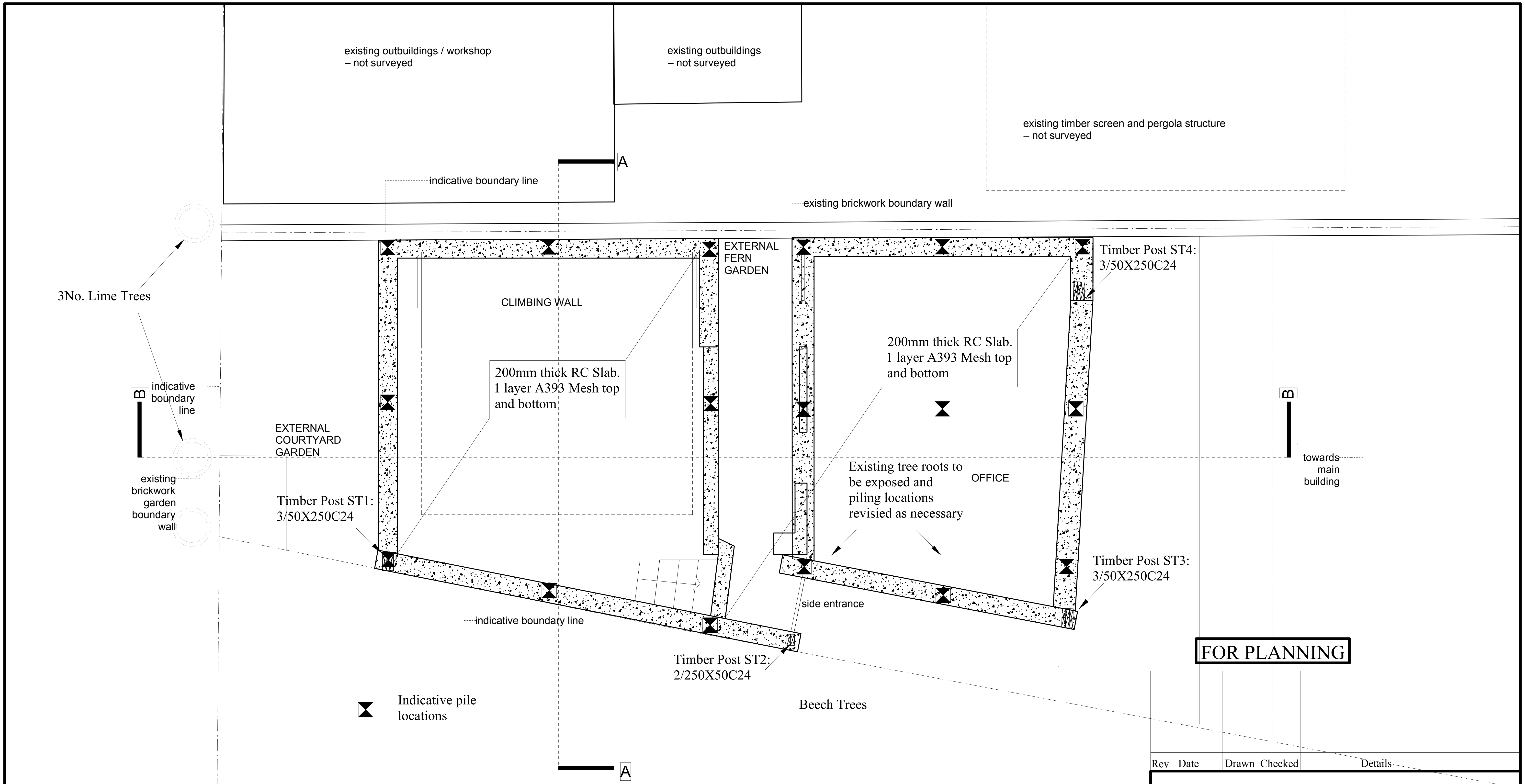
B Structural Calculations

C Arboricultural Investigation Report from Parsons Tree Care

D Borehole Records

APPENDIX A

Sketches of Proposals, Suggested Sequencing and Temporary Works



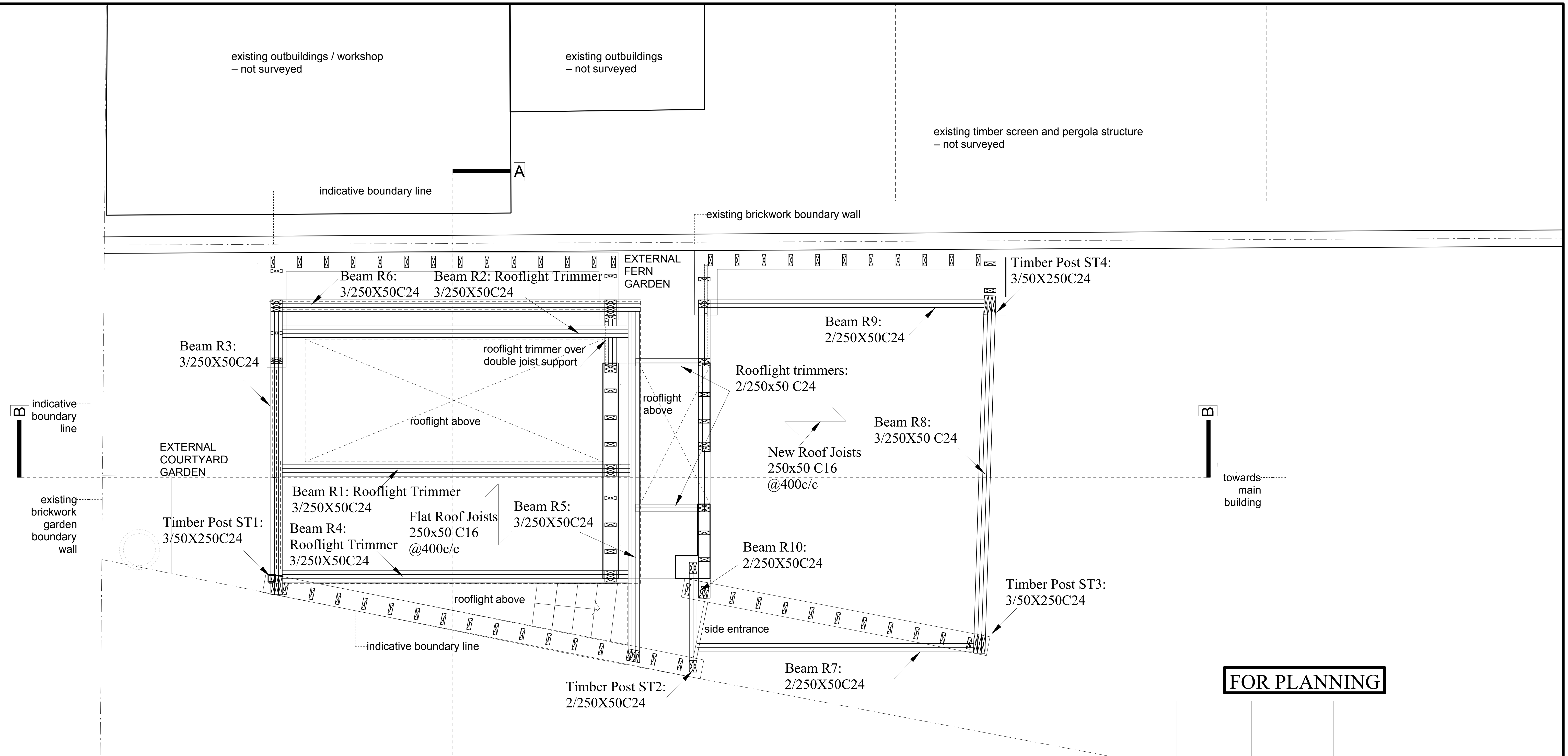
GROUND FLOOR LAYOUT SHOWING FOUNDATION UNDER

SCALE 1:50

eng17 Limited

17 Clacton Road, London E17 8AP
Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Proposed Ground Floor Showing Structure Under		
SCALE ON A3	1 to 50	DATE	08.02.21
DRAWN BY	PC	CHECKED BY	PC
DRAWING No.	210111/01	REV	



A

GROUND FLOOR LAYOUT SHOWING STRUCTURE OVER

SCALE 1:50

Rev	Date	Drawn	Checked	Details

eng17 Limited

17 Clacton Road, London E17 8AP
Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Proposed Ground Floor Showing Structure Over		
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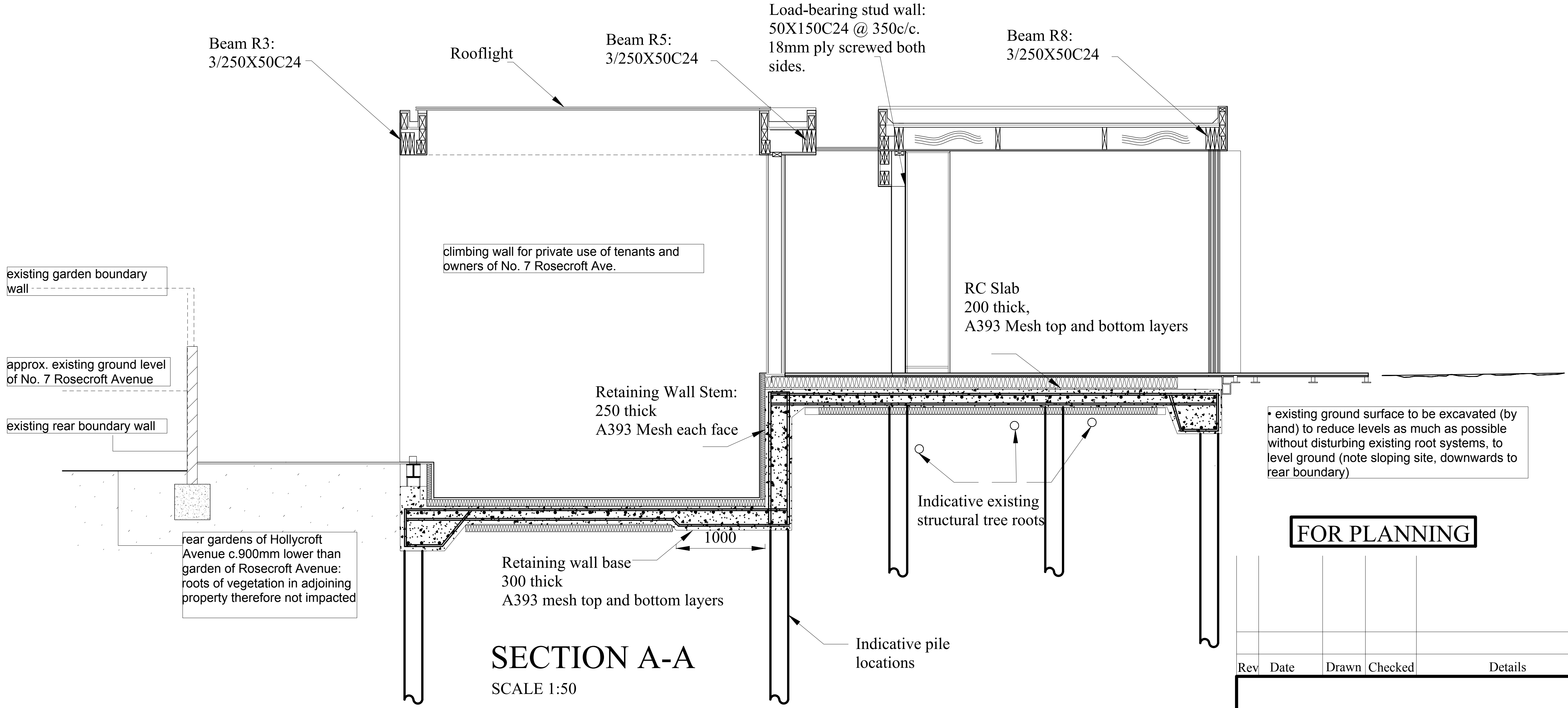
DRAWN BY	PC	CHECKED BY	PC
DRAWING No.	210111/02	REV	



SECTION A-A

SCALE 1:50

FOR PLANNING

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17 Clacton Road, London E17 8AP
Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Section A-A		
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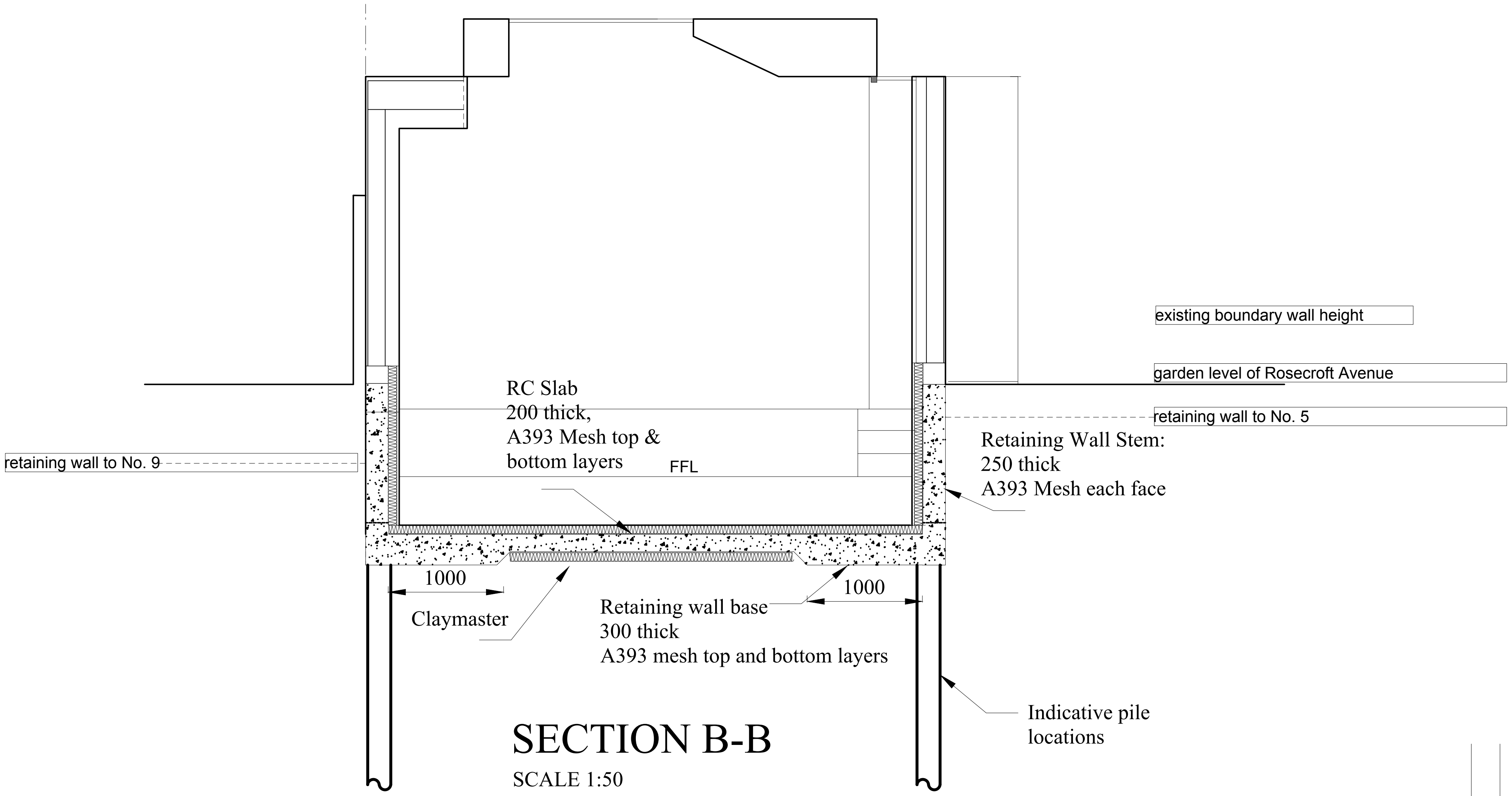
DRAWING No.	210111/03	REV	



