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Mr Faisal Khan

175 Arlington Road, London NW1 7EY

Basement Impact Assessment

September 2023 Job No: 1569

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Reference	1569	Revision	0	Issue Date	Sept. 2023

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1. NON-TECHNICAL SUMMARY

Lim Engineering Ltd has been instructed by Mr Faisal Khan ("the Client") to undertake a Basement Impact Assessment (BIA) for the proposed development at 175 Arlington Road, London NW1 7EY. A summary of the BIA report is provided below.

1. The site is located at 175 Arlington Road, London NW1 7EY. The site location is presented in Figure 1.

2. The site is rectangular in shape measuring 18.3 long and 4.7 wide. The site is currently occupied by a two-storey detached building which includes , a ground level, a first-floor level and a second floor level. The property has a rear garden.

3. The property on the site shares a party wall with the property at 173 Arlington Road along the south site boundary and 177 Arlington Road along the north site boundary.

4. The proposed development at 175 Arlington Road involves the basement to the rear of the property and across the full width of the site. The basement will be constructed using underpin footings along the majority of the basement footprint.

5. A desk top review of the site has been undertaken. The findings have been used to carry out a screening and scoping assessment to identify key areas to investigate and assess the requirements for further investigation and assessment.

6. Reliance has been granted to use the findings from the site investigation carried out in the neighbouring site at 220 Arlington Road. The findings from this site investigation have been used to establish the ground and groundwater conditions in the vicinity of the site and derive geotechnical design parameters. The ground conditions comprise up to 0.7m of Made Ground overlying London Clay Formation to a proven depth of 10m below ground level.

7. The groundwater encountered is likely to be perched groundwater and of limited volume. The soils on site are generally cohesive and substantial groundwater ingress during excavation is not anticipated.

8. A Ground Movement Assessment (GMA) has been carried out to assess the impact of the proposed development on the neighbouring structures and infrastructure. The assessment predicts that the resulting damage category can be controlled to within Category 0 'negligible' damage for the neighbouring structures, assuming a good standard of workmanship and installing the underpins in a hit-and-miss construction sequence.

9. A structural monitoring strategy is recommended to control the works and impact to the neighbouring structures. Prior to construction commencing, baseline survey readings should be established, and a condition survey should be undertaken of adjacent buildings with any cracks and defects recorded and monitored during construction stages. A mitigation strategy should be prepared in advance of construction and implemented, should unacceptable movement occur.

10. The BIA has identified no significant potential hydrogeological impacts and no impact to the wider hydrogeological environment.

13. The BIA has identified that the site is not in an area at risk of flooding and does not affect surface water flow and flooding.

2. INTRODUCTION

It is proposed to develop 175 Arlington Road, London NW1 7EY in the London Borough of Camden (LBC). The proposed development comprises constructing basement under the house and the part of the rear garde. Lim Engineering Ltd has been instructed to undertake a Basement Impact Assessment (BIA), including a detailed ground movement analysis for the proposed development to determine its potential impact on the nearby structures, surface water runoff and groundwater flow.

The London Borough of Camden's guidance document[2], requires a Basement Impact Assessment (BIA) to be undertaken for new basements in the Borough and sets out 5 stages for a BIA to "enable the Council to assess whether any predicted damage to neighbouring properties and the water environment is acceptable or can be satisfactorily ameliorated by the developer". The report comprises the following elements:

- Desk Study
- Screening
- Scoping
- Site investigation
- Ground Movement Assessment
- Impact assessment
- Monitoring Strategy

This report is intended to address the stages presented above. It identifies the key issues relating to land stability, hydrogeology and hydrology as part of the screening process and includes a review and interpretation of existing site investigation data to establish a conceptual site model.

2.1 Sources of Information

The following baseline data has been referenced to complete the BIA in relation to the proposed development:

- Camden geological, hydrogeological and hydrological study[3]
- Proposed development drawings (see Appendix)
- Geological mapping[4] and historical borehole records (see Appendix)
- London Borough of Camden Strategic Flood Risk Assessment[5] (SFRA)
- Environment Flood Risk Map[6]
- Lost Rivers of London[8]
- 175 Arlington Road, London NW1 7EY – Flood Risk Assessment and Drainage Report[9]

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[2] Camden Borough of Camden. (2018). Camden Planning Guidance – Basements. March 2018.

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[3] Ove Arup and Partners. (2010) Camden Geological, Hydrogeological and Hydrological Study: Guidance for subterranean development. London Borough of Camden.
[4] British Geological Society. (2006). Geological Survey of England and Wales 1:63,360/1:50,000 geological map series, New Series, Sheet 256, North London, Bedrock and Superficial, 1:50,000.
[5] URS (2014). London Borough of Camden SFRA – Strategic Flood Risk Assessment. July 2014.
[6] Environment Agency. 2020. Flood Map for Planning. [ONLINE] Available at https://flood-map-for-planning.service.gov.uk/. [Accessed 21 May 2020].
[7] London Topographical Society (2005). *Bomb Damage Maps 1939-1945*. The London City Council.
[8] Barton, N. (1983) The Lost Rivers of London Hertfordshire Historical Publications
[9] Flood Risk Assessment and Drainage Report.

2.2 Site Location

The site is located at No. 175 Arlington Road, London NW1 7EY. The National Grid Reference for the approximate centre of the site is 513219N, 00839W. The site location is shown in Figure 1.

2.3 Site Layout

The site is rectangular in shape and currently comprises a two detached residential property, which includes single story rear addition. The site has rear garden areas.
The existing site is approximately 18.3m long and 4.7m wide. The garden at the rear of the property is approximately up to 9.7m long. Above ground the building on the site is approximately 4.7m wide and 8.4m long. The site is situated on the west side of Arlington Road.

The site is bounded to the east by the pavement of Arlington Road, to the south by 173 Arlington Road, to the north by 177 Arlington Road. The property at 175 Arlington Road shares a strip footing with the property at 173 Arlington Road and 177 Arlington Road along the boundary wall.

A site layout plan is included as Figure 1.

2.4 Topography

The site and surrounding area slopes gently downwards towards the southeast, with Ordnance Survey mapping of the area recording spot height elevations of 28 metres above Ordnance Datum. A topographic survey for the site shows it to be relatively level.
The site is not located on a slope of greater than 7 degrees. The site as not being located within an area of significant landslide potential.

2.5 Proposed Development Plan

It is proposed to construct the basement to the full width of the site and to the part of the rear garden. This will result in the basement extending outside the footprint of the above ground. The formation level of the proposed basement will be at approximately 25mOD. The structure will be supported by ‘L’ shaped underpin footings.
The boundary wall shared with 173 and 177 Arlington Road will be underpinned, along with front elevation. Internal structural loads will be supported by steel frames located at the middle of existing house and at the location of the existing rear wall. Steel frame will be supported on RC raft on basement level.

Proposed development plans are included in Appendix B.

3. DESK STUDY

3.1 Site History

A brief review of the site's historical development has been undertaken using publicly available literature and Lim's in-house resources. The findings are summarized as follows:

Historical mapping dated 1870 records the site as open land forming part of the grounds of Camden.

No significant changes are noted to the site.

175 Arlington Road, London NW1 7EY Road is not recorded as having sustained any damage during the Second World War bombings[7].

3.2 Published geology

The British Geological Survey (BGS) sheet[4] for the area indicates the site to be directly underlain by the London Clay Formation with no record of superficial deposits on site.

The London Clay Formation is an over-consolidated firm to very stiff, becoming hard with depth, fissured, blue to grey silty clay of low to very high plasticity with very low permeability. The upper and lower parts may contain silty or fine grained sand partings. The stratum may also contain laminated, structured, nodular claystone and rare sand partings. Crystals of gypsum (selenite) are often present within the weathered London Clay Formation. The stratum is generally horizontally bedded.

BGS basal contour mapping demonstrates the base of the London Clay Formation is present below the site to an elevation of approximately -20.0mOD, suggesting an overall thickness of approximately 80m on site.

3.3 Unpublished geology

A total of four historical British Geological Survey (BGS) borehole records were reviewed, at distances of between 100m and 550m of the site boundary. The strata encountered within the boreholes are summarized in Table 1.

Table 1. Summary of BGS Borehole Records

Stratum	Top of stratum (mbgl)	Thickness(m)
Soft to firm grey brown sandy silty gravelly CLAY. Occasional selenite crystals at some depth.	0.7 to 0.7	0.45 to 3.0
Firm extremely closely fissured dark gray brown (silty) clay [LONDON CLAY FORMATION]	0.7 to 10.0	Proven to 20mbgl[1]

1. *Thickness proven in three out of nine boreholes only.*

No groundwater was encountered in the boreholes which were recorded as dry on completion.