SECTION B-B

SCALE 1:50

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Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Section B-B		
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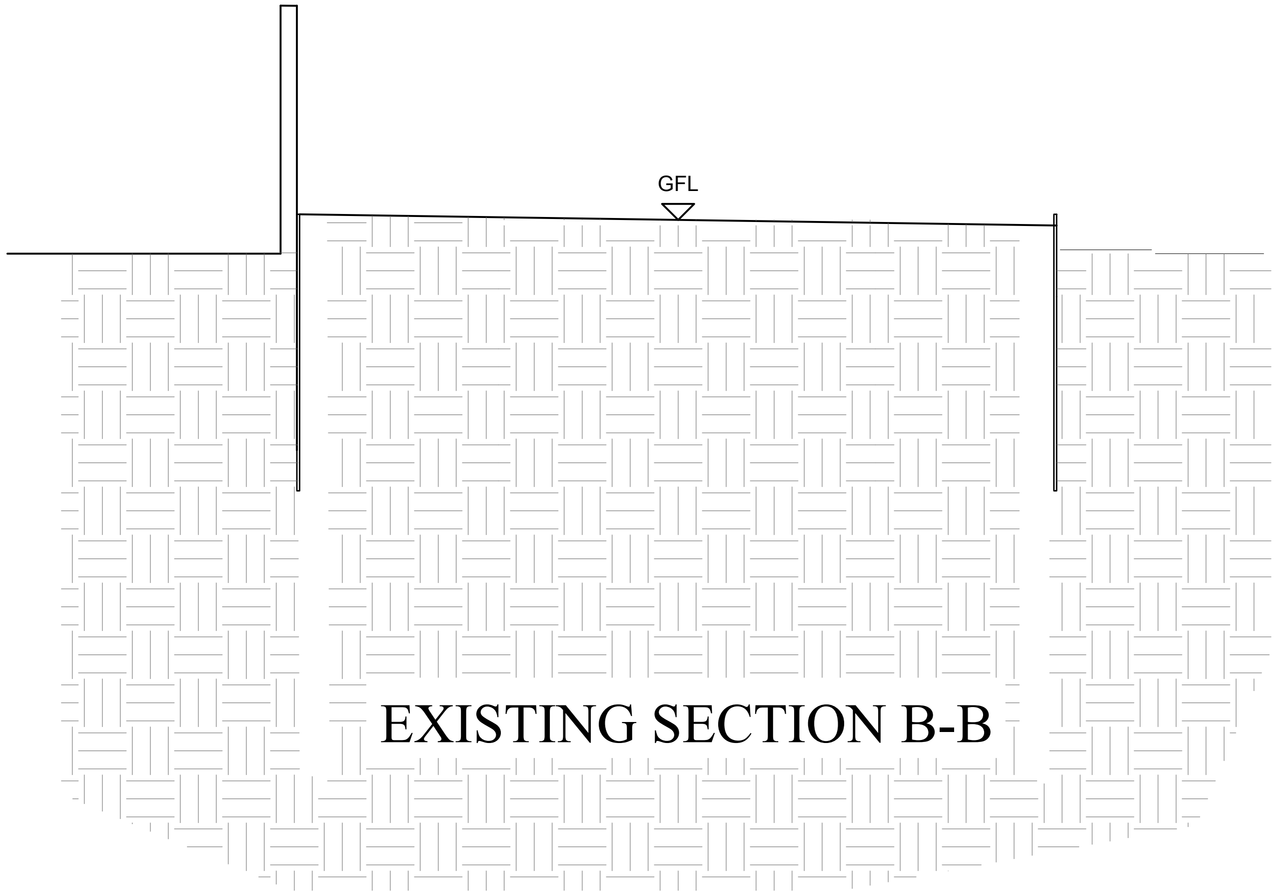
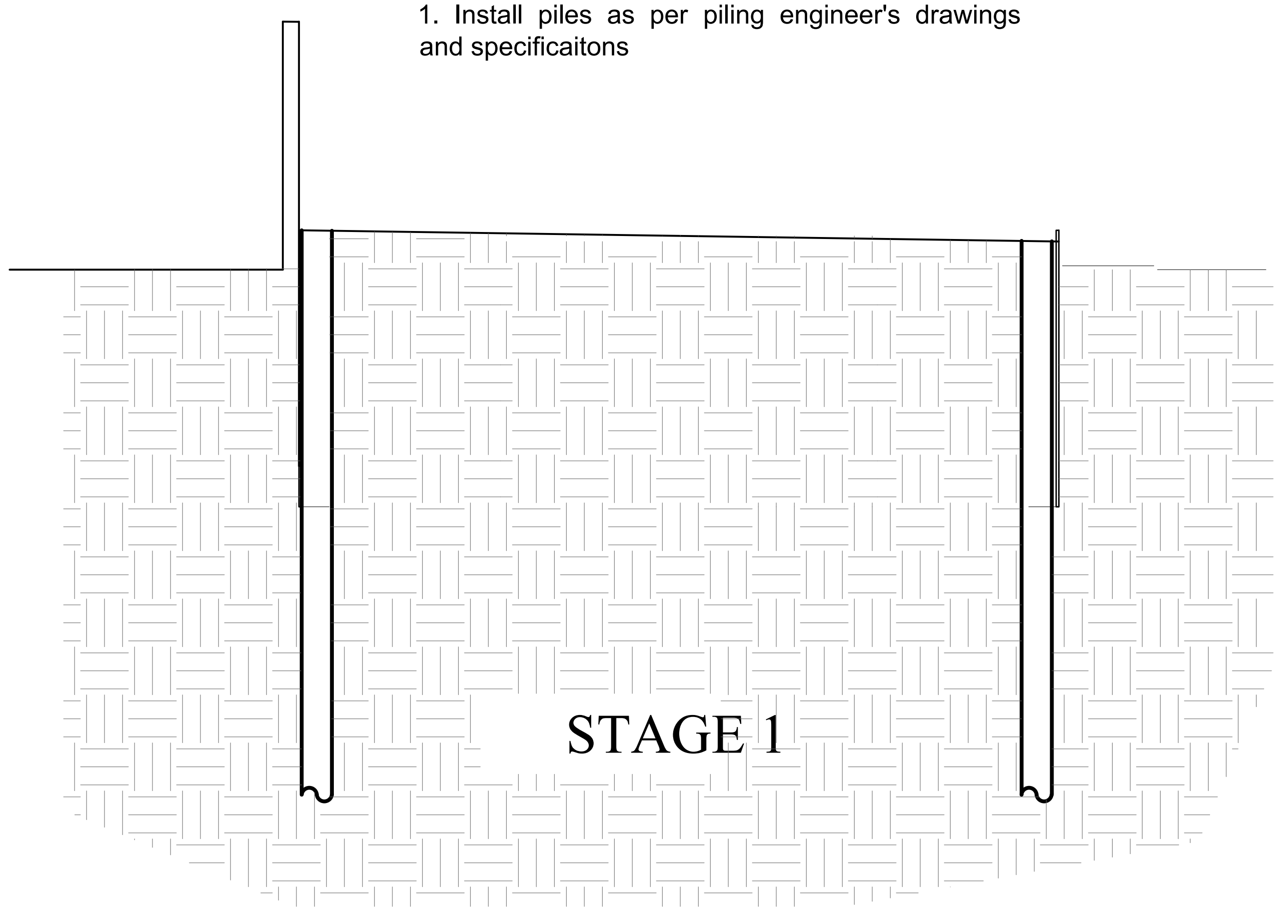
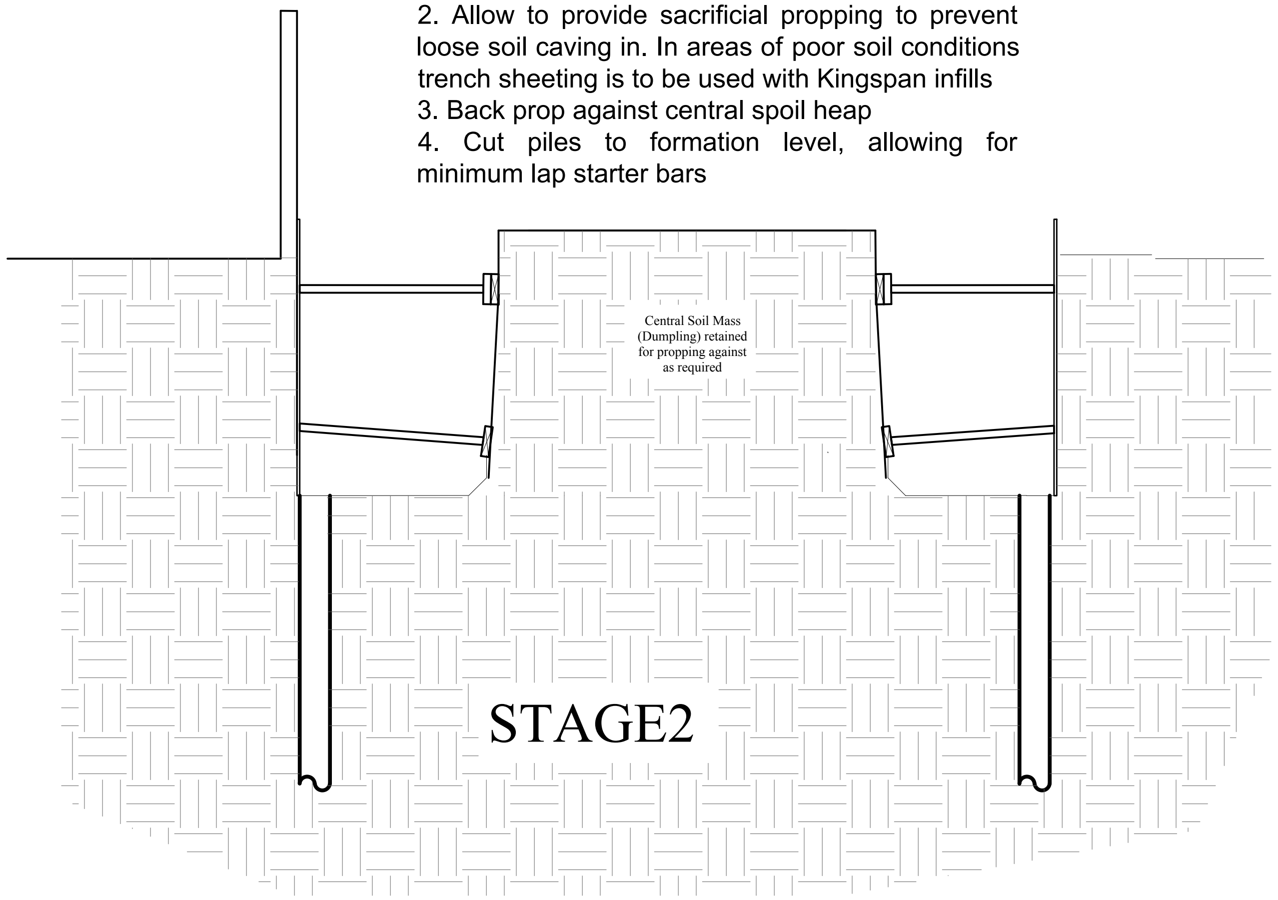
DRAWN BY	PC	CHECKED BY	PC
DRAWING No.	210111/04	REV	



1. Install piles as per piling engineer's drawings and specificaitons



1. Excavate to formation level in 1m wide sections as shown on engineer's drawing S01
2. Allow to provide sacrificial propping to prevent loose soil caving in. In areas of poor soil conditions trench sheeting is to be used with Kingspan infills
3. Back prop against central spoil heap
4. Cut piles to formation level, allowing for minimum lap starter bars



FOR PLANNING

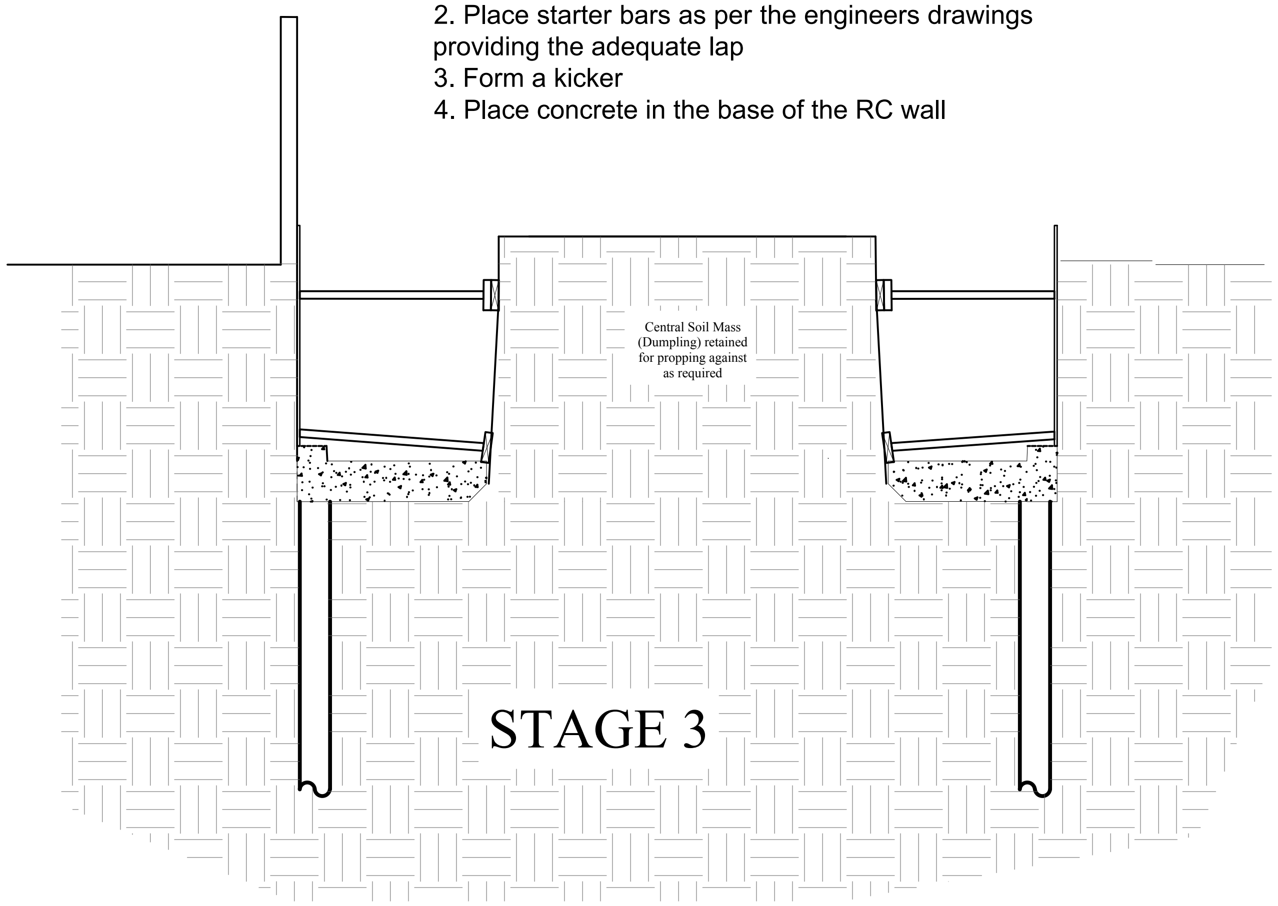
Rev	Date	Drawn	Checked	Details

eng17 Limited

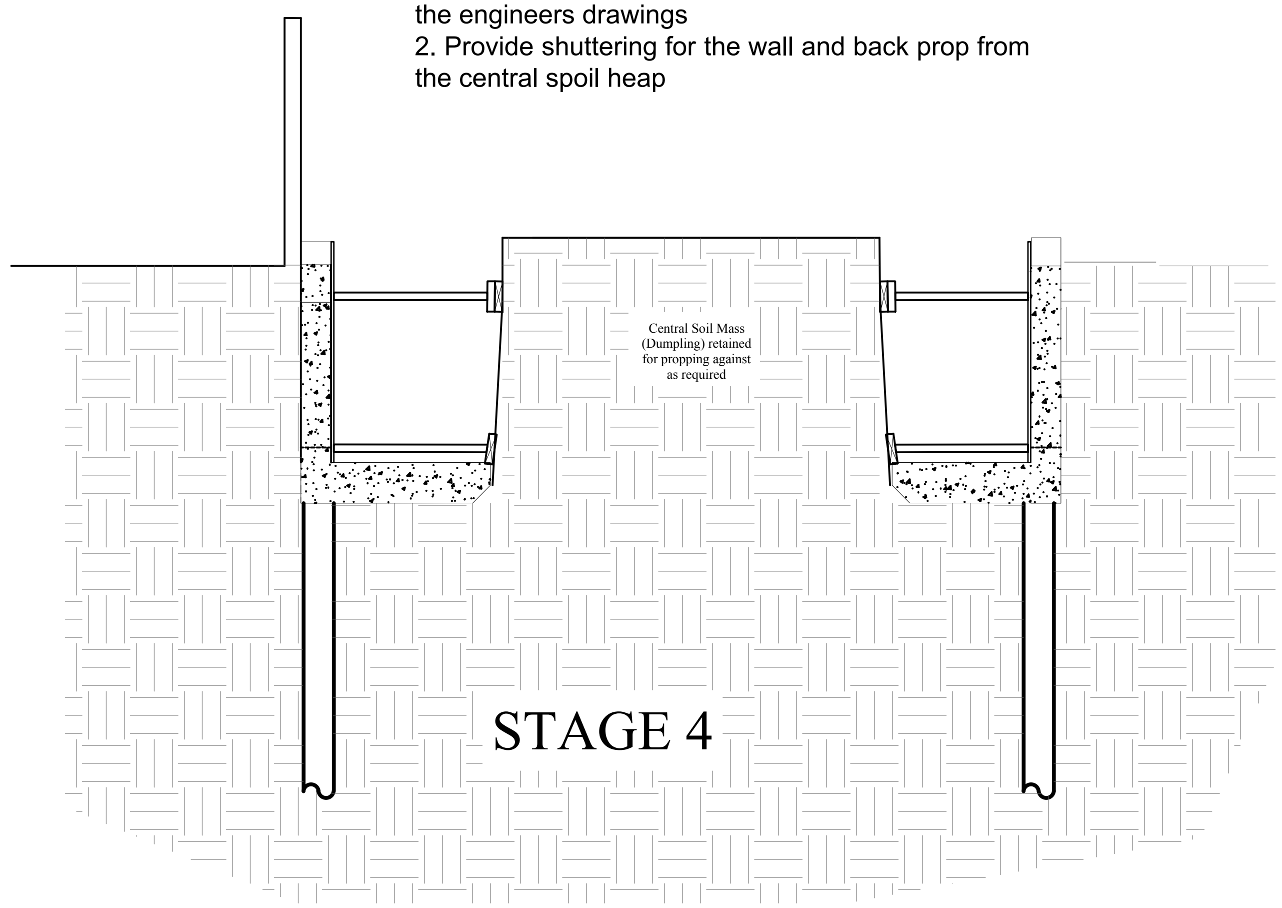
17 Clacton Road, London E17 8AP
Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Temporary Works Stage 1-2		
SCALE ON A3	1 to 50	DATE	08.02.21
DRAWN BY	PC	CHECKED BY	PC
DRAWING No.	210111/TW1	REV	

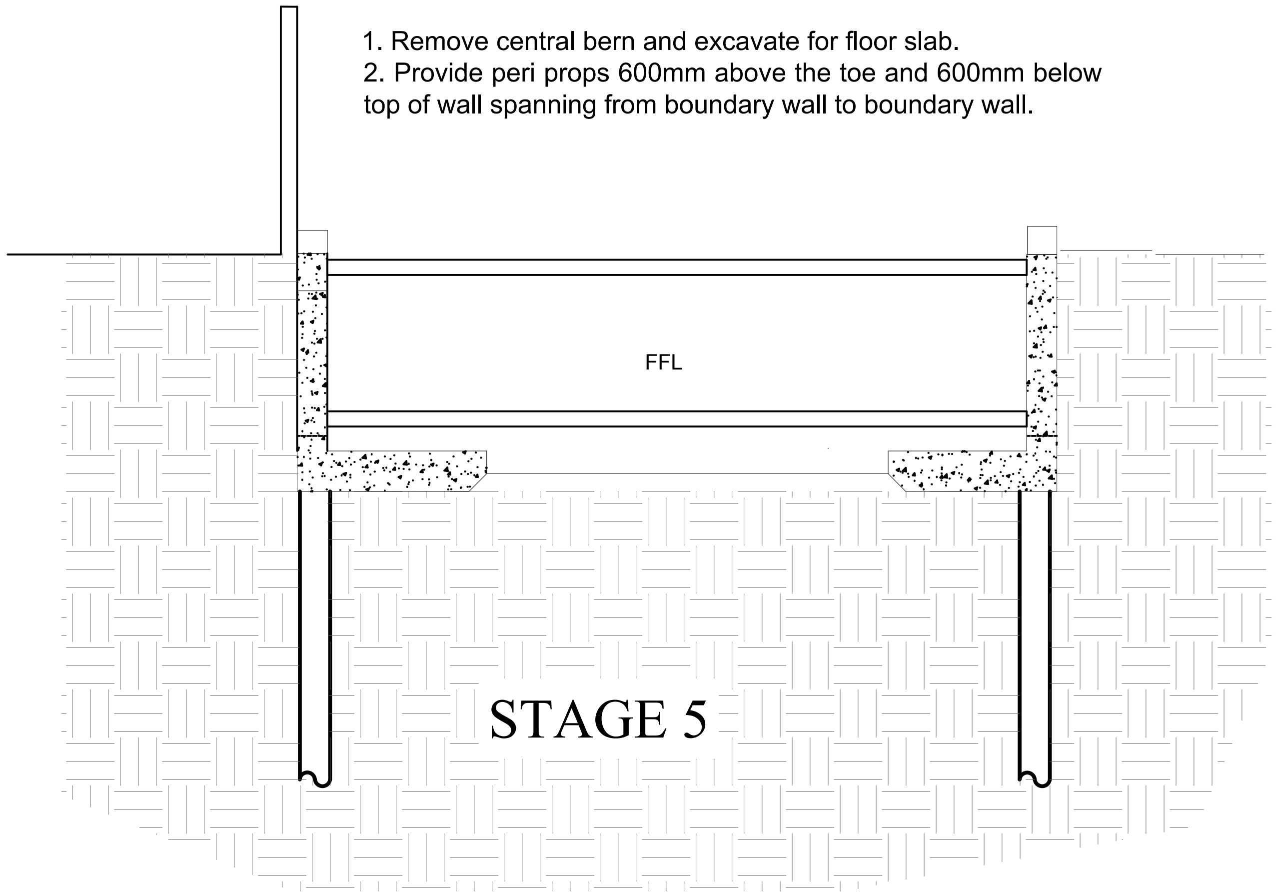
1. Place reinforcing bars/mesh in the base as per the engineers drawings
2. Place starter bars as per the engineers drawings providing the adequate lap
3. Form a kicker
4. Place concrete in the base of the RC wall



1. Place reinforcing bars/mesh in the stem as per the engineers drawings
2. Provide shuttering for the wall and back prop from the central spoil heap



1. Remove central bern and excavate for floor slab.
2. Provide peri props 600mm above the toe and 600mm below top of wall spanning from boundary wall to boundary wall.



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info@eng17.co.uk

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PROJECT	Home / office		
TITLE	Temporary Works Stage 3-5		
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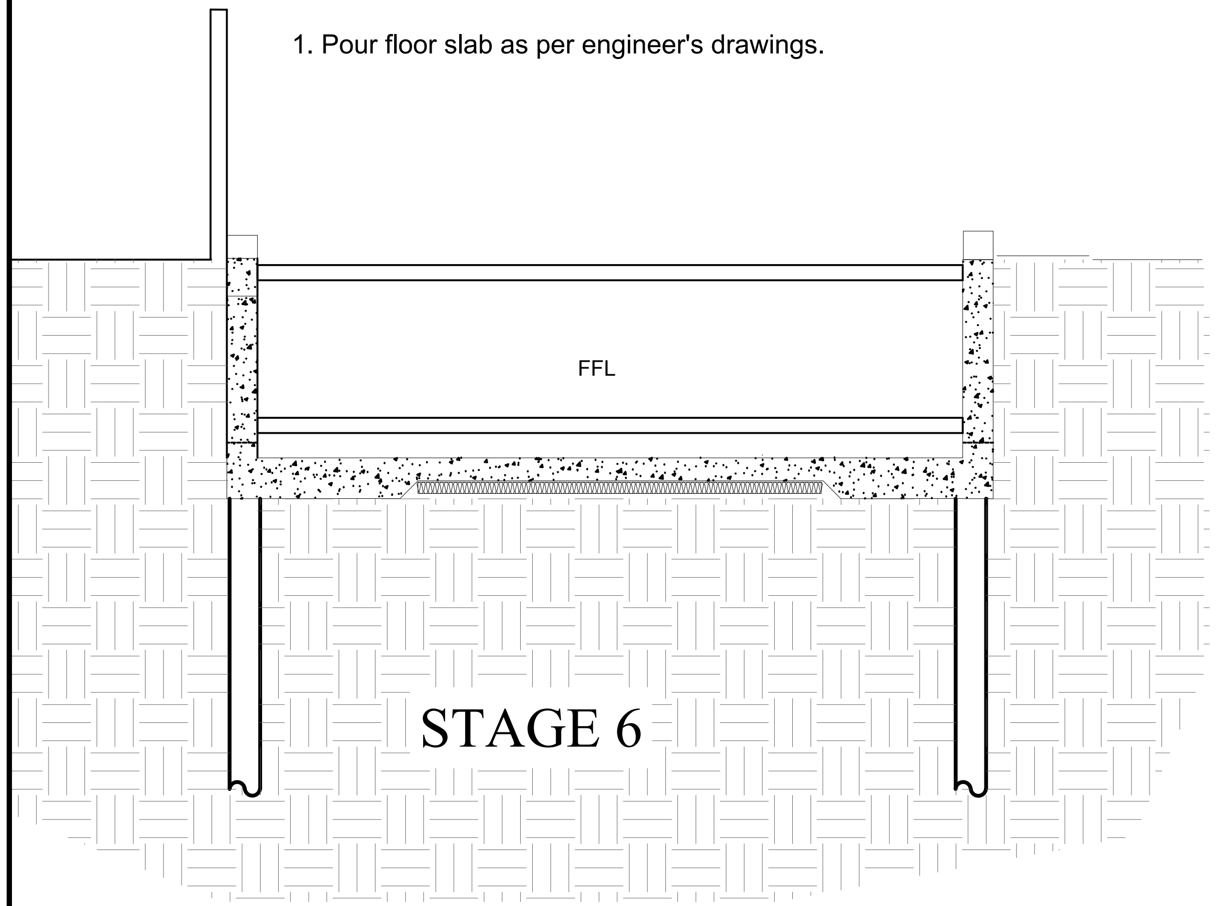

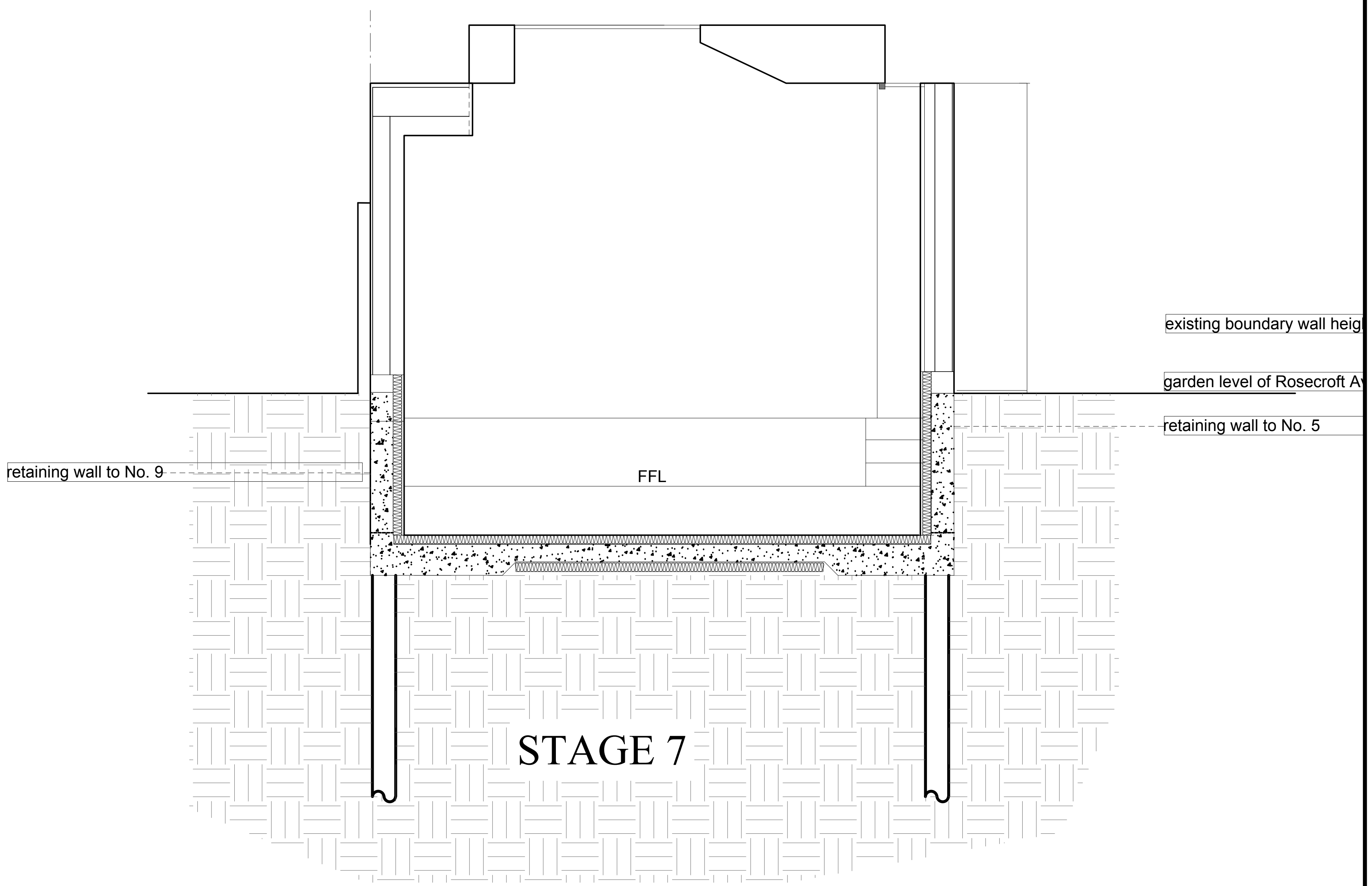

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Tel: 07716757751
info@eng17.co.uk

SITE	7 Rosecroft Avenue, London NW3 7 QA		
PROJECT	Home / office		
TITLE	Temporary Works Stage 3-5		
SCALE ON A3	1 to 50	DATE	08.02.21
DRAWN BY	PC	CHECKED BY	PC
DRAWING No.	210111/TW3	REV	

APPENDIX B

Structural Calculations

ENG17

Structural Calculations

Prepared for: 7 Rosecroft Avenue

Prepared by: Paul Cullen

9 February 2021

Job No. ENG/210111

DESIGN SUMMARY

Home office structural design.
Timber frame superstructure with RC retaining walls and floor slab to support climbing wall.
Piling recommended to to proximity of trees and ground conditions.

DESIGN AND CALCULATION OF ROOF JOISTS

Analysis

Worst case span: 3.7m

See Tedds - Provide 50 x 250 C16 @ 400mm centres

DESIGN AND CALCULATION OF ROOFLIGHT SUPPORT BEAM R1

Analysis

Span: 4.3m
UDL from roof and roof light

Calculation of Load

Description		Dead Load	Live Load
Roof	3.2m/2 (0.65 + 0.75kN/m2	1	1.2
Total		**1**	**1.2**

Reactions: 2.5 +2.6
See Tedds - Provide 3/50 x 200 C24

DESIGN AND CALCULATION OF FRONT DOOR BEAM R8

Analysis

Span: 4.4m
UDL from roof

Calculation of Load

Description		Dead Load	Live Load
Roof	3.7m/2 (0.65 + 0.75)kN/m2	1.2	1.4
Total		**1.2**	**1.4**

Reactions: 3.0 + 3.1kN
See Tedds - Provide 3/50 x 250 C24

CHECKING FRONT DOOR BEAM AS FRAME

Analysis

Assume full wind load from side wall panels transferred through connections and not shear wall section

Calculation of Load

Wind load acting on frame = 3.8m/2 x 2.82m x 0.8kN/m2 = 4.3kN

See Tedds - Provide 3/50 x 250 C24 columns and beam

Design moment connection for 12.9kNm

CHECKING WALL PANELS FOR RACKING

Analysis

Assume panel sizes of 4.0 long x 2.4m high and 5.2m long x 2.4m high

See Tedds - Provide 18mm ply sheathing each side - ok

DESIGN AND CALCULATION OF RETAINING WALLS

Analysis

Design as unpropped cantilever.

Retaining wall stem 1.5m high to retain 1.5m earth

Assume base 1.0m long, no heel

Assume 500mm water - none found on site to depth of 1.0m bgl

Vertical load on stem of wall and roof above = 4.0kN/m

See Tedds
Provide 250mm thick stem with A393 Mesh
300 thick base with A393 Mesh

DESIGN AND CALCULATION OF FLOOR SLAB

Analysis

Assume two way spanning, 5.0m each direction
Max load = 3.0kN/m2
Check for punching shear for 34kN - greater of floor area on central pile and assumed climbing wall support point load.