3.4 Hydrogeology

The site does not overlie a designated superficial or bedrock aquifer and is noted as being underlain by the London Clay Formation, designated a 'non-productive stratum' by the Environment Agency. The site does not fall within a Groundwater Vulnerability Zone as indicated by EA mapping, nor is the site located within a groundwater source protection zone (GSPZ).

No groundwater strikes were encountered in the historic boreholes presented in Appendix C. The natural stratum encountered in these boreholes, London Clay Formation, is characteristically of very low permeability and therefore does not contain a continuous groundwater body. However, the stratum can contain isolated limited lenses of perched water.

3.5 Hydrology, Drainage and Flood Risk

A review of the London Borough of Camden Strategic Flood Risk Assessment[5] (SFRA) shows the site is located within Critical Drainage Area 'Group3_005' and is not located within a Local Flood Risk Zone. Information provided by the Environment Agency[6] shows that the site is located within Flood Zone 1 and therefore is at low risk of tidal or fluvial flooding.

The site is understood to be underlain by the London Clay Formation, which typically has a very low permeability. While the stratum can contain isolated lenses of perched water, the stratum typically does not a continuous groundwater body. Therefore, the risk of causing a rise if groundwater level due to the proposed basement development is considered non-applicable.

It is proposed that an attenuation tank will be installed to limit surface water discharge during high rainfall events. Further details of the proposed drainage strategy can be found in the Flood Risk Assessment and Drainage Report[9].

4. SCREENING

4.1 Introduction

A screening assessment has been undertaken to assess the potential risk to local hydrology, hydrogeology and land stability. The assessment is undertaken in the form of a series of tables, setting out the questions with regard to the primary concerns associated with the proposed development. Where 'yes' or 'unknown' can be simply answered with no analysis, these answers have been provided.

4.2 Subterranean (Groundwater) Screening Assessment

This section answers questions relating to slope/land stability in Table 2.

Table 2. Subterranean (groundwater) flow

Question	Response	Action required
1a. Is the site located directly above an aquifer?	No. The site is directly underlain by the London Clay Formation, designated an unproductive stratum by the Environment Agency.	None.
1b. Will the proposed basement extend beneath the water table surface?	No. The proposed basement is expected to be constructed within the London Clay Formation. This stratum is defined as an unproductive aquifer as it typically has a very low permeability. Groundwater may be present as isolated lenses or perched water if there is topsoil or made ground present above the London Clay.	None
2. Is the site within 100m of a watercourse, well or potential spring line?	No	None
3. Is the site within the catchment of the pond chains on Hampstead Heath?	No. The Hampstead Heath pond chains are located approximately 0.95km to the north.	None
4. Will the proposed basement development result in a change in the proportion of hard surfaced/paved areas?	No. The proposed development will not add to the total area of hardstanding across the site.	None
5. As part of site drainage, will more surface water than at present be discharged to ground (e.g. via soakaways and/or SUDS)?	No. Soakaways are not likely to prove effective in the London Clay due to low infiltration rates.	None
6. Is the lowest point of the proposed excavation close to or lower than, the mean water level in any local pond or spring-line?	No.	None

The proposed development is underlain by the London Clay Formation, designated an 'unproductive stratum' by the EA. A review of available data has been conducted to determine groundwater conditions on site and suggests shallow perched groundwater may be encountered within Made

Ground or fine sand laminations within the London Clay Formation, however, this is not expected to be laterally pervasive.

The proposed basement and new structures will not increase the proportion of hard-standing across the site.

4.3 Slope/Land Stability Screening Assessment

This section answers questions relating to slope/land stability in Table 3.

Table 3. Slope/land stability

Question	Response	Action required
1. Does the site include slopes, natural or man-made, greater than about 1 in 8?	No. The site is relatively level, with a maximum change in topography of 0.3m across the site.	None
2. Will the proposed re-profiling of the landscaping at site change slopes at the property boundary to greater than about 1 in 8?	No. The proposed works do not involve the reprofiling of the site.	None
3. Does the development neighbour land including railway cuttings and the like with a slope greater than about 1 in 8?	No. There are no neighbouring cuttings or slopes.	None
4. Is the site within a wider hillside setting in which the general slope is greater than about 1 in 8?	No. Figure 16 of the Camden Geological, Hydrogeological and Hydrological Study[3] shows the site is not within an area where the slope of the wider area exceeds 7 degrees.	None
5. Is the London Clay the shallowest strata on site?	Yes. The proposed development is in close proximity to two neighbouring properties, and therefore the effect of heave in the underlying London Clay due to basement excavation will need to be considered.	Investigation and assessment
6. Will any trees be felled as part of the proposed development and/or are any works proposed within any tree protection zones where trees are to be retained?	No. It is expected that no trees will be removed as part of the proposed development.	None
7. Is there a history of shrink/swell subsidence in the local area and/or evidence of such at the site?	Unknown. The shallowest stratum beneath the site is the London Clay Formation and therefore the effect of heave in the underlying London Clay due to basement excavation will need to be considered.	None
8. Is the site within 100m of a watercourse or a potential spring line?	Possibly.	Investigation

Question	Response	Action required
9. Is the site within an area of previously worked ground?	No.	None
10. Is the site within an aquifer?	No.	None
11. Is the site within 50m of the Hampstead Heath ponds?	No.	None
12. Is the site within 5m of a highway or pedestrian right of way?	Yes. The front of the property is some 0.5m from the pedestrian along the south side of Arlington Road. There is also a highway, however it is some 2.5m from the property. The impact on the pedestrian footpath will therefore be assessed.	Impact Assessment
13. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Yes. It is understood that the neighboring at 173 and 177 Arlington Road has a two story building	Impact Assessment
14. Is the site over (or within the exclusion zone of) any tunnels?	No.	None

A review of local topography suggests that local and wider hillslopes do not exceed a gradient of 1 in 8. and the Study indicates the site is not located in an area of landslide potential.

In summary, the basement excavation will result in unloading of the London Clay Formation at depth which without significant structural reloading may result in heave movements. The construction of the basement will significantly increase the differential depth of foundations between 173 and 177 Arlington Road. The impact assessment will assess potential damage caused by ground movements to adjacent properties and will recommend measures to mitigate such potentially damaging movements.

The proposed basement will be located approximately 2.5m from the pedestrian footpath along Arlington Road. There the impact from the ground movements resulting from the proposed development will be assessed.

4.4 Surface Flow and Flooding Screening Assessment

This section answers questions relating to surface flow and flooding in Table 4.

Table 4. Surface water and flooding

Question	Response	Action required
1. Is the site within the catchment area of the pond chains on Hampstead Heath?	No	None
2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off), be materially changed from the existing route?	No. The proposed development will be built beneath areas of existing hardstanding.	None
3. Will the proposed development result in a change in the proportion of hard surfaced/paved external areas?	No. The proposed development will not result in a change in the total area of hardstanding.	None
4. Will the proposed basement result in a change to the profile of the inflows of surface water being received by adjacent properties or downstream watercourses?	No. The amount of surface water is not expected to increase as a result of the proposed development. The Flood Risk Assessment and Drainage Report[9] for the proposed development demonstrates that surface water can be managed by the combination of permeable paving and an attenuation tank.	None
5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No. The proposed excavation would remove the majority of any Made Ground that may be present on site and as such will not impact on water quality.	None
6. Is the site in an area known to be at risk from surface flooding, or is it at risk from flooding because the proposed basement is below the static water level of a nearby surface water feature?	No.	None

The proposed development is for a basement extension, the majority of which will be beneath the footprint of the existing building. There is not considered to be a significant change in surface water flows. The proposed basement will not increase the proportion of hard-standing across the site.

4.5 Non-technical Summary of Screening Process

On the basis of this screening exercise, further stages of basement impact assessment are required for this site as presented in Table 5.

Table 5. Summary of Basement Impact Assessment Requirements

Item	Description
1.	*Groundwater flow and Slope/land stability* The basement will be constructed entirely within the London Clay and therefore groundwater is not expected to be encountered. Given the relatively impermeable nature of the London Clay, infiltration will be negligible.

Item	Description
2.	*Slope/land stability* Investigation and assessment – The proposed development and neighbouring properties are potentially at risk from heave/settlement of the London Clay Formation. The impact of the basement construction on adjacent party walls and the neighbouring pedestrian footpath requires consideration and an impact assessment is required.
3.	*Surface flow and flooding* None – the proposed basement and new structures will increase the proportion of hard-standing across the site. However, due to the impermeable nature of the underlying London Clay Formation, the run-off surface attenuation characteristics are not significantly affected. The site is not located in an area at risk from surface water flooding.

The outcomes of the screening assessment are carried forward into the Basement Impact Assessment in the following report sections.