See Tedds - Provide 200mm thick slab with A393 Mesh top and bottom layers

CHECKING SCREW PILE LOAD CAPACITY

Analysis

TBC following piling design

Tekla Tedds eng17 17 Clacton Road London E17 8AP	Project 6 Beech Drive				Job no.	
	Calcs for Roof Joist check				Start page no./Revision 1	
	Calcs by PC	Calcs date 04/02/2021	Checked by	Checked date	Approved by	Approved date

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth b = **47** mm

Joist depth h = **250** mm

Joist spacing s = **400** mm

Timber strength class **C16**

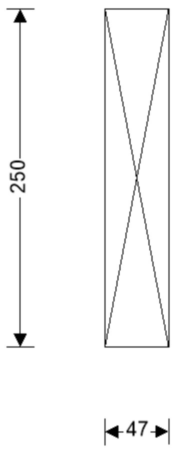
Service class of timber **1**

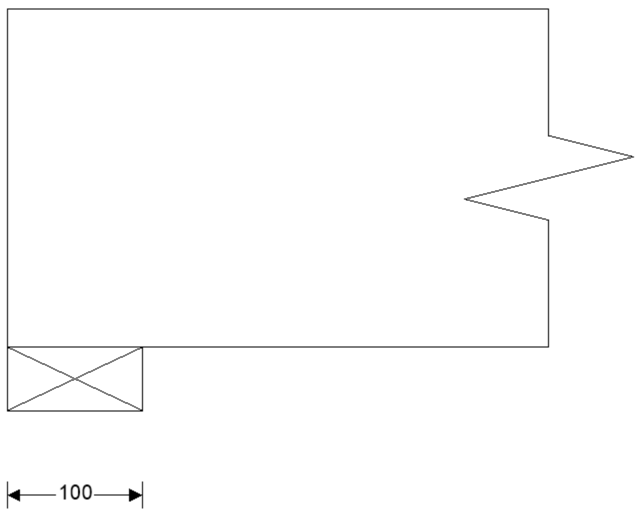
Span details

Number of spans N_{span} = **1**

Length of bearing L_b = **100** mm

Effective length of span L_{s1} = **3700** mm





Section properties

Second moment of area $I = b \times h^3 / 12 =$ **61197917** mm^4

Section modulus $Z = b \times h^2 / 6 =$ **489583** mm^3

Loading details

Joist self weight $F_{swt} = b \times h \times \rho_{char} \times g_{acc} =$ **0.04** kN/m

Dead load F_{d_udl} = **0.75** kN/m^2

Imposed UDL(Medium term) F_{i_udl} = **0.60** kN/m^2

Imposed point load (Short term) F_{i_pt} = **0.90** kN

Modification factors

Service class for bending parallel to grain K_{2m} = **1.00**

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor K_7 = **1.02**

Load sharing factor K_8 = **1.10**

Consider medium term loads

Load duration factor K_3 = **1.25**

Maximum bending moment M = **0.985** kNm

Maximum shear force V = **1.065** kN

Maximum support reaction R = **1.065** kN

Maximum deflection δ = **2.792** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm²

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.435** N/mm²

Applied bending stress σ_{m_max} = M / Z = **2.012** N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.921** N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.136** N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm²

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.025** N/mm²

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.227** N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **11.100** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **2.609** mm

Shear deflection δ_{shear} = **0.183** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **2.792** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Load duration factor K_3 = **1.50**

Maximum bending moment M = **1.407** kNm

Maximum shear force V = **1.521** kN

Maximum support reaction R = **1.521** kN

Maximum deflection δ = **3.546** mm

Check bending stress

Bending stress σ_m = **5.300** N/mm²

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.922** N/mm²

Applied bending stress σ_{m_max} = M / Z = **2.874** N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress τ = **0.670** N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **1.106** N/mm^2

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.194** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.630** N/mm^2

Applied bearing stress $\sigma_{c\_max} = R / (b \times L_b)$ = **0.324** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **11.100** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **3.285** mm

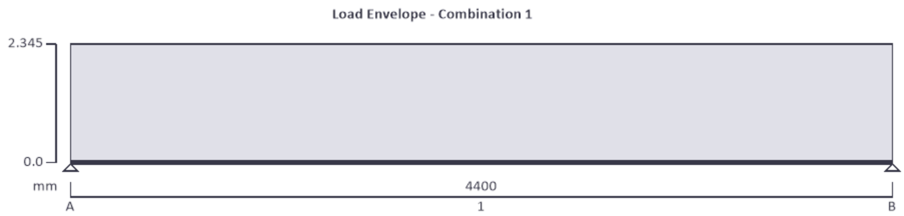
Shear deflection δ_{shear} = **0.261** mm

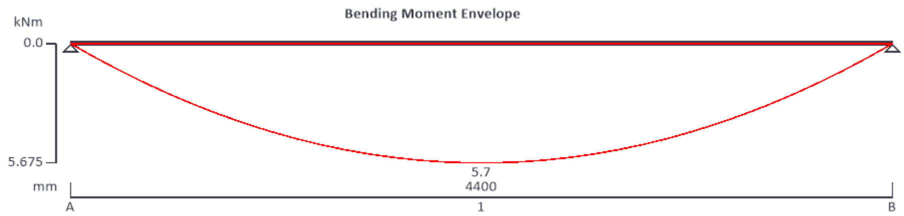
Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **3.546** mm

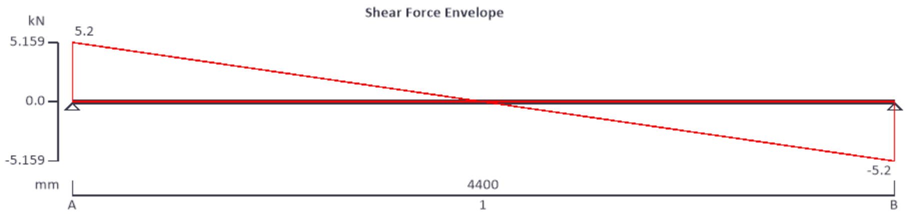
PASS - Actual deflection within permissible limits

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.000 kN/m
Imposed full UDL 1.200 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **5.675** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **5.675** kNm	
Maximum shear	F_{max} = **5.159** kN	F_{min} = **-5.159** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **5.159** kN	
Total load on beam	W_{tot} = **10.319** kN	

Reactions at support A R_{A_max} = **5.159** kN R_{A_min} = **5.159** kN

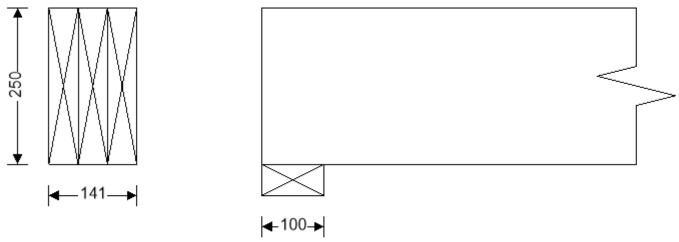
Unfactored dead load reaction at support A R_{A_Dead} = **2.519** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **2.640** kN

Reactions at support B R_{B_max} = **5.159** kN R_{B_min} = **5.159** kN

Unfactored dead load reaction at support B R_{B_Dead} = **2.519** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.640** kN



Timber section details

Breadth of sections b = **47** mm

Depth of sections h = **250** mm

Number of sections in member N = **3**

Overall breadth of member $b_b = N \times b$ = **141** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Long term**

Length of span L_{s1} = **4400** mm

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **35250** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **1468750** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **828375** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **183593750** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **58400438** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **72.2** mm

$i_y = \sqrt{(I_y / A)}$ = **40.7** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.00**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.02**

Load sharing - cl.2.10.11 K_8 = **1.10**

Minimum modulus of elasticity - Table 20 K_9 = **1.21**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19 **5.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.77**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.640}$ N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.366}$ N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.139}$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.417}$ N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x = \mathbf{3.864}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.459}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.781}$ N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) = \mathbf{0.220}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.281}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 = \mathbf{8712}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = \mathbf{13.200}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{7.156}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.355}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{7.511}$ mm

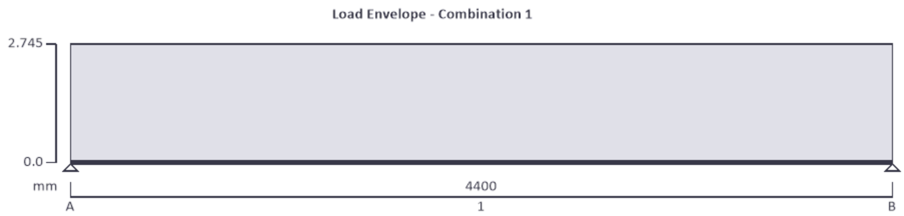
$\delta_a / \delta_{adm} = \mathbf{0.569}$

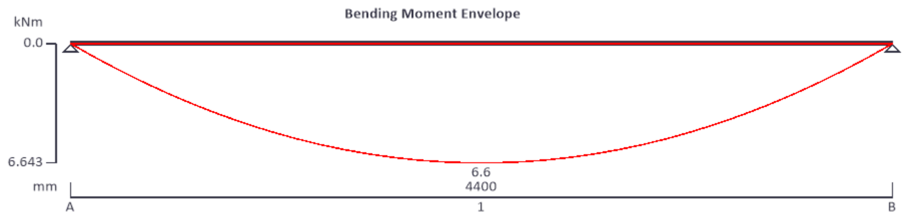
PASS - Total deflection is less than permissible deflection

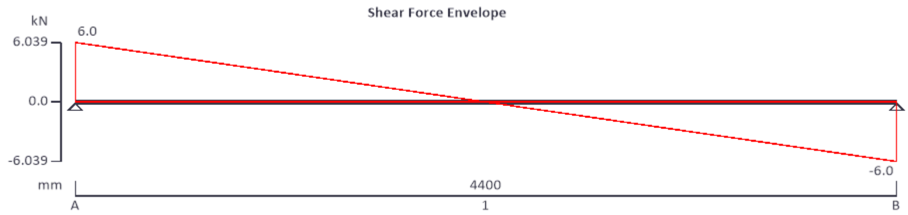
Tekla Tedds eng17 17 Clacton Road London E17 8AP	Project 6 Beech Drive				Job no.	
	Calcs for R8				Start page no./Revision 1	
	Calcs by PC	Calcs date 04/02/2021	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.200 kN/m
Imposed full UDL 1.400 kN/m

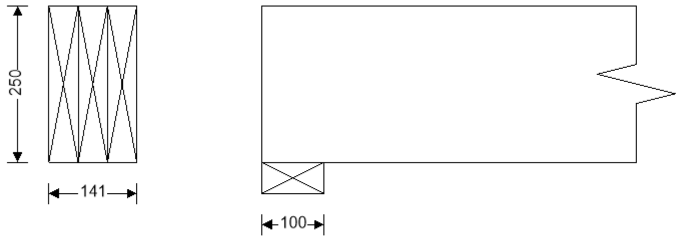
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **6.643** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **6.643** kNm	
Maximum shear	F_{max} = **6.039** kN	F_{min} = **-6.039** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **6.039** kN	
Total load on beam	W_{tot} = **12.079** kN	

Reactions at support A R_{A_max} = **6.039** kN R_{A_min} = **6.039** kN
Unfactored dead load reaction at support A R_{A_Dead} = **2.959** kN
Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **3.080** kN
Reactions at support B R_{B_max} = **6.039** kN R_{B_min} = **6.039** kN
Unfactored dead load reaction at support B R_{B_Dead} = **2.959** kN
Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **3.080** kN



Timber section details
Breadth of sections b = **47** mm
Depth of sections h = **250** mm
Number of sections in member N = **3**
Overall breadth of member $b_b = N \times b$ = **141** mm
Timber strength class **C24**

Member details
Service class of timber **1**
Load duration **Long term**
Length of span L_{s1} = **4400** mm
Length of bearing L_b = **100** mm

Section properties
Cross sectional area of member $A = N \times b \times h$ = **35250** mm²
Section modulus $Z_x = N \times b \times h^2 / 6$ = **1468750** mm³
$Z_y = h \times (N \times b)^2 / 6$ = **828375** mm³
Second moment of area $I_x = N \times b \times h^3 / 12$ = **183593750** mm⁴
$I_y = h \times (N \times b)^3 / 12$ = **58400438** mm⁴
Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **72.2** mm
$i_y = \sqrt{(I_y / A)}$ = **40.7** mm

Modification factors
Duration of loading - Table 17 K_3 = **1.00**
Bearing stress - Table 18 K_4 = **1.00**
Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.02**
Load sharing - cl.2.10.11 K_8 = **1.10**
Minimum modulus of elasticity - Table 20 K_9 = **1.21**

Lateral support - cl.2.10.8
Ends held in position and members held in line, as by direct connection of sheathing, deck or joists
Permissible depth-to-breadth ratio - Table 19 **5.00**
Actual depth-to-breadth ratio $h / (N \times b)$ = **1.77**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **2.640** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b) =$ **0.428** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.162**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.417** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **4.523** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.537**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.781** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.257** N/mm^2

$\tau_a / \tau_{adm} =$ **0.329**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 =$ **8712** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **13.200** mm

Bending deflection $\delta_{b\_s1} =$ **8.376** mm

Shear deflection $\delta_{v\_s1} =$ **0.415** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **8.792** mm

$\delta_a / \delta_{adm} =$ **0.666**

PASS - Total deflection is less than permissible deflection

TIMBER MEMBER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

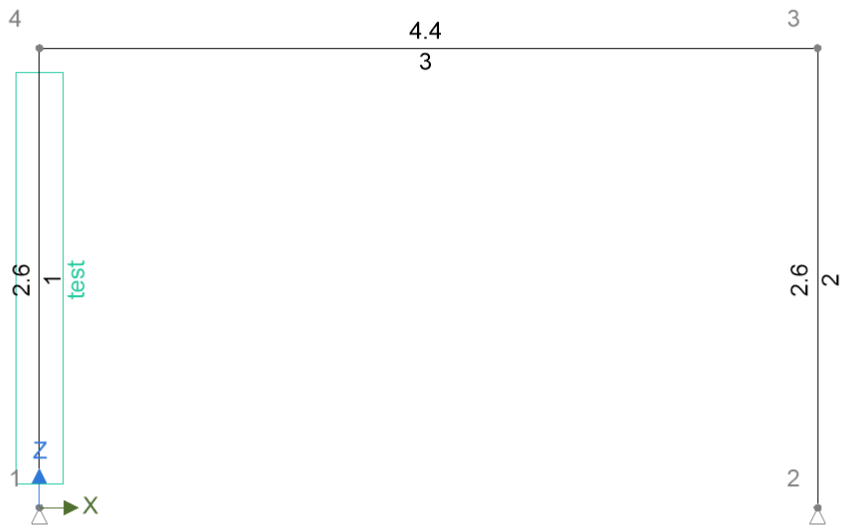
Tedds calculation version 2.2.04

ANALYSIS

Tedds calculation version 1.0.31

Geometry

Geometry (m) - C24 (EC5)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
C24 (EC5)	420	11	0.69	0

Sections

Name	Area	Moment of inertia		Shear area parallel to	
		Major	Minor	Minor	Major
	(cm²)	(cm⁴)	(cm⁴)	(cm²)	(cm²)
3/50x250	375	19531.3	7031.3	312.5	312.5
3/50x250(1)	375	19531.3	7031.3	312.5	312.5

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X	Z	X	Z	Rot.	Name	Angle	X	Z	Rot.
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°
1	0	0	Fixed	Fixed	Free		0	0	0	0
2	4.4	0	Fixed	Fixed	Free		0	0	0	0
3	4.4	2.6	Free	Free	Free		0	0	0	0
4	0	2.6	Free	Free	Free		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	2.6	1	4	3/50x250(1)	C24 (EC5)	Fixed	Fixed	Fixed	
2	2.6	2	3	3/50x250(1)	C24 (EC5)	Fixed	Fixed	Fixed	
3	4.4	4	3	3/50x250	C24 (EC5)	Fixed	Fixed	Fixed	

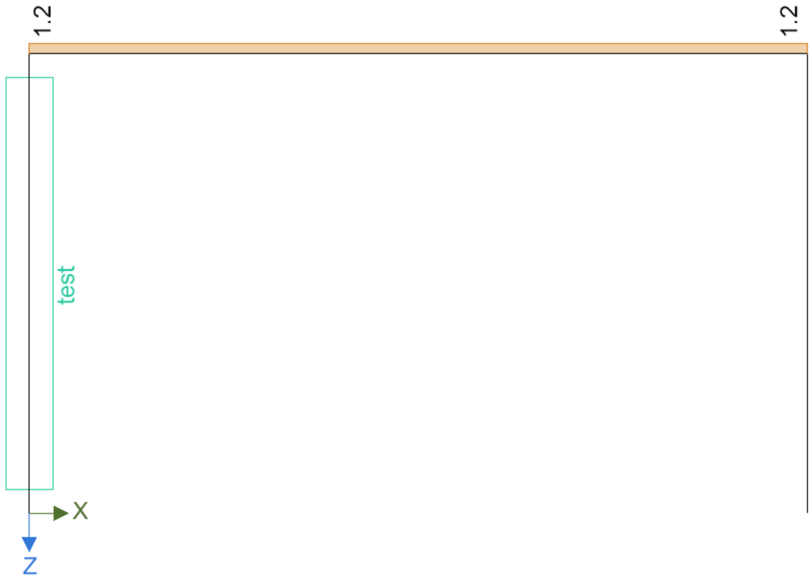
Members

Name	Elements	
	Start	End
test	1	1

Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m,kN)

Load combination factors

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

Node loads

Node	Load case	Force X (kN)	Force Z (kN)	Moment (kNm)
4	Imposed	4.3	0	0

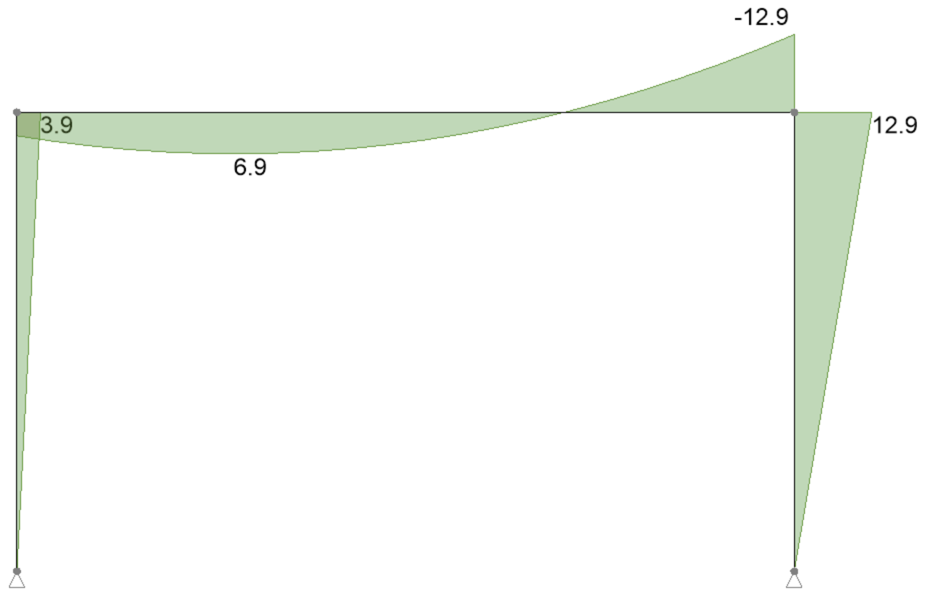
Element Loads

Element	Load case	Load Type	Orientation	Description
3	Permanent	UDL	GlobalZ	1.2 kN/m
3	Imposed	UDL	GlobalZ	1.4 kN/m