5. SCOPING

On the basis of the screening report, an intrusive investigation is required on site. The intrusive investigation should:

1. Determine the ground conditions on site and their variability;
2. Install groundwater monitoring standpipes to determine groundwater levels;
3. Undertake in-situ testing to assess the strengths of the ground and to support geotechnical assessment; and
4. Obtain soil samples for geotechnical laboratory testing in order to classify the soils on site, to determine where desiccation is present on site, and to support geotechnical design.

A site investigation was undertaken for the neighbouring site of 220 Arlington Road. Given that the investigation boreholes were undertaken in close proximity to the study site and that the underlying geology is the relatively consistent London Clay Formation, it is considered that the findings of the investigation at 220 Arlington Road are appropriate inform the assessment for the proposed basement at 175 Arlington Road.

The investigation is summarised within Section 6 of this report.

6. GROUND INVESTIGATION

6.1 Introduction

This report section presents the findings of the ground investigation carried out at the neighbouring site of 220 Arlington Road.

6.2 Fieldwork

An intrusive investigation was undertaken by RSA in August 2006, comprising one cable percussion borehole (BH01) to a depth of 10mbgl,

Standard Penetration Tests (SPTs) and undisturbed U100 samples were undertaken within the boreholes and groundwater monitoring wells were installed within boreholes BH01.

Selected soil samples were submitted to an accredited laboratory for geotechnical testing including the following:

- Atterberg Limits tests;
- Undrained triaxial compression tests;
- Moisture content; and
- BRE analysis in accordance with BRE SD1[13].

[11] British Standards Institution. (1990). *Methods of Test for Soils for Civil Engineering purposes*. BS1377:1990.
[12] British Standards Institution. (2015). *Code of practice for ground investigations*. BS5930:1999
[13] Building Research Establishment. (2005). Special Digest 1 – Concrete in aggressive ground, third edition.

6.3 Ground Conditions

The ground conditions encountered during the intrusive investigation broadly corresponded to the published geology and are summarised in Table 6 below.

Table 6. Summary of ground conditions

Stratum	Depth to top of stratum (mOD) [mbgl]	Thickness (m)
[MADE GROUND] Dark grey subangular fine-course gravel size ash. Clinker, brick and mortar fragments with occasional pieces of glass, plastic wood and pottery.	28 [0.0]	0.7
Firm to stiff dark brown CLAY. Firm extremely closely fissured dark grey brown (silty) clay. [LONDON CLAY FORMATION]	28 to 18 [0.7 to 3)]	Proven to 20mOD (10mbgl)

The ground conditions are discussed in the following sections together with the results of the in-situ and laboratory geotechnical tests.

6.3.1 Made Ground

Made Ground was found to comprise concrete or gravel overlying soft to firm dark grey to light orange brown gravelly clay to a level of between 27mOD to 28mOD. No visible or olfactory evidence of contamination was recorded.

Standard SPT testing within this stratum recorded 'N' values of between 4 and 16, corresponding to a 'very soft' to 'firm' clay[12].

6.3.2 London Clay Formation

The surface of the London Clay Formation was encountered at between 18mOD to 28mOD and the stratum was found to comprise firm to stiff brown clay. The London Clay Formation extended to the base of borehole BH01 at 18mOD (10mbgl)

Triaxial testing undertaken on samples collected between 2.0mbgl and 10.0mbgl recorded undrained shear strength (c_u) values between 34kPa to 130kPa, generally increasing with depth. These values correspond to clay of 'medium' to 'high' strength[12]. These values are supported by the in-situ SPT testing which recorded 'N' values of between 7 and 20, corresponding to a 'soft' to 'stiff'[12] clay. Plots of SPT and c_u against level are presented as Plate 1 and Plate 2, respectively.

The results of the geotechnical laboratory analyses have indicated index properties for the London Clay in the following ranges:

- Moisture Contents between 29% and 35%;
- Liquid Limits between 69% and 78%;
- Plastic Limits between 24% and 27%; and
- Plasticity Indices between 45% and 52%.

Based on the above data, the London Clay Formation may be classified as clay of 'very high'[12] plasticity with a high[14] volume change potential which is consistent with published data.

6.4 Groundwater

No groundwater was encountered during drilling.
It is anticipated that the groundwater encountered within the London Clay Formation is perched water within the claystone band and is not representative of wider groundwater table.

6.5 Sulfate and pH Conditions

No Sulfate and pH Conditions results have been found in this test though two samples of Made Ground and two samples of London Clay Formation were analysed in similar condition for pH and sulfate. In addition, three water samples were collected and also underwent analysis. The laboratory results are summarised in Table 7 and

Table 8.

Table 7. Summary of pH and sulfate results – Soil samples

Sample location	Sample depth (mbgl)	Strata	pH	Total sulfate as SO_4 (mg/kg)	Water Soluble sulfate as SO_4 (2:1 leachate equivalent) (g/l)	Total sulphur (mg/kg)	Oxidisable Sulphide (OS % SO_4)
WS01	0.5	Made Ground	7.7	830	0.044	-	0
WS01	1.0	Made Ground	7.6	690	0.16	-	0

[14] NHBC Standards. (2019). Chapter 4.2 – Building near trees.

Sample location	Sample depth (mbgl)	Strata	pH	Total sulfate as SO_4 (mg/kg)	Water Soluble sulfate as SO_4 (2:1 leachate equivalent) (g/l)	Total sulphur (mg/kg)	Oxidisable Sulphide (OS % SO_4)
WS01	4.0	London Clay Formation	7.5	120,000	2.6	40,000	0
WS06	3.0	London Clay Formation	7.8	1,300	0.49	480	0.014

Table 8. Summary of pH and sulfate results – Water samples

Sample location	Strata	pH	Sulfate as SO_4(mg/l)
WS01	Made Ground	6.9	156
WS02	Made Ground	7.0	287
BH01	London Clay Formation	7.0	3900

The implications of these results on the building design are discussed in further detail in Section 6.8.

6.6 Geotechnical Design Parameters

Geotechnical design parameters are recommended based on the available information from the intrusive investigation and published information. These are summarised in Table 9. The values are unfactored (Serviceability Limit State) parameters and are considered to be characteristic values for the local soils. Plots showing the SPT ‘N’ values and undrained shear strength, c_u with level above Ordnance Datum level are presented in Plate 1 and Plate 2.

Table 9. Geotechnical design parameters

Stratum	Design Level (mOD)	Bulk Unit Weight γ_b (kN/m³)	Undrained Cohesion c_u (kPa) [c′]	Friction Angle φ' (°)	Young’s Modulus E_u (MPa) [E′]
Made Ground (cohesive)	27	19	35 [1]	28[a,b]	21[b] [15.75]
London Clay Formation	20	20	40 + 7z[c] [5]	21[a]	24 + 4.2z[d] [18 + 3.15z][e]

a. BS 8002:2015 Code of practice for Earth retaining structures, British Standards institution.
b. Burland et. al (Eds) (2001) Building response to tunnelling, CIRIA Special Publication 200, CIRIA
c. z = depth below upper surface of the London Clay
d. Based on 600 Cu - Burland, Standing J.R., and Jardine F.M. (eds) (2001), Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.
e. Based on 0.75Eu - Burland, Standing J.R., and Jardine F.M. (eds) (2001), Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.

Plate 1. SPT 'N' values against level

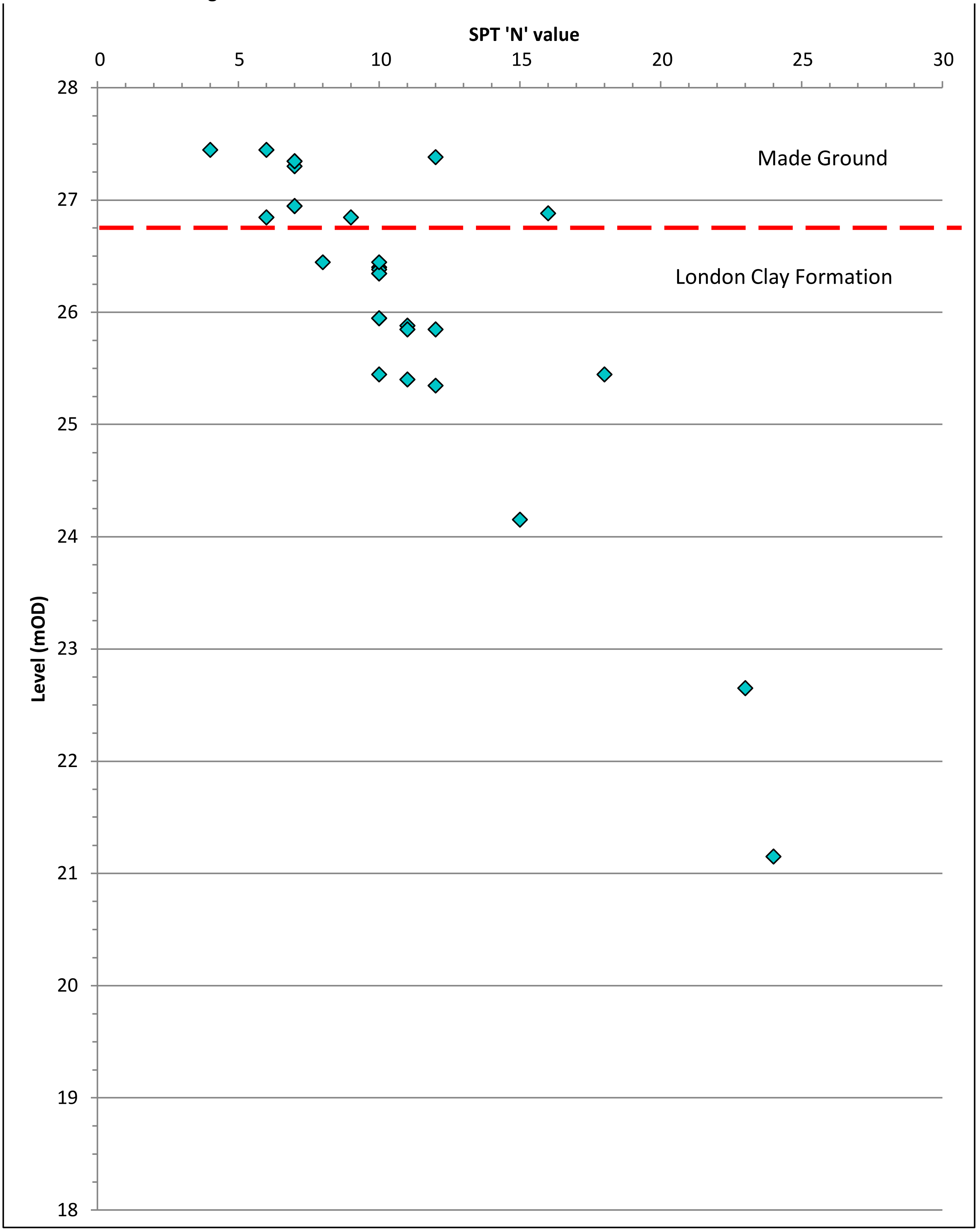
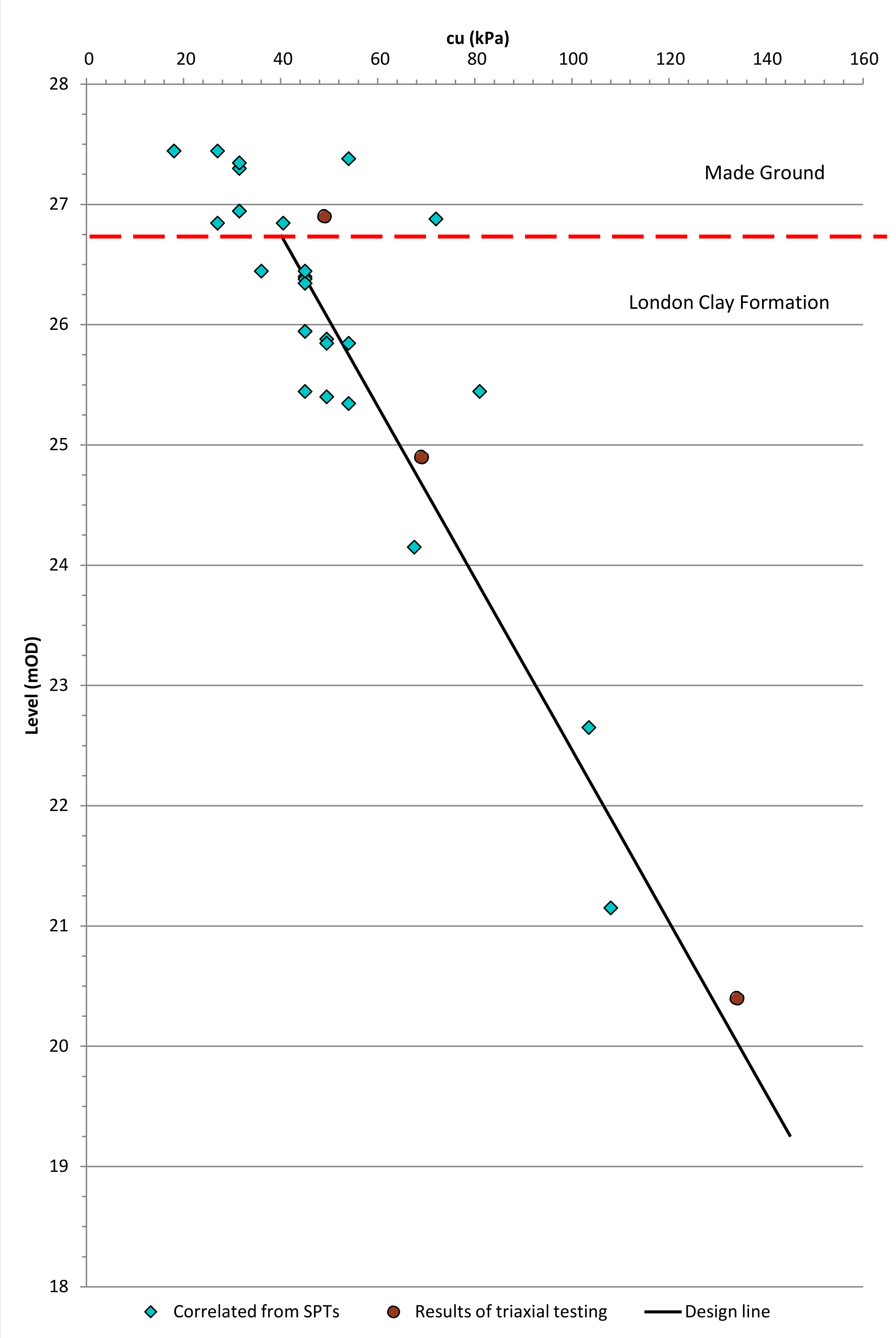


Plate 2. Undrained shear strength, c_u against level



6.7 Concrete Aggressive Ground Classification

The Design Sulfate (DS) and Aggressive Chemical Environment for Concrete (ACEC) classifications for each stratum encountered has been determined based on the results presented in Table 7 and

Table 8. The subsequent assessment carried out to determine the DS and ACEC classifications has been undertaken in accordance with the method specified in BRE Special Digest 1[15].

Pyritic soils are typically found in soils with an Oxidisable Sulphide (OS) percentage of greater than 0.3%. The London Clay Formation is typically pyritic, however layers close to the surface can be weathered causing existing pyrite in the soil to oxidise and convert to sulfate, thereby increasing the overall concentration of sulfate. The results in Table 7 indicate that the London Clay Formation soil samples recovered are not pyritic and are therefore likely to be weathered. The resulting DS and ACEC classification are presented in Table 10.

Table 10. DS and ACEC classification for stratum encountered

Stratum	DS Class	ACEC Class
Made Ground	DS-1	AC-1
London Clay Formation	DS-4	DS-3s

[15] Building Research Establishment. (2005). Special Digest 1:2005 – Concrete in aggressive ground, Third edition.

7. CONSTRUCTION METHODOLOGY

7.1 Introduction

This section outlined the assumed construction sequence and loads/pressures used in the ground movements assessment.

7.2 Construction Sequence

Construction sequence for the proposed development as provided in Appendix F. For the purpose of this report the construction sequence has been rationalised into three construction stages, based on the critical stages in terms of resulting ground movements. The rationalised construction sequence is presented in Table 11, along with the corresponding construction stages.

Table 11. Rationalised construction sequence

Construction Stage	Stage Description	Corresponding Construction Stages No.
Stage 1 - Construction of underpins	The installation of the underpins will result in load being transferred to previously unloaded ground, resulting in short-term elastic undrained settlement of the surrounding soil. In addition, a gap is left between the top of the underpin and the existing foundation which is infilled using 'dry pack'. The amount of settlement caused by the existing foundation bearing onto the underpin is dependent on the quality of the workmanship.	1 to 6
Stage 2 - Excavate basement to formation level	The excavation of the basement will reduce the total in-situ stresses in the surrounding ground, causing the ground to heave.	7 to 9
Stage 3 – Construction of basement slab and long-term ground movements	The construction of the basement slab and internal structural members will cause the ground at formation level to settle. In the long-term, additional ground movements will occur due to the ground conditions changing from undrained to drained as the soil consolidates. The consolidation process causes the water content of the soil to either increase or decrease, depending on the net change in total in-situ stresses caused by the construction of the basement.	10 to 17

7.3 Excavation Loads

The excavation of the basement will unload the underlying soils, resulting in heave movements. The magnitude of the unloading has been calculated based on the depth of each soil stratum excavated and the respective unit weight presented in Table 9.