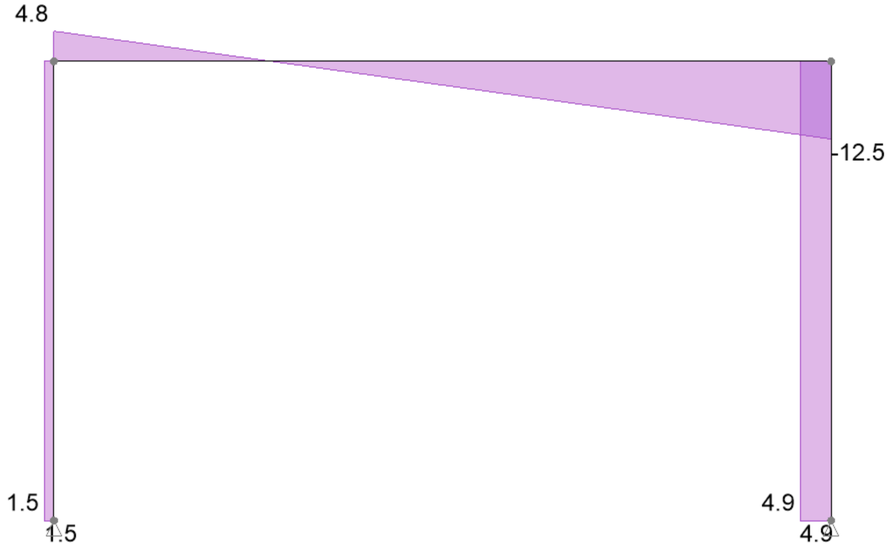
<u>Results</u>

Forces

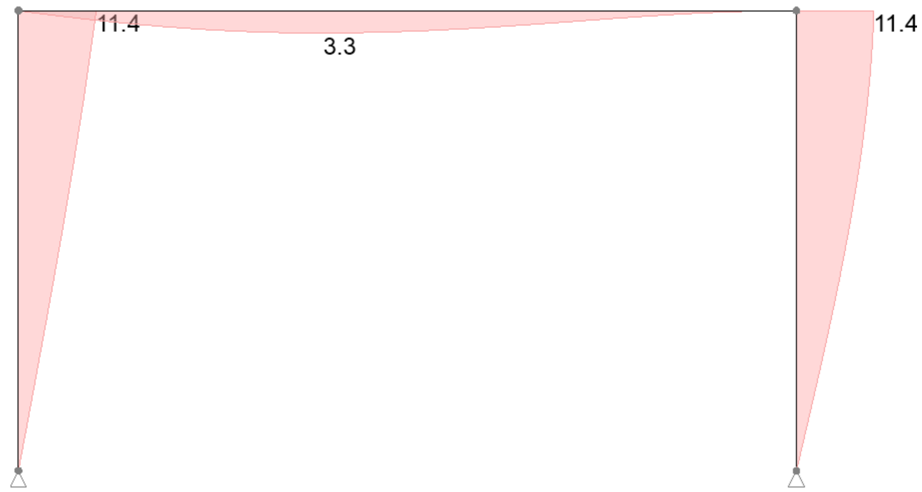
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



<u>test - Span 1</u>

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

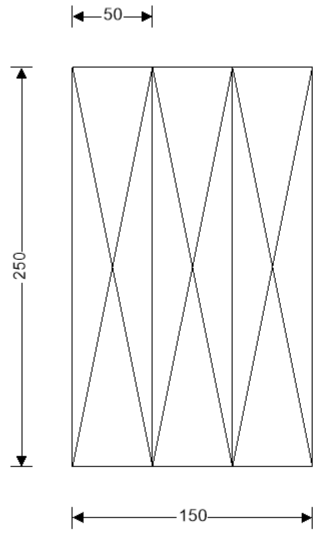
Timber section details

Number of timber sections in member N = **3**

Breadth of sections b = **50** mm

Depth of sections h = **250** mm

Timber strength class - EN 338:2016 Table 1 **C24**



3/50x250 timber sections
Cross-sectional area, A, 37500 mm^2
Section modulus, W_y, 1562500 mm^3
Section modulus, W_z, 312500 mm^3
Second moment of area, I_y, 195312500 mm^4
Second moment of area, I_z, 7812500 mm^4
Radius of gyration, i_y, 72.2 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m.k}$, 24 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length — L_b = **100** mm

<u>Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)</u>

Modification factors

Duration of load and moisture content - Table 3.1 — k_{mod} = **0.7**
Deformation factor - Table 3.2 — k_{def} = **0.6**
Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.67**
System strength factor - cl.6.6 — k_{sys} = **1.1**

Check compression parallel to the grain - cl.6.1.4

Design axial compression — P_d = **5.374** kN
Design compressive stress — $\sigma_{c,0,d} = P_d / A$ = **0.143** N/mm^2
Design compressive strength — $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **12.438** N/mm^2
$\sigma_{c,0,d} / f_{c,0,d}$ = **0.012**

PASS - Design parallel compression strength exceeds design parallel compression stress

<u>Check design at start of span</u>

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis — $F_{c,y,90,d}$ = **1.501** kN
Effective contact length — $L_{b,ef} = L_b$ = **100** mm
Design perpendicular compressive stress - exp.6.4 — $\sigma_{c,y,90,d} = F_{c,y,90,d} / (N \times b \times L_{b,ef})$ = **0.100** N/mm^2
Design perpendicular compressive strength — $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.481** N/mm^2
$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.068**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **1.501** kN
Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times N \times b \times h)$ = **0.090** N/mm^2
Design shear strength — $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2
$\tau_{y,d} / f_{v,y,d}$ = **0.038**

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2600$ mm $=$ **2340** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y =$ **32.424**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **0.55**

Effective length for z-axis bending $L_{e,z} =$ **0** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z =$ **0**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **0**

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor $\beta_c =$ **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **0.676**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **0.470**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.935**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) =$ **1.064**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) =$ **0.012**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) =$ **0.011**

PASS - Column stability is acceptable

Check design at end of span

Check shear force - Section 6.1.7

Design shear force $F_{y,d} =$ **1.501** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times N \times b \times h) =$ **0.090** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M =$ **2.369** N/mm²

$\tau_{y,d} / f_{v,y,d} =$ **0.038**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} =$ **3.901** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y =$ **2.497** N/mm²

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M =$ **14.215** N/mm²

$\sigma_{m,y,d} / f_{m,y,d} =$ **0.176**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} =$ **0.176**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} =$ **0.123**

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2600$ mm $=$ **2340** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y =$ **32.424**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **0.55**

Effective length for z-axis bending $L_{e,z} =$ **0** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z =$ **0**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **0**

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor $\beta_c =$ **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **0.676**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **0.470**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.935}$

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{1.064}$

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.188}$

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.134}$

PASS - Column stability is acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34 $k_{crit} = \mathbf{1.000}$

Beam stability check - exp.6.35 $(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.042}$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design at end of span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = \mathbf{11.4}$ mm

Quasi-permanent variable load factor $\psi_2 = \mathbf{0.3}$

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def}) = \mathbf{18.3}$ mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 125 = \mathbf{20.8}$ mm

$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.879}$

PASS - Allowable deflection exceeds final deflection

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

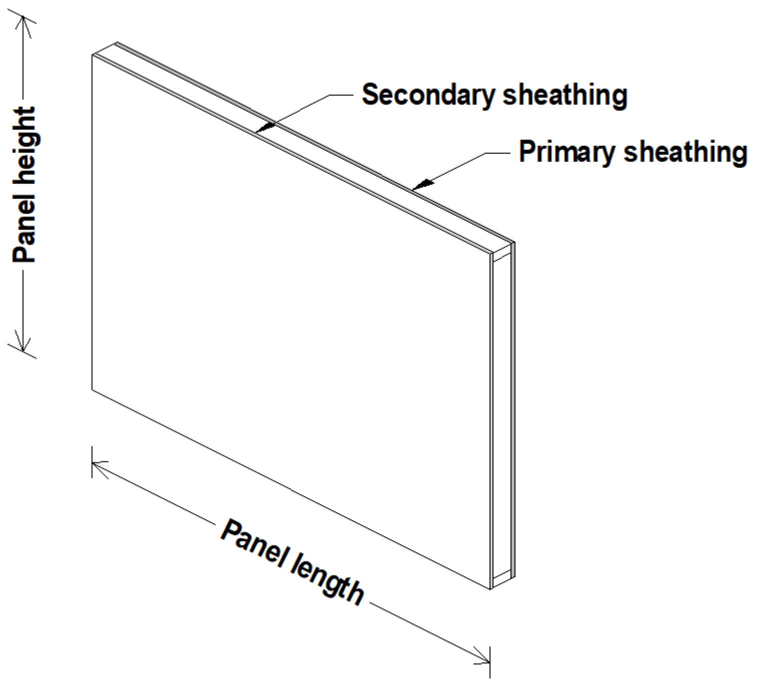
Building details

Building name **home / office**

Windward elevation **Front**

Building storey **Ground**

Racking panel no.1 -



Wall panel details

Length of panel	L = **4.000** m
Height of panel	H_{wp} = **2.400** m
Total area of wall panel	$A_t = L \times H_{wp}$ = **9.600** m^2
Aggregate area of framed panel openings	A_a = **0.000** m^2
Timber members	**38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall	F_{udl} = **2.400** kN/m
For calculation equivalent uniformly distributed load	$F = \min(F_{udl}, 10.5\text{ kN/m})$ = **2.400** kN/m

Primary sheathing details

Primary board type	**Plywood**
Standard board thickness	t_p = **9.50** mm
Proposed board thickness	T_p = **18.00** mm **WARNING - Greater than 1.25 × standard thickness**
Ratio of proposed to standard board thickness	$B_p = \min(\max(T_p / t_p, 0.75), 1.25)$ = **1.25**
Nail diameter	D_p = **3.00** mm
Standard perimeter nail spacing	s_p = **150** mm
Proposed perimeter nail spacing	S_p = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance R_{bp} = **1.680** kN/m

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter — $K_{101p} = D_p / 3\text{ mm} = \mathbf{1.000}$

Variation in nail spacing — $K_{102p} = \mathbf{1.000}$

Variation in board thickness — $K_{103p} = 2.8 \times B_p - B_p^2 - 0.8 = \mathbf{1.138}$

Material modification factors — $K_{mp} = K_{101p} \times K_{102p} \times K_{103p} = \mathbf{1.138}$

Secondary sheathing details

Secondary board type — **Plasterboard**

Standard board thickness — t_s = **12.50** mm

Proposed board thickness — T_s = **12.50** mm

Ratio of proposed to standard board thickness — $B_s = \min(\max(T_s / t_s, 0.75), 1.25) = \mathbf{1.00}$

Screw diameter — D_s = **3.50** mm

Standard perimeter screw spacing — s_s = **150** mm

Proposed perimeter screw spacing — S_s = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance — R_{bs} = **0.120** kN/m

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter — $K_{101s} = \mathbf{1.000}$

Variation in screw spacing — $K_{102s} = \mathbf{1.000}$

Variation in board thickness — $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8 = \mathbf{1.000}$

Material modification factors — $K_{ms} = K_{101s} \times K_{102s} \times K_{103s} = \mathbf{1.000}$

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels — $K_{104} = 2.4\text{ m} / H_{wp} = \mathbf{1.000}$

Length of walls — $K_{105} = (L / 2.4\text{ m})^{0.4} = \mathbf{1.227}$

Fully framed openings in walls — $K_{106} = (1 - 1.3 \times A_a / A_t)^2 = \mathbf{1.000}$

Vertical load on timber frame wall — $K_{107} = 1 + [(0.09 \times (F / 1\text{ kN/m}) - 0.0015 \times (F / 1\text{ kN/m})^2) \times (2.4\text{ m} / L)^{0.4}]$

$K_{107} = \mathbf{1.169}$

Interaction — $K_{108} = \mathbf{1.100}$

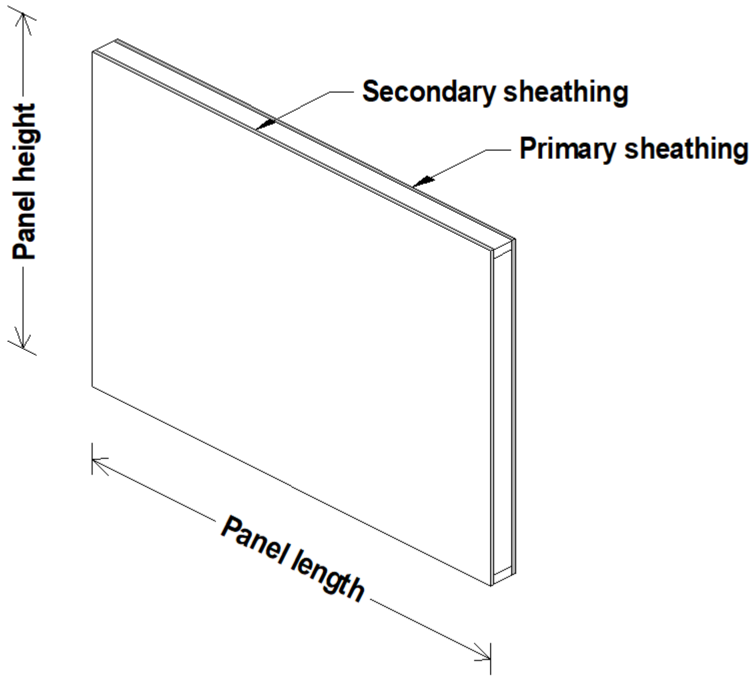
Wall modification factors — $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = \mathbf{1.577}$

Racking resistance of wall panel

Racking resistance of wall panel — $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **12.815** kN

Racking resistance of plasterboard only — $R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **0.757** kN

Racking panel no.2 -



Wall panel details

Length of panel — L = **5.200** m

Height of panel — H_{wp} = **2.400** m

Total area of wall panel — $A_t = L \times H_{wp}$ = **12.480** m^2

Aggregate area of framed panel openings — A_a = **0.000** m^2

Timber members — **38 mm x 72 mm or larger**

Uniformly distributed load on timber frame wall — F_{udl} = **2.400** kN/m

For calculation equivalent uniformly distributed load $F = \min(F_{udl}, 10.5\text{ kN/m})$ = **2.400** kN/m

Primary sheathing details

Primary board type — **Plywood**

Standard board thickness — t_p = **9.50** mm

Proposed board thickness — T_p = **18.00** mm **WARNING - Greater than 1.25 × standard thickness**

Ratio of proposed to standard board thickness — $B_p = \min(\max(T_p / t_p, 0.75), 1.25)$ = **1.25**

Nail diameter — D_p = **3.00** mm

Standard perimeter nail spacing — s_p = **150** mm

Proposed perimeter nail spacing — S_p = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance — R_{bp} = **1.680** kN/m

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter — $K_{101p} = D_p / 3\text{ mm}$ = **1.000**

Variation in nail spacing — K_{102p} = **1.000**

Variation in board thickness — $K_{103p} = 2.8 \times B_p - B_p^2 - 0.8$ = **1.138**

Material modification factors — $K_{mp} = K_{101p} \times K_{102p} \times K_{103p}$ = **1.138**

Secondary sheathing details

Secondary board type — **Plasterboard**

Standard board thickness t_s = **12.50** mm
Proposed board thickness T_s = **12.50** mm
Ratio of proposed to standard board thickness $B_s = \min(\max(T_s / t_s, 0.75), 1.25)$ = **1.00**
Screw diameter D_s = **3.50** mm
Standard perimeter screw spacing s_s = **150** mm
Proposed perimeter screw spacing S_s = **150** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance R_{bs} = **0.120** kN/m

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter K_{101s} = **1.000**
Variation in screw spacing K_{102s} = **1.000**
Variation in board thickness $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8$ = **1.000**
Material modification factors $K_{ms} = K_{101s} \times K_{102s} \times K_{103s}$ = **1.000**

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels $K_{104} = 2.4\text{ m} / H_{wp}$ = **1.000**
Length of walls K_{105} = **1.320**
Fully framed openings in walls $K_{106} = (1 - 1.3 \times A_a / A_t)^2$ = **1.000**
Vertical load on timber frame wall $K_{107} = 1 + [(0.09 \times (F / 1\text{ kN/m}) - 0.0015 \times (F / 1\text{ kN/m})^2) \times (2.4\text{ m} / L)^{0.4}]$
K_{107} = **1.152**
Interaction K_{108} = **1.100**
Wall modification factors $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108}$ = **1.673**

Racking resistance of wall panel

Racking resistance of wall panel $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms})$ = **17.669** kN
Racking resistance of plasterboard only $R_{PO} = L \times K_w \times R_{bs} \times K_{ms}$ = **1.044** kN

Total racking resistance of building

Total racking resistance of all panels R_{total} = **30.484** kN
Total contribution of masonry M_{total} = **0.000** kN
Total contribution of plasterboard P_{total} = **1.801** kN
Total contribution of category 1 and 2 materials $S_{total} = R_{total} - M_{total} - P_{total}$ = **28.683** kN
Maximum contribution of plasterboard $P_{max} = 0.5 \times S_{total}$ = **14.342** kN
Total racking resistance of building $R_{build} = S_{total} + M_{total} + \min(P_{total}, P_{max})$ = **30.484** kN