Table 12. Excavation unloading

Zone	Thickness of Made Ground Excavated (m)	Unloading from Excavation of Made Ground (kPa)	Thickness of Made London Clay Formation (m)	Unloading from Excavation of London Clay Formation (kPa)	Excavation unloading (kPa)
Excavation – Basement extension	0.7	13.3	2.3	43.7	57

a. Depth of excavation is based on an assumed existing formation level of 28mOD and a proposed excavation formation level of 25mOD.

7.4 Underpin Loads

For the purpose of this report the upper-bound value of 85kN/m has been assumed.

A preliminary underpin foundation design has undertaken to determine the required width of the underpin. Based on the geotechnical parameters presented in Table 9 an allowable bearing pressure of 150kPa is recommended for the proposed development. Therefore, for this report a footing width of 1.6m at a formation level of 25mOD has been assumed. The underpin bearing pressure has been split into two areas; one area below the stem of the underpin and another area below the toe of the underpin. A bearing pressure below the stem of the underpin of 140kPa has been calculated. The bearing pressure below the underpin toe includes the unloading due the excavated soil presented in Table 12, of 57kPa. The net bearing pressure has therefore been calculated to be 83kPa.

7.5 Basement Raft Loads

Sensitivity analysis for the raft bearing pressures by apply a low, medium and high spring stiffness to model the interaction between the raft and soil below. For the purpose of this report, the higher stiffness pressures have been adopted and rationalised into three load areas.

8. GROUND MOVEMENT ASSESSMENT AND DAMAGE CATEGORY ASSESSMENT

8.1 Introduction

The following report section presents the predicted ground movements arising from the construction stages presented in Section 7.2. The section also presents the methodology usied to predict the building damage category for the neighbouring building and the resulting impact.

The possible ground movement mechanisms resulting from the proposed development are outlined below:

- Heave movements: London Clay is susceptible to short-term and long-term time dependent swelling after unloading, which will occur as a result of the basement creating upward ground movements;
- Underpin deflection: Underpins act as stiff concrete retaining structures. Therefore, deflections due to structural deformation of the underpins can be assumed to be negligible, provided that the underpins are properly designed, constructed and sufficiently supported in the temporary condition. The impact of the lateral expansion of the London Clay on the neighbouring building is likely to be very localised, as it is proposed to install the underpin wall in 1m wide sections in a 'hit-and-miss' sequence;
- Underpin settlement: The underpins will settle as the structural loads are transferred onto previously unloaded soils;
- Workmanship settlement: During the construction some movement is expected due to load transfer from the existing building onto the new underpin foundations. The magnitude of this movement depends on the quality of the workmanship, and for the purpose of this assessment it is assumed to be of the order of 5mm.

Ground movements have been determined using a combination of Lim in-house experience of underpin foundations.

8.2 Ground Movement Assessment

The following section presents the ground movements as the basement development progresses at the stages presented in Table 11. The ground movements presented for each stage are cumulative, meaning that the ground movements from each stage are carried forward to the next stage. A critical section line has been placed through the property at 173 and 177 Arlington Road to measure the amount of ground movement that is predicted to occur.

8.2.1 Stage 1 – Construction of Underpins

This stage models the ground movements resulting from the installation of the underpin strip footings.

The ground movements across the critical section line below 173 and 177 Arlington Road are summarised in Table 13. The results include 5mm of installation settlement due to workmanship as discussed in Section 8.1.

Table 13. Vertical ground movements along 173 and 177 Arlington Road critical section line – Stage 1

Minimum vertical ground movements (mm)	Maximum vertical ground movements (mm)	Foundation formation Level (mOD)
0	6.6	25

Note: Negative vertical movements indicate heave, positive values indicate settlement.

8.2.2 Stage 2 – Excavate Basement to Formation Level

This stage models the ground movements resulting from the excavation of the basement to formation level.

The ground movements across the critical section line below 173 and 177 Arlington Road are summarised in Table 14.

Table 14. Vertical ground movements along 173 and 177 Arlington Road critical section line – Stage 2

Minimum vertical ground movements (mm)	Maximum vertical ground movements (mm)	Foundation formation Level (mOD)
0	4.3	25

Note: Negative vertical movements indicate heave, positive values indicate settlement.

8.2.3 Stage 3 - Construction of Basement slab and Long-term Ground Movements

This stage models the ground movements in the long-term once the basement slab and internal load bearing members have been construction, and the soil surrounding has consolidated. This stage has been modelled in PDISP using the basement raft pressures, excavation heave pressures and underpin bearing pressures. The drained Young's Modulus values, E' presented in Table 9 have been used to model the ground conditions once the Made Ground and London Clay Formation have consolidated.

The ground movements across the critical section line below 173 and 177 Arlington Road are summarised in Table 15. The results in the table include the ground movements and the 5mm installation settlement due to workmanship during the installation of the underpins.

Table 15. Vertical ground movements along 173 and 177 Arlington Road critical section line – Stage 3

Minimum vertical ground movements (mm)	Maximum vertical ground movements (mm)	Foundation formation Level (mOD)
0	5.2	25

Note: Negative vertical movements indicate heave, positive values indicate settlement.

8.3 Building Damage Assessment

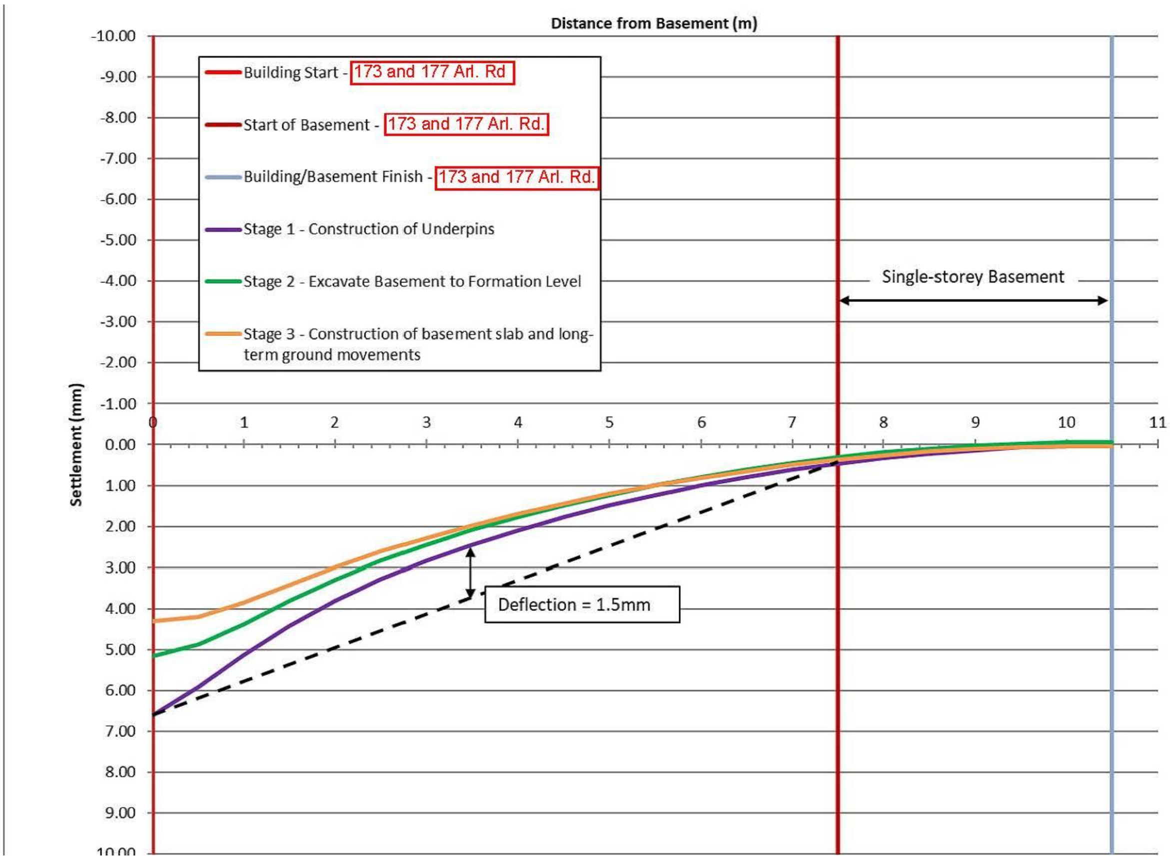
The calculated ground movements have been used to assess the potential 'damage category' that may apply to the neighbouring structures/infrastructure due to the proposed development. The methodology proposed by Burland and Worth[16] and later supplemented by the work of Boscardin and Cording[17] has been used, as described in CIRIA Special Publication 200[18]. General categories are summarised below in Table 16.

Table 16. Classification of damage visible to walls

Category	**Description**
0 (Negligible)	Hairline cracks of less than about 0.1mm are classed as negligible
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm)
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks >3mm)
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and window (crack width 15 to 25mm but depends on number of cracks)
5 (Very severe)	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks)

The vertical displacements lines along the critical section line across 173 and 177 Arlington Road have been plotted in Plate 3. This plot includes the settlement due to workmanship when installing the underpin footings. The results indicate the maximum angular distortion between the 175 and 173/177 Arlington Road party walls and the nearside basement wall below 173 and 177 Arlington Road (see Plate 3) is of the order of 1/1130. These are conservative values as the calculation does not include the stiffness of the foundation slab of the property and assumes fully flexible loaded zones. The predicted angular distortions across the width of the property are within the limits identified by Skempton and MacDonald[19] for structural damage, where it is stated that the safe limit of angular distortions for a concrete framed structure is 1/200 for structural damage and 1/500 for limiting damage to partitions and walls within a concrete framed building. Rankine (1988)[20] states for angular distortions of less than 1/500 and maximum building settlement of less than 10mm the impact on buildings is Risk Category (negligible – superficial damage unlikely).

Plate 3. Vertical ground movements across 173 and 177 Arlington Road



16 Burland, J.B., and Wroth, C.P. (1974). Settlement of buildings and associated damage, State of the art review. Conference on Settlement of Structures, Cambridge, Pentrech Press, London, pp 611-654.

17 Boscardin, M.D., Cording, E.J. (1989). Building response to excavation induced settlement. J Geotech Eng ASCE, 115(1), pp 1-21.

18 Burland, Standing, J.R., and Jardine, F.M. (eds) (2001). Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.

19 Skempton, A.W. and MacDonald, D.H. (1956). Allowable settlement of buildings. Proceedings of the Institute of Civil Engineers, part 3, vol. 5, pp 727-768.

20 Rankin, W.J. (1988). *Ground movements resulting from urban tunnelling: predictions and effects*

The Damage Category for the neighbouring property at 173 and 177 Arlington Road has been determined by plotting the horizontal strain and deflection ratio values as summarised in Table 17 and presented graphically in Plate 4. The damage category limits have been based on the slenderness (length/height) of the neighbouring building and the assumed structural material of the building (timber-masonry).

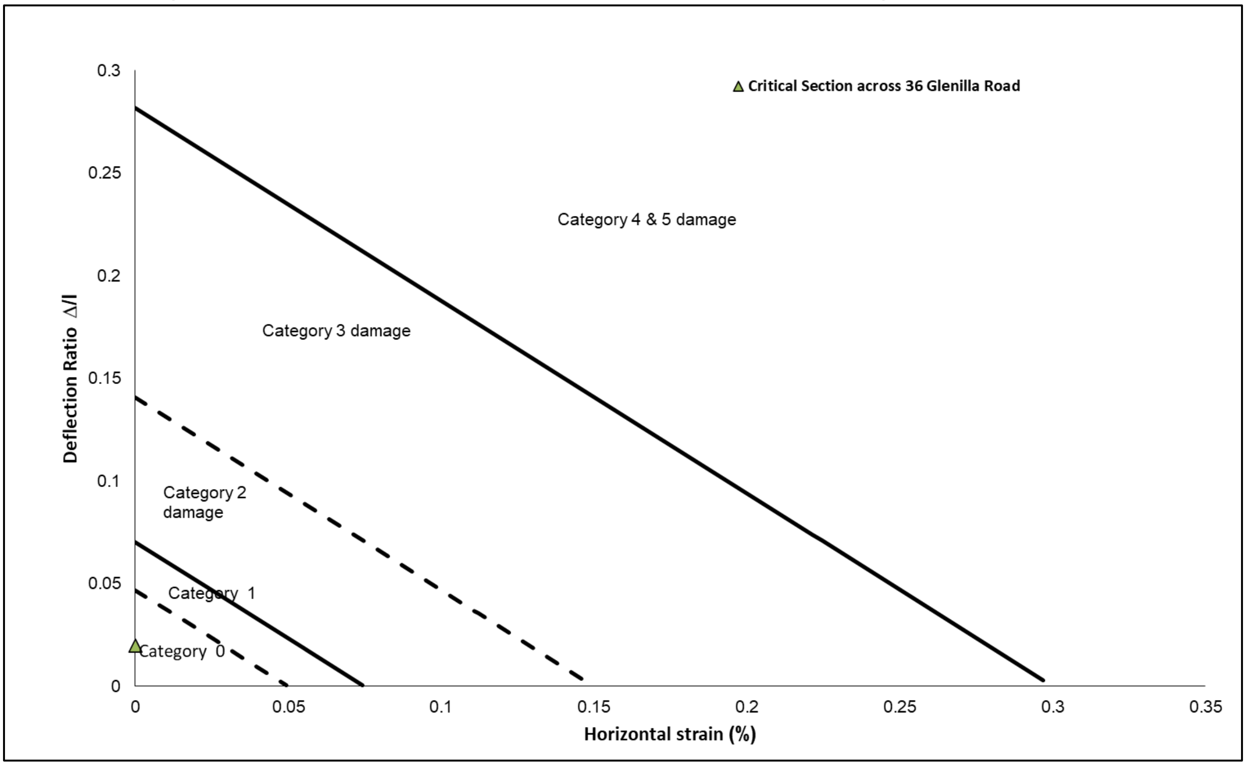
The results show that the anticipated damage category for the neighbouring building is Category 0 'negligible' damage including hairline cracks of less than about 0.1mm. This assessment assumes a good standard of workmanship and adopting a hit-and-miss construction sequence when constructing the underpin foundations.

Table 17. Summary of ground movements and corresponding Damage Category

Critical Construction Stage	Maximum Net Horizontal Movement (mm)	Maximum Deflection (mm)	*Horizontal Strain, $\delta_h/L^{a,bc}$ (%)*	*Deflection Ratio, Δ/L^a*	Damage Category
Stage 3	0	1.5	0	0.02	Category 0 (Negligible)

a. See Box 6.3 CIRIA C760 (2017)6, Guidance on embedded retaining wall design (Δ – relative deflection; L – Length of adjacent structure in metres)
b. See Figure 6.27 CIRIA C760 (2017)6, Guidance on embedded retaining wall design (δh – horizontal movement in metres)

Plate 4. Building Interaction Chart - Critical section line across 173 and 177 Arlington Road



8.4 Control of Construction Works (Monitoring Strategy)

The results of the GMA indicate that with good quality of workmanship and adopting a hit-and-miss contraction sequence when constructing the underpin foundations, damage to adjacent structures generated by the construction of the proposed basement can be restricted to within Category 0 'negligible' damage.

A formal monitoring strategy should be implemented across the site to observe and ground movements during construction.

The system should operate broadly in accordance with the 'Observational Method' as defined in CIRIA Report 185[21]. Monitoring can be undertaken by installing survey targets to the top of the basement wall and face of the adjacent buildings. Prior to construction, baseline readings should be established. Once construction commences regular readings should be taken and analysed to determine whether unacceptable horizontal movements, vertical movements and tilting has occurred.

Mitigation measures should be prepared prior to construction implemented if unacceptable movements occur. Predefined trigger values and associated mitigation should be agreed following discussions concerning the party wall.

It is recommended that a condition survey is undertaken on all adjacent walls and property facades prior to works commencing and ideally when monitoring baselines are established. Existing cracks and structural defects should be carefully recorded, documented and regularly inspected as construction progresses.

[21] Nicholson, D., Tse, Che-Ming., Penny, C. The Observational Method in ground engineering: principals and applications, CIRIA report 185, 1999.

9. BUILDING IMPACT ASSESSMENT – NON-TECHNICAL SUMMARY

9.1 Land Stability

It has been found that the site does not contain or neighbour any slopes/cuttings with a gradient of more than 7 degrees. The site itself is relatively level.

The building damage category for the neighbouring property at 173 and 177 Arlington Road can be controlled to within Damage Category 0 'Negligible' damage. This assumes a good standard of workmanship and installing the underpins in a hit-and-miss construction sequence. .

9.2 Hydrogeology and Groundwater Flooding

The BIA has concluded that there is a very low risk of groundwater flooding and that there are no impacts to the wider hydrogeological environment. It is expected that groundwater protection measures (such as cavity wall drainage) will be included in the final design to mitigate against possible groundwater intrusion into the proposed basement.