Racking panel summary

Racking panel no.1

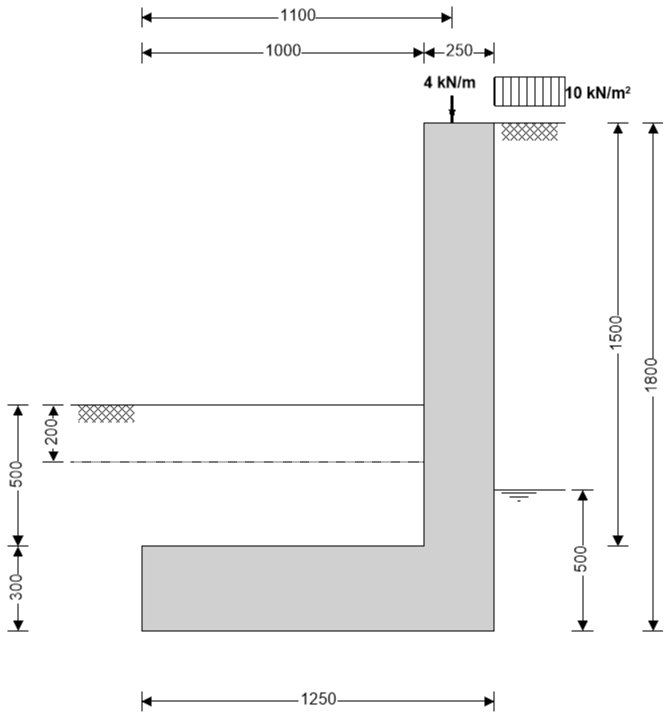
Panel dimensions 4000 mm long x 2400 mm high with studs no smaller than 38 mm × 72 mm
Loading 2.400 kN/m UDL to top rail
Primary sheathing 18.0 mm Plywood with 3.00 mm dia.nails at 150 mm centres
Secondary sheathing 12.5 mm Plasterboard with 3.50 mm dia.screws at 150 mm centres
Masonry cladding None

Racking panel no.2

Panel dimensions 5200 mm long x 2400 mm high with studs no smaller than 38 mm × 72 mm
Loading 2.400 kN/m UDL to top rail
Primary sheathing 18.0 mm Plywood with 3.00 mm dia.nails at 150 mm centres
Secondary sheathing 12.5 mm Plasterboard with 3.50 mm dia.screws at 150 mm centres
Masonry cladding None

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.07



Wall details

Retaining wall type	**Unpropped cantilever**
Height of retaining wall stem	h_{stem} = **1500** mm
Thickness of wall stem	t_{wall} = **250** mm
Length of toe	l_{toe} = **1000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1250** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **900** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **1800** mm
Depth of cover in front of wall	d_{cover} = **500** mm
Depth of unplanned excavation	d_{exc} = **200** mm
Height of ground water behind wall	h_{water} = **500** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **200** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **1800** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m³

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **18.6** deg

Base material details

Firm clay

Moist density γ_{mb} = **18.0** kN/m³

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

Allowable bearing pressure $P_{bearing}$ = **100** kN/m²

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.369}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

Surcharge load on plan Surcharge = **10.0** kN/m²

Applied vertical dead load on wall W_{dead} = **2.8** kN/m

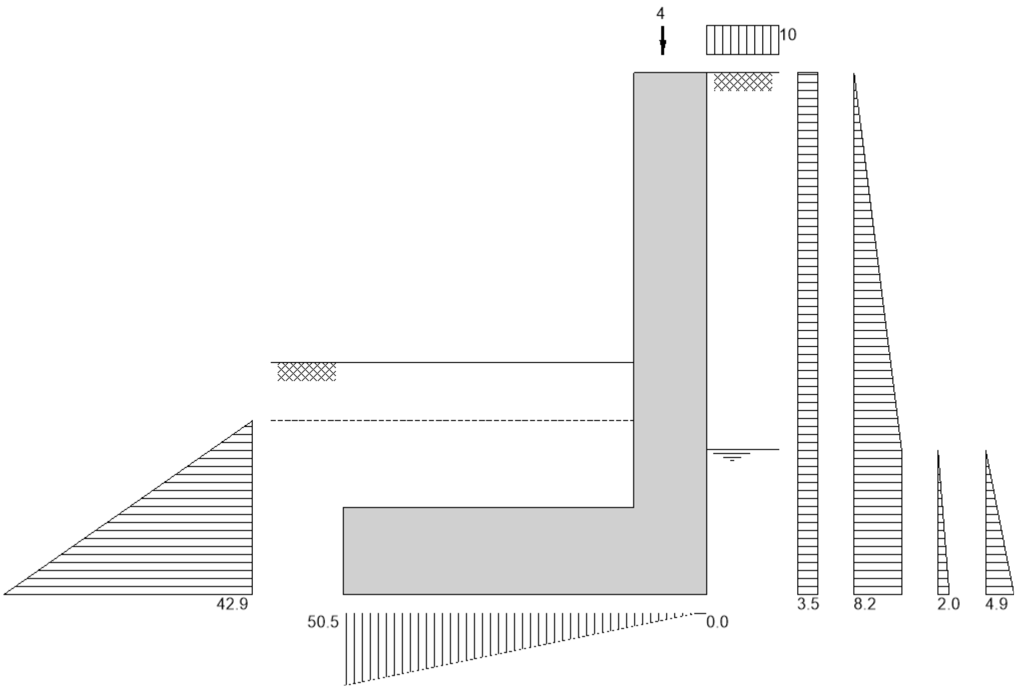
Applied vertical live load on wall W_{live} = **1.2** kN/m

Position of applied vertical load on wall l_{load} = **1100** mm

Applied horizontal dead load on wall F_{dead} = **0.0** kN/m

Applied horizontal live load on wall F_{live} = **0.0** kN/m

Height of applied horizontal load on wall h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m²

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **8.9** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **8.9** kN/m

Soil in front of wall $w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **9** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **4** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + w_p + W_v$ = **30.7** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times Surcharge \times h_{eff}$ = **6.3** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **5.3** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **4.1** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **0.5** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **1.2** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **17.4** kN/m

Calculate stability against sliding

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **12.9** kN/m

Resistance to sliding $F_{res} = F_p + (W_{total} - w_p - W_{live}) \times \tan(\delta_b)$ = **19.8** kN/m

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **5.7** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **5** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **1** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.1** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.2** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **11.9** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **10** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **5.5** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **3.1** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **18.6** kNm/m

Check stability against overturning

Total overturning moment M_{ot} = **11.9** kNm/m

Total restoring moment M_{rest} = **18.6** kNm/m

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Soil in front of wall $M_{p\_r} = w_p \times l_{toe} / 2$ = **4.5** kNm/m

Design vertical live load $M_{live} = W_{live} \times l_{load}$ = **1.3** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{p\_r} + M_{live}$ = **12.4** kNm/m

Total vertical reaction $R = W_{total}$ = **30.7** kN/m

Distance to reaction $x_{bar} = M_{total} / R$ = **405** mm

Eccentricity of reaction $e = abs((l_{base} / 2) - x_{bar})$ = **220** mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe} = R / (1.5 \times x_{bar})$ = **50.5** kN/m²

Bearing pressure at heel p_{heel} = 0 kN/m² = **0** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.07

Ultimate limit state load factors

Dead load factor	γ_{f_d} = **1.4**
Live load factor	γ_{f_l} = **1.6**
Earth and water pressure factor	γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem	$W_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **12.4** kN/m
Wall base	$W_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **12.4** kN/m
Soil in front of wall	$W_{p\_f} = \gamma_{f\_d} \times l_{toe} \times d_{cover} \times \gamma_{mb}$ = **12.6** kN/m
Applied vertical load	$W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **5.8** kN/m
Total vertical load	$W_{total\_f} = W_{wall\_f} + W_{base\_f} + W_{p\_f} + W_{v\_f}$ = **43.2** kN/m

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **17** kN/m
Moist backfill above water table	$F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **12.6** kN/m
Moist backfill below water table	$F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **9.7** kN/m
Saturated backfill	$F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **1.2** kN/m
Water	$F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **1.7** kN/m
Total horizontal load	$F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **42.1** kN/m
Passive resistance of soil in front of wall	$F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **18** kN/m

Factored overturning moments

Surcharge	$M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **15.3** kNm/m
Moist backfill above water table	$M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **11.7** kNm/m
Moist backfill below water table	$M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **2.4** kNm/m
Saturated backfill	$M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.2** kNm/m
Water	$M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.3** kNm/m
Total overturning moment	$M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **29.9** kNm/m

Restoring moments

Wall stem	$M_{wall\_f} = W_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **13.9** kNm/m
Wall base	$M_{base\_f} = W_{base\_f} \times l_{base} / 2$ = **7.7** kNm/m
Soil in front of wall	$M_{p\_r\_f} = W_{p\_f} \times l_{toe} / 2$ = **6.3** kNm/m
Design vertical load	$M_{v\_f} = W_{v\_f} \times l_{load}$ = **6.4** kNm/m
Total restoring moment	$M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{p\_r\_f} + M_{v\_f}$ = **34.4** kNm/m

Factored bearing pressure

Total moment for bearing	$M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **4.5** kNm/m
Total vertical reaction	$R_f = W_{total\_f}$ = **43.2** kN/m
Distance to reaction	$x_{bar\_f} = M_{total\_f} / R_f$ = **104** mm
Eccentricity of reaction	$e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **521** mm

Reaction acts outside middle third of base

Bearing pressure at toe	$p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **277.4** kN/m^2
Bearing pressure at heel	p_{heel_f} = 0 kN/m^2 = **0** kN/m^2
Rate of change of base reaction	rate = $p_{toe\_f} / (3 \times x_{bar\_f})$ = **890.23** kN/m^2/m
Bearing pressure at stem / toe	$p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **0** kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0\ \text{kN/m}^2) = \mathbf{0}\ \text{kN/m}^2$

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0\ \text{kN/m}^2) = \mathbf{0}\ \text{kN/m}^2$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{35}\ \text{N/mm}^2$

Characteristic strength of reinforcement $f_y = \mathbf{500}\ \text{N/mm}^2$

Base details

Minimum area of reinforcement $k = \mathbf{0.13}\ \%$

Cover to reinforcement in toe $c_{toe} = \mathbf{40}\ \text{mm}$

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2 = \mathbf{43.2}\ \text{kN/m}$

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{9.9}\ \text{kN/m}$

Shear from weight of soil $V_{toe\_wt\_soil} = W_{p\_f} - (\gamma_{f\_d} \times \gamma_m \times l_{toe} \times d_{exc}) = \mathbf{7.6}\ \text{kN/m}$

Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} - V_{toe\_wt\_soil} = \mathbf{25.7}\ \text{kN/m}$

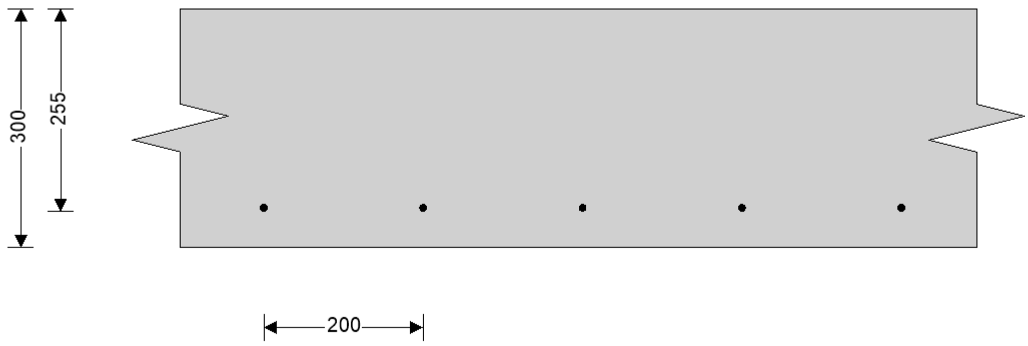
Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2 = \mathbf{44.1}\ \text{kNm/m}$

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{6.3}\ \text{kNm/m}$

Moment from weight of soil $M_{toe\_wt\_soil} = (W_{p\_f} - (\gamma_{f\_d} \times \gamma_m \times l_{toe} \times d_{exc})) \times (l_{toe} + t_{wall}) / 2 = \mathbf{4.7}\ \text{kNm/m}$

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} - M_{toe\_wt\_soil} = \mathbf{33.1}\ \text{kNm/m}$



Check toe in bending

Width of toe $b = \mathbf{1000}\ \text{mm/m}$

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{255.0}\ \text{mm}$

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.015}$

Compression reinforcement is not required

Lever arm $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} = \mathbf{242}\ \text{mm}$

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{314}\ \text{mm}^2/\text{m}$

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390}\ \text{mm}^2/\text{m}$

Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{390}\ \text{mm}^2/\text{m}$

Reinforcement provided **A393 mesh**

Area of reinforcement provided $A_{s\_toe\_prov} = \mathbf{393}\ \text{mm}^2/\text{m}$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.101}\ \text{N/mm}^2$

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{}(f_{cu} / 1\ N/mm^2), 5) \times 1\ N/mm^2 =$ **4.733** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} =$ **0.424** N/mm²

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} =$ **35** N/mm²

Characteristic strength of reinforcement $f_y =$ **500** N/mm²

Wall details

Minimum area of reinforcement k = **0.13** %

Cover to reinforcement in stem $c_{stem} =$ **40** mm

Cover to reinforcement in wall $c_{wall} =$ **40** mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times (h_{eff} - t_{base} - d_{ds}) =$ **14.2** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 =$ **12.6** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} =$ **3.9** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 =$ **0.2** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 =$ **0.3** kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} =$ **31.1** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 =$ **12.7** kNm/m

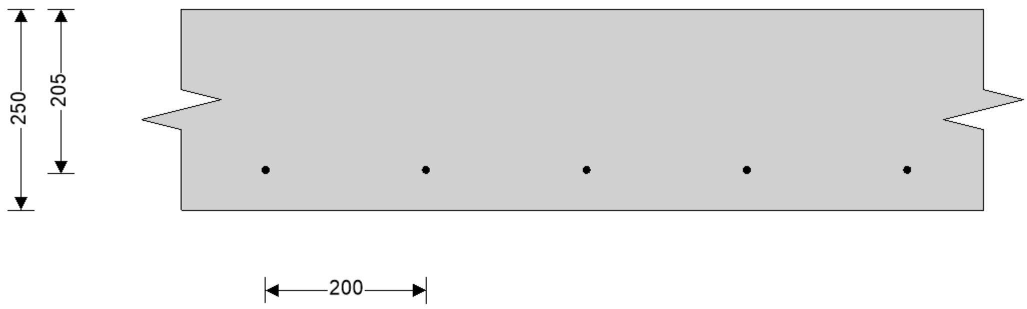
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 =$ **9.8** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2 =$ **0.4** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3 =$ **0** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3 =$ **0** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} =$ **23** kNm/m



Check wall stem in bending

Width of wall stem b = **1000** mm/m

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **205.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.016**

Compression reinforcement is not required

Lever arm $z_{stem} = \min(0.5 + \sqrt{}(0.25 - (\min(K_{stem}, 0.225) / 0.9)), 0.95) \times d_{stem}$

$z_{stem} =$ **195** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **272** mm²/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **325** mm²/m

Area of tension reinforcement required $A_{s\_stem\_req} = Max(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **325** mm²/m

Reinforcement provided **A393 mesh**

Area of reinforcement provided $A_{s\_stem\_prov} =$ **393** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.151** N/mm²

Allowable shear stress $v_{adm} = min(0.8 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5) \times 1 N/mm^2 =$ **4.733** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_stem} =$ **0.482** N/mm²

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Basic span/effective depth ratio $ratio_{bas} =$ **7**

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov}) =$ **275.9** N/mm²

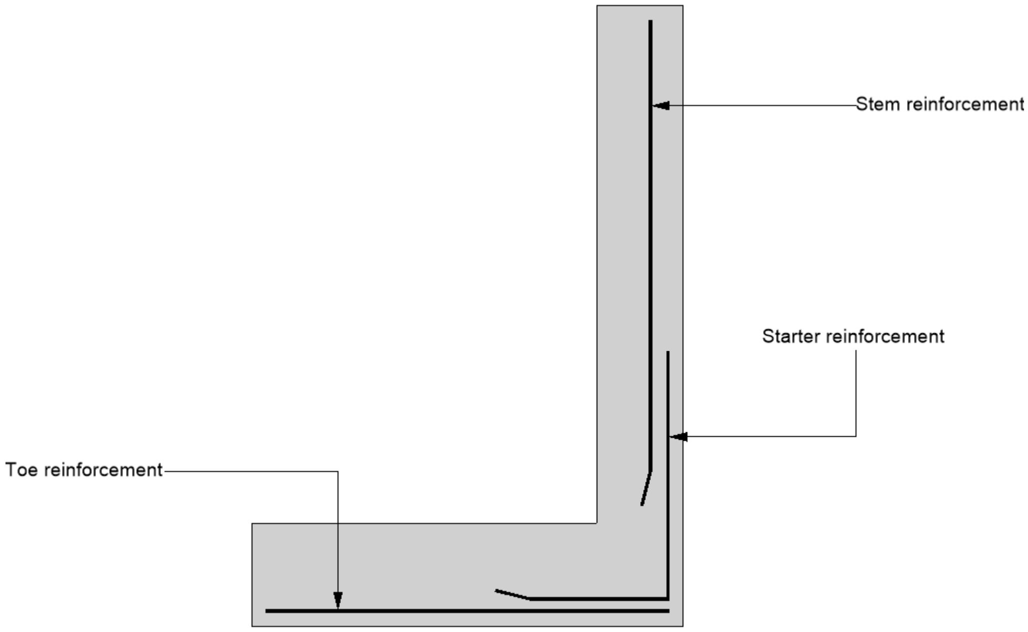
Modification factor $factor_{tens} = min(0.55 + (477 N/mm^2 - f_s)/(120 \times (0.9 N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2) =$ **1.71**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens} =$ **11.96**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem} =$ **7.32**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe mesh - A393 - (393 mm^2/m)