9.3 Hydrology, Surface Water Flooding and Sewer Flooding

The BIA has concluded there is a low risk of surface water/sewer flooding and that there are no impacts to the wider hydrogeological environment. A drainage strategy has been created for the proposed development by a third party[9]. The Drainage Strategy for the site proposes the installation of an attenuation tank to reduce the risk of surface water flooding

10 Construction Method Sequence

Summary

As the property has been altered several times in recent history there are numerous load paths and structural types present. The approach is therefore to install the new steelwork at ground floor level and establish clear load paths of the existing structure over by extending existing beams and removing redundant structure that would otherwise impede and complicate the basement excavation. Where steelwork will temporarily take support off the existing walls these areas will be underpinned first to reduce the risks associated with underpinning concentrated loads. After this the rest of the basement construction will follow industry standard methodology of underpinning, excavating and propping in sequence with propping only removed after the basement works are complete.

Sequence

1) Install primary underpins (those required for temporary bearing of ground floor steelwork)
2) Reduce dig and needle through base of ground floor walls and install new beams at base supported off existing footings and primary underpins.
3) Install sacrificial underpins to support new and existing beams at ground floor level to allow for excavation and basement construction prior to permanent load bearing elements being installed.
4) Underpin Party Wall
5) Carry out RC underpins to perimeter of basement
6) Excavate and demolish redundant walls and foundations as required and install multilevel propping as progressing
7) Excavate and construct drainage pit
8) Install below ground drainage and other services
9) Cast basement raft slab with pull-out reinforcement against sacrificial underpins
10) Install new internal walls and steelwork columns
11) Reinstate ground floor structure.
12) Basement complete. Propping removed.
13) Remove loadbearing walls to upper floors by needling and propping and introducing new steel frames.
14) Where floor joists are to be removed and reinstated at different levels these should be carried out sequentially with only a single room being worked on at any one time to ensure the stability of the structure generally. Alternatively, temporary bracing can be installed above floor level in each room if works are to be carried out concurrently.

FIGURE

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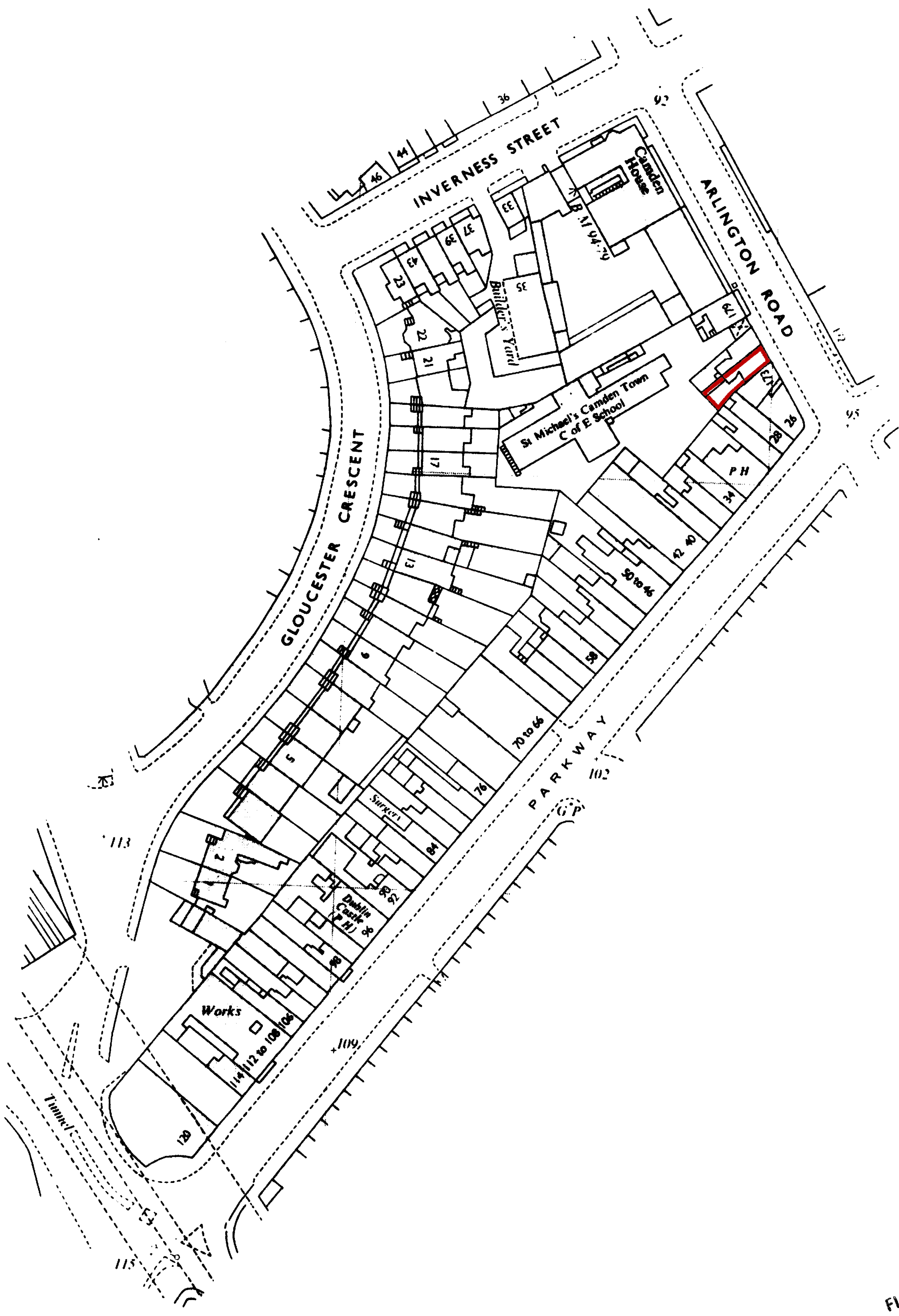
Old Reference LN VII 22 A

BOROUGH OF CAMDEN



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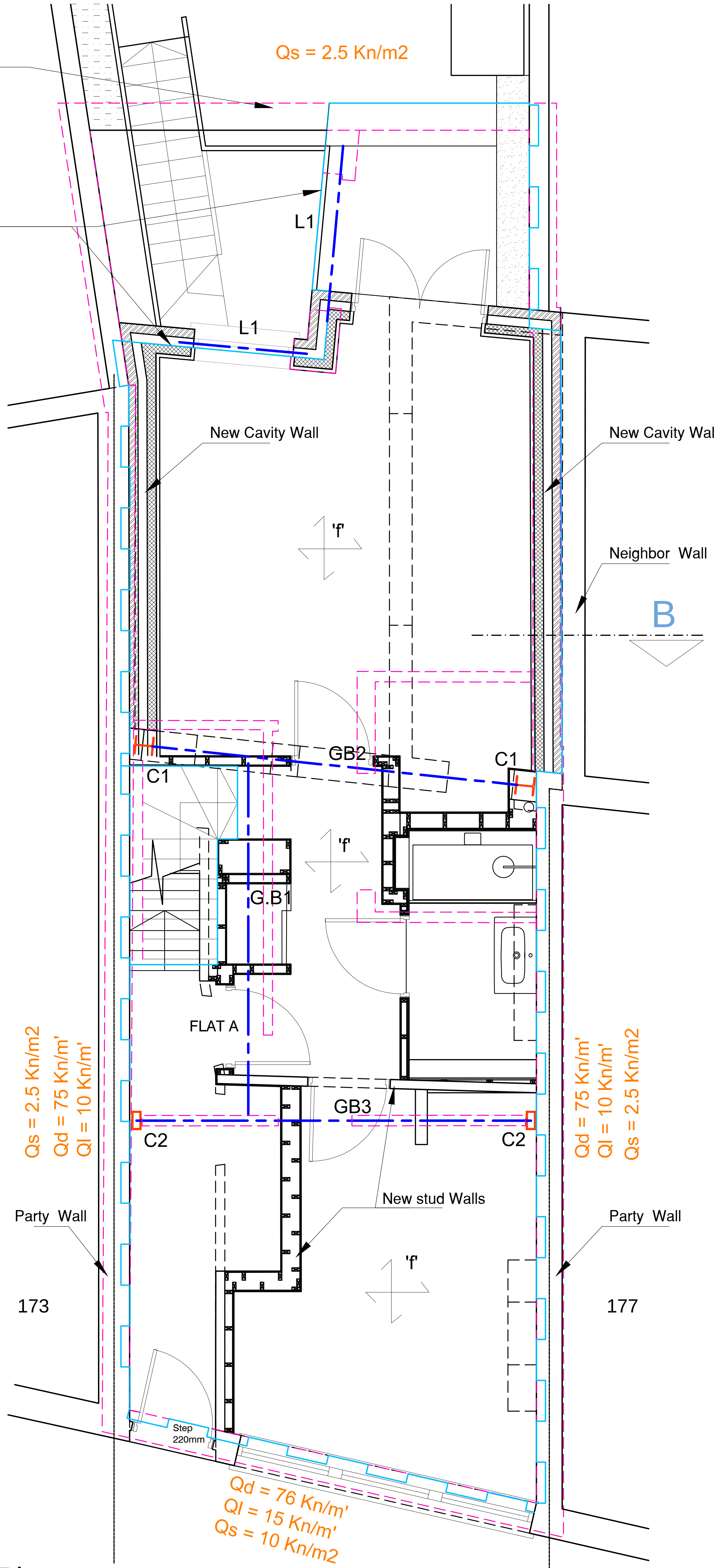