Stem mesh - A393 - (393 mm^2/m)

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

TWO WAY SPANNING SLAB DEFINITION – SIMPLY SUPPORTED

Overall depth of slab h = **200** mm

Outer sagging steel

Cover to outer tension reinforcement resisting sagging c_{sag} = **35** mm

Trial bar diameter D_{tryx} = **10** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **160** mm

Inner sagging steel

Trial bar diameter D_{tryy} = **10** mm

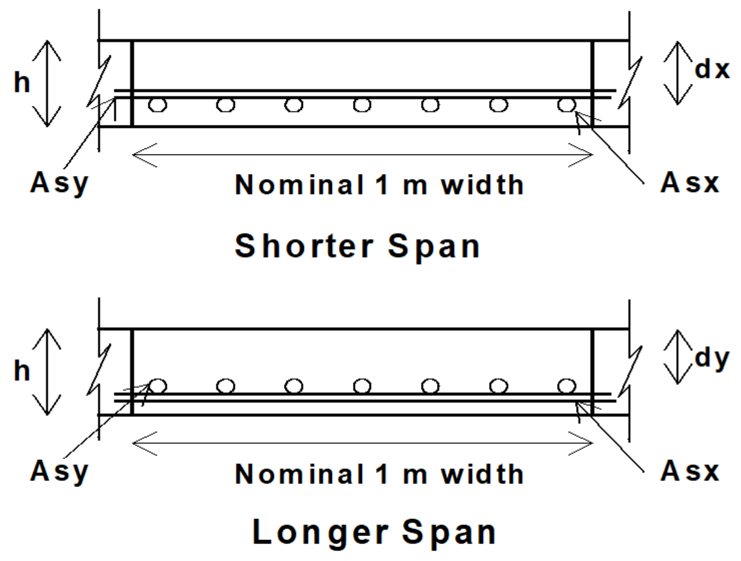
Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2$ = **150** mm

Materials

Characteristic strength of reinforcement f_y = **500** N/mm^2

Characteristic strength of concrete f_{cu} = **35** N/mm^2



Two-way spanning slab (simple)

MAXIMUM DESIGN MOMENTS

Length of shorter side of slab l_x = **5.000** m

Length of longer side of slab l_y = **5.000** m

Design ultimate load per unit area n_s = **3.0** kN/m^2

Moment coefficients

$\alpha_{sx} = (l_y / l_x)^4 / (8 \times (1+(l_y / l_x)^4)) = \mathbf{0.063}$

$\alpha_{sy} = (l_y / l_x)^2 / (8 \times (1+(l_y / l_x)^4)) = \mathbf{0.063}$

Maximum moments per unit width - simply supported slabs

$m_{sx} = \alpha_{sx} \times n_s \times l_x^2 = \mathbf{4.7}$ kNm/m

$m_{sy} = \alpha_{sy} \times n_s \times l_x^2 = \mathbf{4.7}$ kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) $m_{sx} = \mathbf{4.7}$ kNm/m

Moment Redistribution Factor $\beta_{bx} = \mathbf{1.0}$

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu}) = \mathbf{0.005}$

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2)) = \mathbf{0.156}$

Outer compression steel not required to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel (cl. 3.5.4)

Design sagging moment (per m width of slab) $m_{sy} = \mathbf{4.7}$ kNm/m

Moment Redistribution Factor $\beta_{by} = \mathbf{1.0}$

Area of reinforcement required

$K_y = abs(m_{sy}) / (d_y^2 \times f_{cu}) = \mathbf{0.006}$

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2)) = \mathbf{0.156}$

Inner compression steel not required to resist sagging

Two way Spanning Slab requiring tension steel only - mesh (sagging)

$z_x = \min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)}))) = \mathbf{152}$ mm

Neutral axis depth $x_x = (d_x - z_x) / 0.45 = \mathbf{18}$ mm

$z_y = \min ((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)}))) = \mathbf{142}$ mm

Neutral axis depth $x_y = (d_y - z_y) / 0.45 = \mathbf{17}$ mm

Area of outer tension steel required

$A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x) = \mathbf{71}$ mm²/m

Area of inner tension steel required

$A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y) = \mathbf{76}$ mm²/m

Tension steel

Provide A393 Mesh tension steel resisting sagging

$A_{sx\_prov} = A_{sl} = \mathbf{393}$ mm²/m $A_{sy\_prov} = A_{st} = \mathbf{393}$ mm²/m

$D_x = d_{sl} =$ **10** mm $D_y = d_{st} =$ **10** mm

Area of tension steel provided sufficient to resist sagging

Check min and max areas of steel resisting sagging

Total area of concrete $A_c = h =$ **200000** mm²/m

Minimum % reinforcement k = **0.13** %

$A_{st\_min} = k \times A_c =$ **260** mm²/m

$A_{st\_max} = 4\ \% \times A_c =$ **8000** mm²/m

Steel defined:

Outer steel resisting sagging $A_{sx\_prov} =$ **393** mm²/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging $A_{sy\_prov} =$ **393** mm²/m

Area of inner steel provided (sagging) OK

SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)

Outer tension steel resisting sagging moments

Depth to tension steel from compression face $d_x =$ **160** mm

Area of tension reinforcement provided (per m width of slab) $A_{sx\_prov} =$ **393** mm²/m

Design ultimate shear force (per m width of slab) $V_x =$ **0** kN/m

Characteristic strength of concrete $f_{cu} =$ **35** N/mm²

Applied shear stress

$v_x = V_x / d_x =$ **0.00** N/mm²

Check shear stress to clause 3.5.5.2

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **4.73** N/mm²

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

$f_{cu\_ratio} = \text{if}(f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.400**

$v_{cx} = 0.79\ N/mm^2 \times \min(3, 100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67, (400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cx} =$ **0.56** N/mm²

Applied shear stress

$v_x =$ **0.00** N/mm²

No shear reinforcement required

SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)

Inner tension steel resisting sagging moments

Depth to tension steel from compression face $d_y =$ **150** mm

Area of tension reinforcement provided (per m width of slab) $A_{sy\_prov} =$ **393** mm²/m

Design ultimate shear force (per m width of slab) V_y = **0** kN/m

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Applied shear stress

$v_y = V_y / d_y$ = **0.00** N/mm^2

Check shear stress to clause 3.5.5.2

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **4.73** N/mm^2

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm^2 , 40/25 , f_{cu}/(25 N/mm^2)) = **1.400**

$v_{cy} = 0.79\ N/mm^2 \times \min(3,100 \times A_{sy\_prov} / d_y)^{1/3} \times \max(0.67,(400\ mm) / d_y)^{1/4} / 1.25 \times f_{cu\_ratio}^{1/3}$

v_{cy} = **0.58** N/mm^2

Applied shear stress

v_y = **0.00** N/mm^2

No shear reinforcement required

<u>SHEAR PERIMETERS FOR A CIRCULAR CONCENTRATED LOAD (CL 3.7.7)</u>

Diameter of loaded circle D_L=**300** mm

Depth to tension steel d_x = **160** mm

Dimension from edge of load to shear perimeter $l_p = k_p \times d_x$ = **240** mm where k_p = **1.50**

For punching shear cases not affected by free edges or holes:

Total length of inner perimeter at edge of loaded area $u_{0\_gen} = \pi \times D_L$ = **942** mm

Total length of outer perimeter at l_p from loaded area $u_{gen} = 4 \times D_L + 8 \times l_p$ = **3120** mm

<u>PUNCHING SHEAR AT CONCENTRATED LOADS (CL 3.7.7)</u>

Tension steel resisting sagging

Total length of inner perimeter at edge of loaded area u_0 = **942** mm

Total length of outer perimeter at dimension l_p from loaded area u = **3120** mm

Depth to outer steel d_x = **160** mm

Depth to inner steel d_y = **150** mm

Average depth to "tension" steel $d_{av} = (d_x + d_y)/2$ = **155.0** mm

Area of outer steel per m effective through the perimeter A_{sx_prov} = **393** mm^2 /m

Area of inner steel per m effective through the perimeter A_{sy_prov} = **393** mm^2 /m

Max shear effective across either perimeter under consideration V_p = **34** kN

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Applied shear stress

Stress around loaded area $v_{max} = V_p / (u_0 \times d_{av})$ = **0.235** N/mm²

Stress around perimeter $v = V_p / (u \times d_{av})$ = **0.071** N/mm²

Check shear stress to clause 3.7.7.2

$v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × $\sqrt{}$(f_{cu}), 5 N/mm²) = **4.733** N/mm²

Shear stress - OK

Shear stresses to clause 3.7.7.4

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm² , 40/25 , f_{cu}/(25 N/mm²)) = **1.400**

Effective steel area for shear strength determination: A_{s_eff} =**393** mm²/m

v_c = 0.79 N/mm² × min(3, 100×(A_{s_eff} / d_{av}))$^{1/3}$ × max(0.67, (400 mm / d_{av})$^{1/4}$) / 1.25 × $f_{cu\_ratio}^{1/3}$

v_c = **0.567** N/mm²

No shear reinforcement required

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length l_x = **5.000** m

Design ultimate moment in shorter span per m width m_{sx} = **5** kNm/m

Depth to outer tension steel d_x = **160** mm

Tension steel

Area of outer tension reinforcement provided A_{sx_prov} = **393** mm²/m

Area of tension reinforcement required A_{sx_req} = **71** mm²/m

Moment Redistribution Factor β_{bx} = **1.00**

Modification Factors

Basic span / effective depth ratio (Table 3.9) $ratio_{span\_depth}$ = **20**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **60.2** N/mm²

$factor_{tens}$ = min (2 , 0.55 + (477 N/mm² - f_s) / (120 × (0.9 N/mm² + m_{sx} / d_x^2))) = **2.000**

Calculate Maximum Span

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **6.40** m

Check the actual beam span

Actual span/depth ratio l_x / d_x = **31.25**

Span depth limit $ratio_{span\_depth} \times factor_{tens}$ = **40.00**

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness h = **200** mm

Effective depth to bottom outer tension reinforcement d_x = **160.0** mm

Diameter of tension reinforcement D_x = **10** mm

Diameter of links L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **35.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **35.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** mm

Cover over steel resisting sagging OK

APPENDIX C

Arboricultural Investigation Report from Parsons Tree Care

PARSONS TREE CARE

Arboricultural Report

29th September 2020

Client: Ariel Klien
Site: 7 Rosecroft Avenue, London NW3 7QA

Trial dig - BS5837:2012: Trees in relation to design, demolition and construction – Recommendations.

With regards to the proposed installation of a garden room at the above site address, trial pits have been dug to ascertain the rooting activity in the rear garden.

This revision is to be read in conjunction with the initial report dated 6th June 2019 alongside the tree protection plan and tree constraints plan (*TPP 7 Rosecroft Av & TCP 7 Rosecroft Av)* as well the architects revised drawings A/02/501, which plot root findings on the updated scale plan.

Trial pit 1: Dig on south boundary uncovered x3 <50mm roots growing from T4 and X1 <100mm root growing from T6 green beech. See updated drawings (A/02/501) for locations and photos for illustrations of these roots.
Pits were hand dug to ascertain locations for pads to support and cantilever the garden room floor structure. All four pad locations are achievable without severing any primary roots found. An in significant quantity of fibrous root system will need to be removed when excavating soil for the pads.

Photo 1:

Photo 2:

Photo 1 illustrates x3 roots with a diameter less than 50mm, whilst one slightly larger root measured at 100mm grows from T6 as illustrated in photo 2.

Despite a small loss of fibrous root system from T4 and T6 the beech trees should not be impacted by the proposed installation of foundation pads to support the floor structure of the garden room.

***Amended proposal along rear boundary: to install a climbing wall. Refer to drawing no. A/02/501 Constraints are the roots of T7-9 inclusive.

Trial pit 2: A trial trench has been dug to a depth of 700mm. Findings uncovered no structural or woody roots. Fibrous root system is present from previously removed vegetation such as ivy. The rear boundary wall drops down by another 200mm to meet 15 Hollycroft Avenue property where the three Lime trees grow. It would seem the roots of these lime trees haven't grown under the presumed 400mm wall foundation and up into the mass of soil in the rear garden of 7 Rosecroft Avenue. See photos below for clarification.

Photo 1:

Photo 2:

Conclusion:

Meeting the client and architect on site at the time of the trail dig enabled us to formulate a plan to mitigate excavation in the area adjacent to the row of Beech trees by way of sinking 4 concrete pads in locations sympathetic to the rooting activity of the Beech trees and supporting the floor base on these pads.

email: frankparsons@me.com
website: parsonstreecare.uk
Parsons Tree Care Ltd

mobile: 07791 652 889
Company number 10337138
Registered office: 2 Accommodation Rd
London NW11 8ED

PARSONS TREE CARE

With regards to the roots of the Lime trees in 15 Hollycroft. The findings along the rear boundary wall (south west boundary) were not surprising as tree roots are hydrotropic and they is plenty of available soil / lawn in the rear garden in which they grow. Therefore excavation in this vicinity would be permissible not encroaching more than 20% into the RPA of T6. Heavy machinery is not permitted or accessible so excavation would be by hand dig and I am in touch with the contractor should they encounter any substantial rooting activity along the rear boundary where the proposed climbing wall is to be installed.
I do not foresee a detrimental impact on trees numbered 1-9 during the project so long as the tree protection plan is to be followed and communication is maintained at this current level.

Yours sincerely,

Frank Parsons
RFS certificate in Arboriculture
AA Technicians certificate in Arboriculture
(Level 4 Diploma in Arboriculture)

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Parsons Tree Care Ltd

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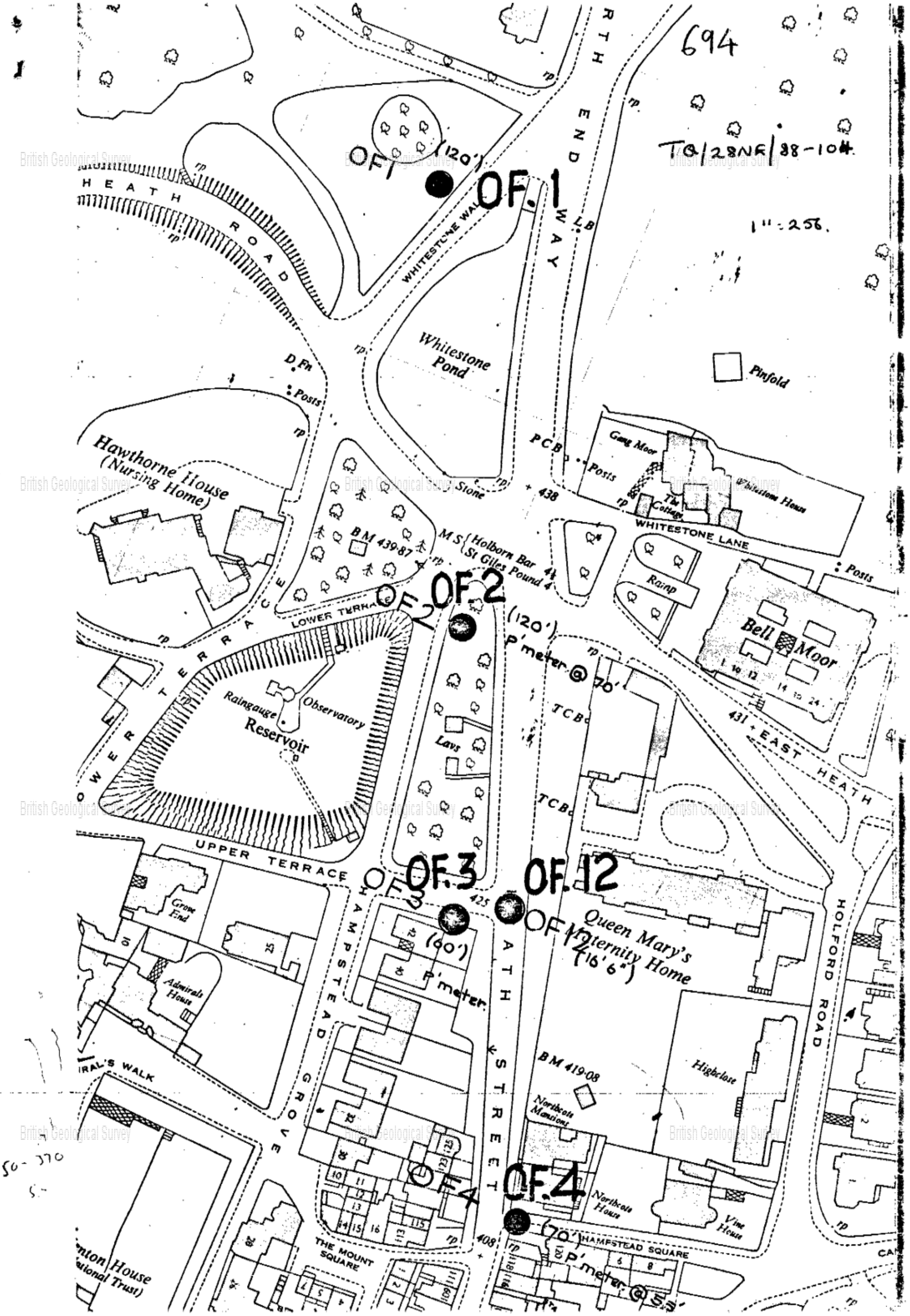
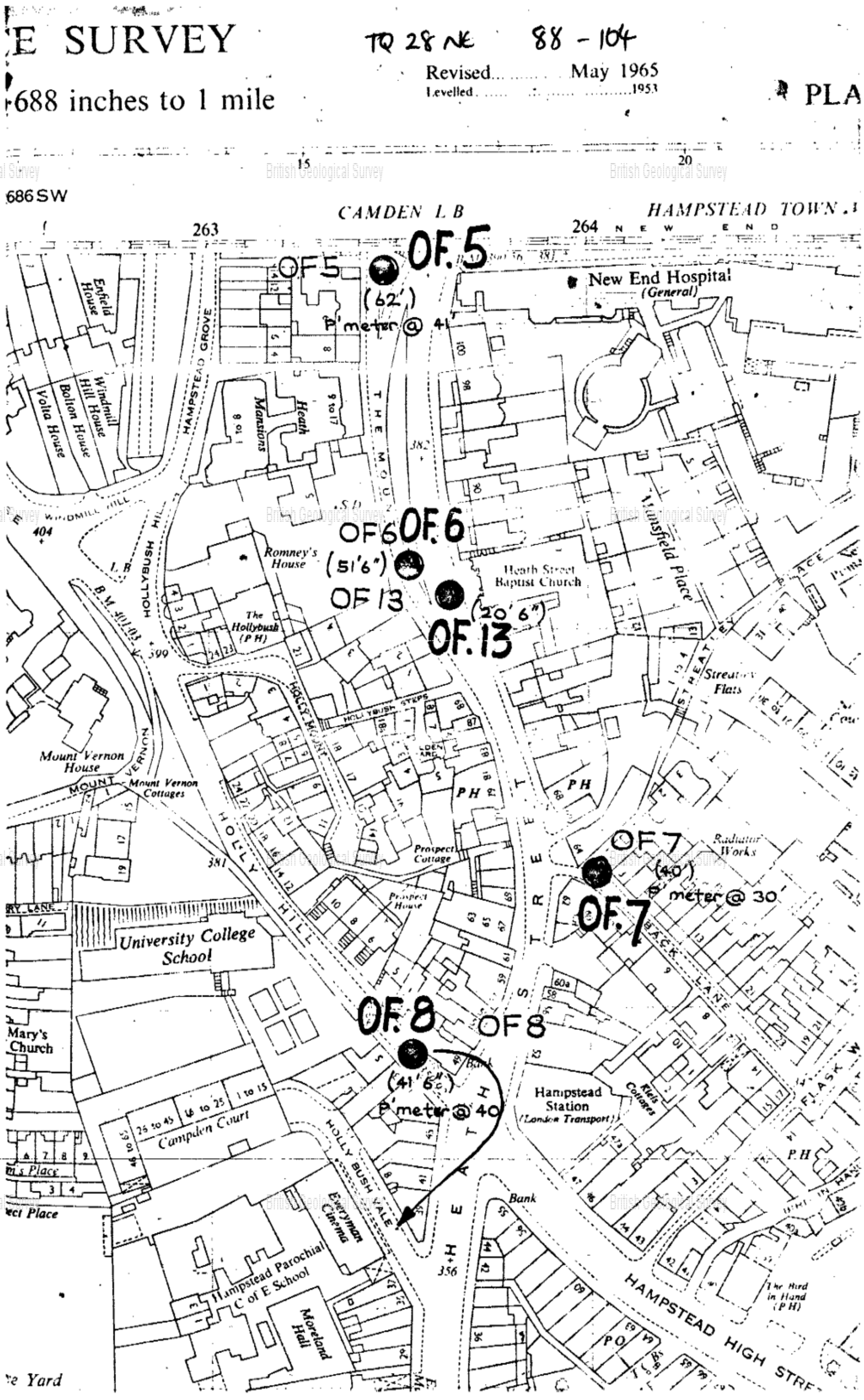
APPENDIX D

Borehole Records


694
TQ/28NE/88-104
1" = 256.
OF1 (120')
OF.1
OF 2
OF.2
(120')
P'meter @ 70'
OF.3
OF3
(60')
P'meter
OF.12
OF12
(16'6")
OF.4
OF4
(70')
P'meter @ 5
HEATH ROAD
WHITESTONE WALK
NORTH END WAY
Whitestone Pond
Pinfold
D Fn
Posts
Hawthorne House (Nursing Home)
Stone
438
PCB
Gang Moor
The Cottage
Whitestone House
WHITESTONE LANE
BM 439·87
MS {Holborn Bar 4 / St Giles Pound 4}
Ramp
Bell Moor
431
EAST HEATH
LOWER TERRACE
Raingauge
Observatory
Reservoir
Lavs
TCB
UPPER TERRACE
HAMPSTEAD GROVE
HEATH STREET
425
Grove End
Admirals House
Queen Mary's Maternity Home
HOLFORD ROAD
ADMIRAL'S WALK
BM 419·08
Highclose
Northcote Mansions
Northcote House
Vine House
HAMPSTEAD SQUARE
THE MOUNT SQUARE
408
Fenton House (National Trust)
50-370


E SURVEY
688 inches to 1 mile
TQ 28 NE 88 - 104
Revised May 1965
Levelled 1953
PLA
15
20
686 SW
CAMDEN L B
HAMPSTEAD TOWN
263
264
NEW END
OF5
OF.5
(62')
P'meter @ 41
New End Hospital
(General)
Enfield House
Windmill Hill House
Bolton House
Volta House
HAMPSTEAD GROVE
Heath Mansions
THE MOUNT
382
WINDMILL HILL
404
OF6
OF.6
(51'6")
Romney's House
OF 13
(20' 6")
OF.13
Heath Street Baptist Church
Mansfield Place
HOLLYBUSH HILL
The Hollybush (P H)
399
HOLLYBUSH STEPS
Mount Vernon House
MOUNT VERNON
Mount Vernon Cottages
HOLLY MOUNT
HOLLY HILL
381
Prospect Cottage
Prospect House
P H
HEATH STREET
Streatley Flats
OF7
(40')
P'meter @ 30
OF.7
Radiator Works
BACK LANE
University College School
OF.8
OF8
(41' 6")
P'meter @ 40
Mary's Church
Campden Court
HOLLY BUSH VALE
Hampstead Station
(London Transport)
Bank
Everyman Cinema
Hampstead Parochial C of E School
Moreland Hall
356
FLASK WALK
HAMPSTEAD HIGH STREET
The Bird in Hand (P H)
P O
P H
Yard

LT3 Junction of Hampstead High Street and Heath Street.
Height 344 ft. O.D.

	Thickness (ft)	Depth (ft)
Concrete	2¼	
Rubble, bricks & stones	6¾	2¼
Brown loamy clay	5¾	
Blue clay	12½	14¾
Blue sandy clay	50¾	27¼
	78	

London Clay **NO KEY PLAN**
(Station tunnel intrados 177 ft. below ground level).

BOREHOLES SUNK AT BRANCH HILL LODGE

B/G (1951) TQ/28NE/101
Height 381. 99 ft. O.D. 2610.8604.

	Thickness (ft)	Depth (ft)	
Made ground	4		
Brown sandy clay	4	4	ALL
Loamy sand	1	8	C.B.
Brown sandy clay	6	9	
Grey sandy clay	3	15	
Brown sandy clay	12	18	
Blue sandy clay	6	30	
	36		

B/H (1951)
Height 427. 44 ft. O.D. TQ/28NE/102.
2605.8608.

	Thickness (ft)	Depth (ft)	
Soil and stones	6		
Brown sandy clay	4	6	
Sandy clay and pebbles	6	10	CB.
Brown sandy clay	5½	16	
Loamy sand	8½	21½	
Brown sandy clay	7	30	
White loamy sand	3	37	

GROUND ENGiNEERiNG

Geo-Environmental Specialists
01733 566566

Site: 378 FINCHLEY ROAD, LONDON NW3

BOREHOLE BH1

Date: 06/06/06

Hole Size: 150mm dia to 20.00m

Ground Level: 83.34m. O.D.

Depth m	Type	Blows	(Date) Casing	Description of Strata	Depth m	O.D. Level m
				MADE GROUND - CONCRETE	0.20	83.14
0.20-0.70	B1			MADE GROUND - Firm, friable, dark brown/brown/grey mottled slightly gravelly, sandy CLAY with occasional brick, concrete, coal and ash fragments	1.10	82.24
1.00-1.50	B2					
1.15-1.45	S	N12	0.90	Stiff brown/orange brown/light grey mottled CLAY with occasional selenite crystals. Becoming fissured below 2.50m		
1.75	D1					
2.00-2.40	U1	38	1.20 ▼s			
2.03	W1					
2.45	D2			(WEATHERED LONDON CLAY)		
2.75	D3					
3.00-3.40	U2	48	1.20			
3.45	D4				3.60	79.74
3.75	D5			Very stiff, closely fissured to stiff, brown/orange brown CLAY with occasional selenite crystals		
4.00-4.40	U3	55	1.20			
4.45	D6			(WEATHERED LONDON CLAY)		
4.75	D7					
5.00-5.40	U4	55	1.20			
5.45	D8				5.50	77.84
				Stiff, becoming very stiff below 7.00m, closely fissured, dark grey CLAY with occasional silt and fine sand seams		
6.00	D9					
6.50-6.90	U5	60	1.20	(LONDON CLAY)		
6.95	D10					
7.50	D11					
8.00-8.40	U6	62	1.20			
8.45	D12					
9.00	D13					
9.50-9.90	U7	70	1.20			
9.95	D14				10.00	73.34

Inst.: BENEATH INSTALLATION (below 4.00m)

REMARKS
1. Breaking out concrete from 0.00m to 0.20m for 0.50 hours
2. Excavating a pit from 0.20m to 1.00m for 1 hour
3. Borehole cased to 1.20m depth
4. Fibrous live roots observed to 1.75m depth
5. Standpipe installed to 4.00m depth

Project No 10575
Scale 1:50
Page 1/2

KEY
D - Disturbed Sample
B - Bulk Sample
U - Undisturbed Sample
W - Water Sample
S/C - SPT Spoon/Cone
▽ Water Strike
▼ Water Rise
N - SPT Blows for 0.3m
• - Blows for quoted penetration
V - Vane Shear Test Cohesion ( ) kPa
▼c Level on completion
c▼w Level casing withdrawn
▼s Standpipe Level

Groundwater Strikes

No	Struck	Rose to	Rate	Cased	Sealed

Groundwater Observations

Date	Hole	Casing	Water
06/06/06	20.00	1.20	dry
06/06/06	20.00	0.00	dry
20/07/06	4.00	0.00	2.03

GROUND ENGINEERING
Geo-Environmental Specialists
01733 566566

Site: 378 FINCHLEY ROAD, LONDON NW3

BOREHOLE BH1

Date: 06/06/06

Hole Size: 150mm dia to 20.00m

Ground Level: 83.34m. O.D.

Samples and in-situ Tests: Depth m	Type	Blows	(Date) Casing	Inst.	Description of Strata	Legend	Depth m	O.D. Level m
					Very stiff, closely fissured to stiff, dark brown/ dark grey CLAY with occasional light brown silt and fine sand seams up to 6mm thick. Rare bivalve shell fragments at 15.00m		10.00	73.34
10.50	D15			BENEATH INSTALLATION				
11.00-11.40	U8	78	1.20					
11.45	D16							
12.00	D17							
12.50-12.90	U9	85	1.20					
12.95	D18							
13.50	D19							
14.00-14.40	U10	90	1.20					
14.45	D20							
15.00	D21				(LONDON CLAY)			
15.50-15.90	U11	90	1.20					
15.95	D22							
16.50	D23							
17.00-17.40	U12	95	1.20					
17.45	D24							
18.00	D25							
18.50-18.90	U13	100	1.20					
18.95	D26							
19.55-19.95	U14	100	1.20					
20.00	D27						20.00	63.34

REMARKS

Borehole completed at 20.00m depth

Project No 10575

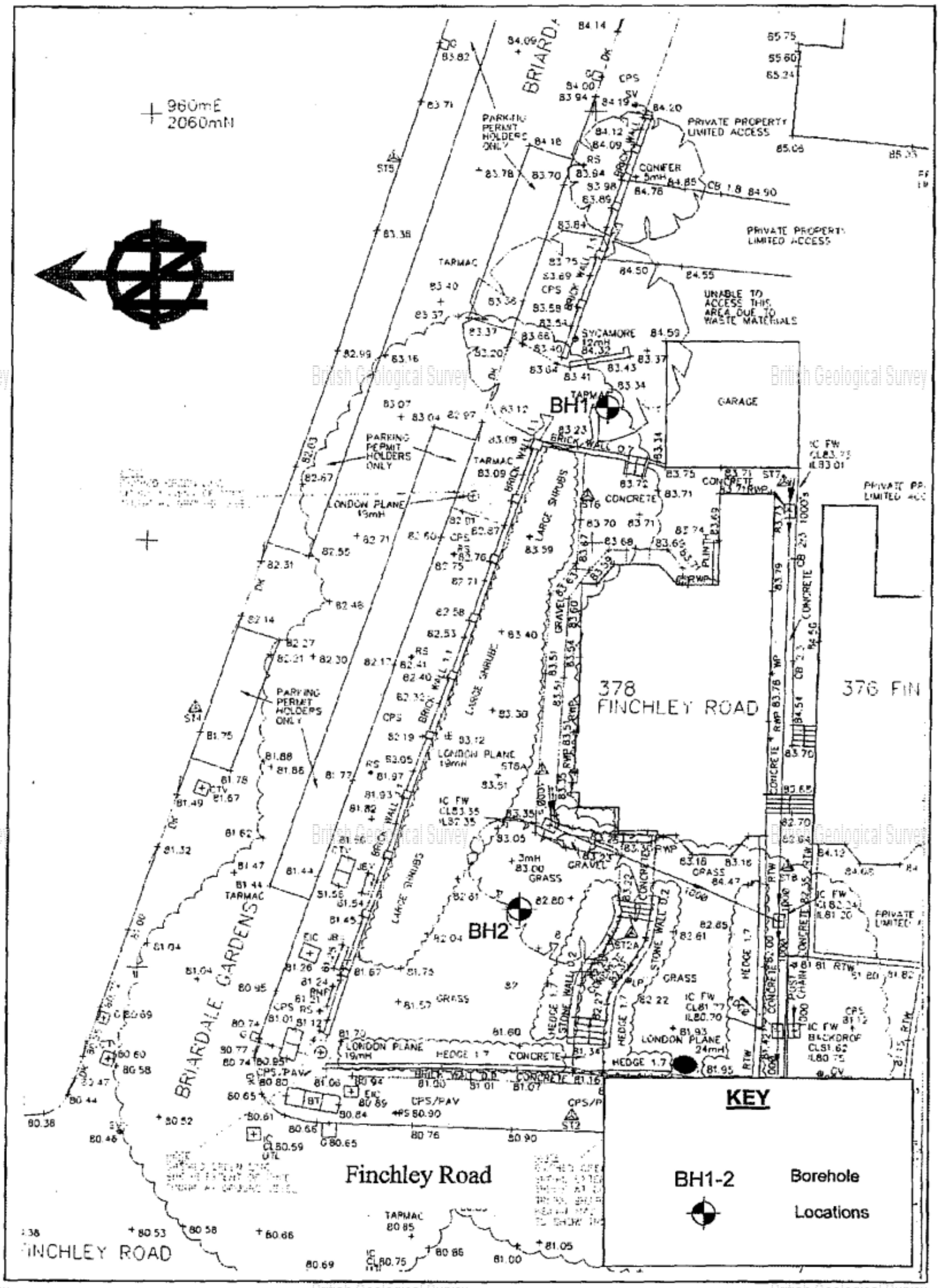
Scale 1:50

Page 2/2

KEY
D - Disturbed Sample
B - Bulk Sample
U - Undisturbed Sample
W - Water Sample
S/C - SPT Spoon/Cone
∇ Water Strike
▼ Water Rise
N - SPT Blows for 0.3m
• - Blows for quoted penetration
V - Vane Shear Test Cohesion ( ) kPa
▼c Level on completion
c.▼w Level casing withdrawn
▼s Standpipe Level

Groundwater Strikes						Groundwater Observations			
			Depth m				Depth m		
No	Struck	Rose to	Rate	Cased	Sealed	Date	Hole	Casing	Water

Borehole Location Plan



NOT TO SCALE

Project : 378 Finchley Road, NW3

Client : Mr Manoj Shah

GROUND ENGiNEERiNG

Peterborough Tel : 01733 566566

Project No.

C10575