Ground Floor Plan

0m 1 2 3 4

LOADING AT GROUND FLOOR LEVEL

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APPENDIX

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STRUCTURAL DRAWINGS

JOB NO: 1569 **DATE: September 2023**

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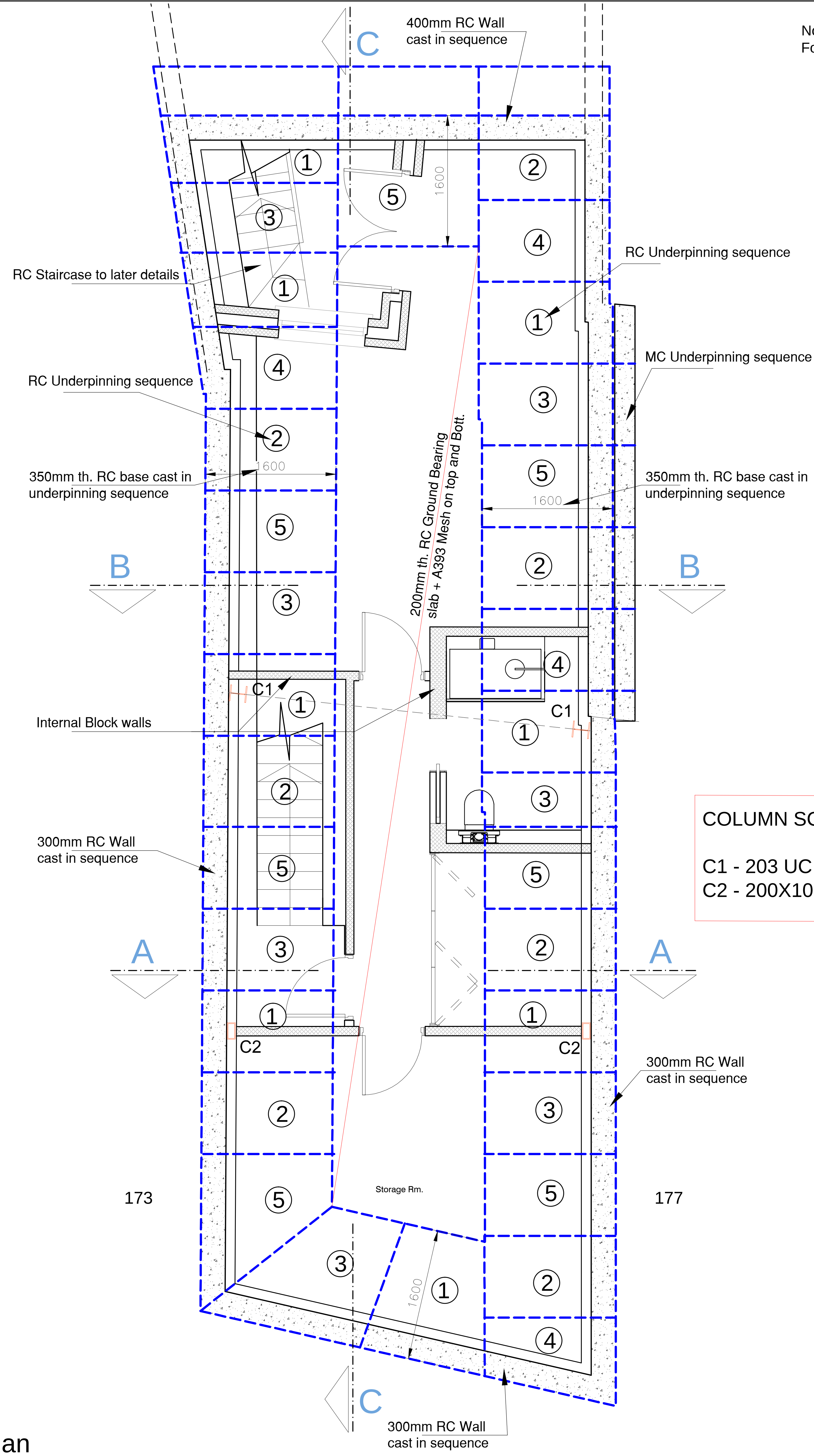

COLUMN SCHEDULE:

C1 - 203 UC 46
C2 - 200X100X8 RHS

Basement Plan

0m 1 2 3 4

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BASEMENT FLOOR PLAN

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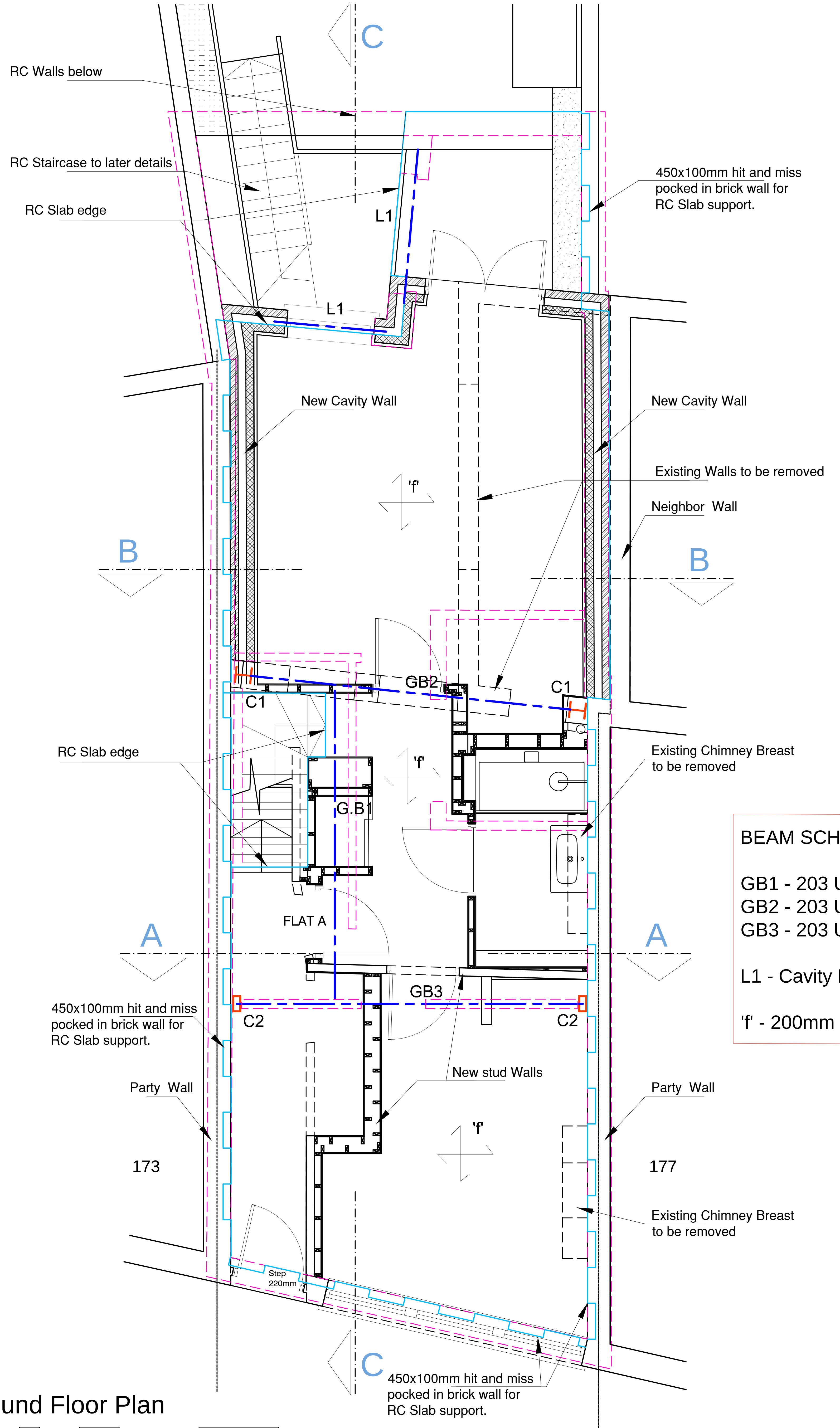

BEAM SCHEDULE:

GB1 - 203 UC 46
GB2 - 203 UC 86
GB3 - 203 UC 86

L1 - Cavity Lintel

'f' - 200mm Th. RC Slab

Ground Floor Plan

0m 1 2 3 4

First

0m

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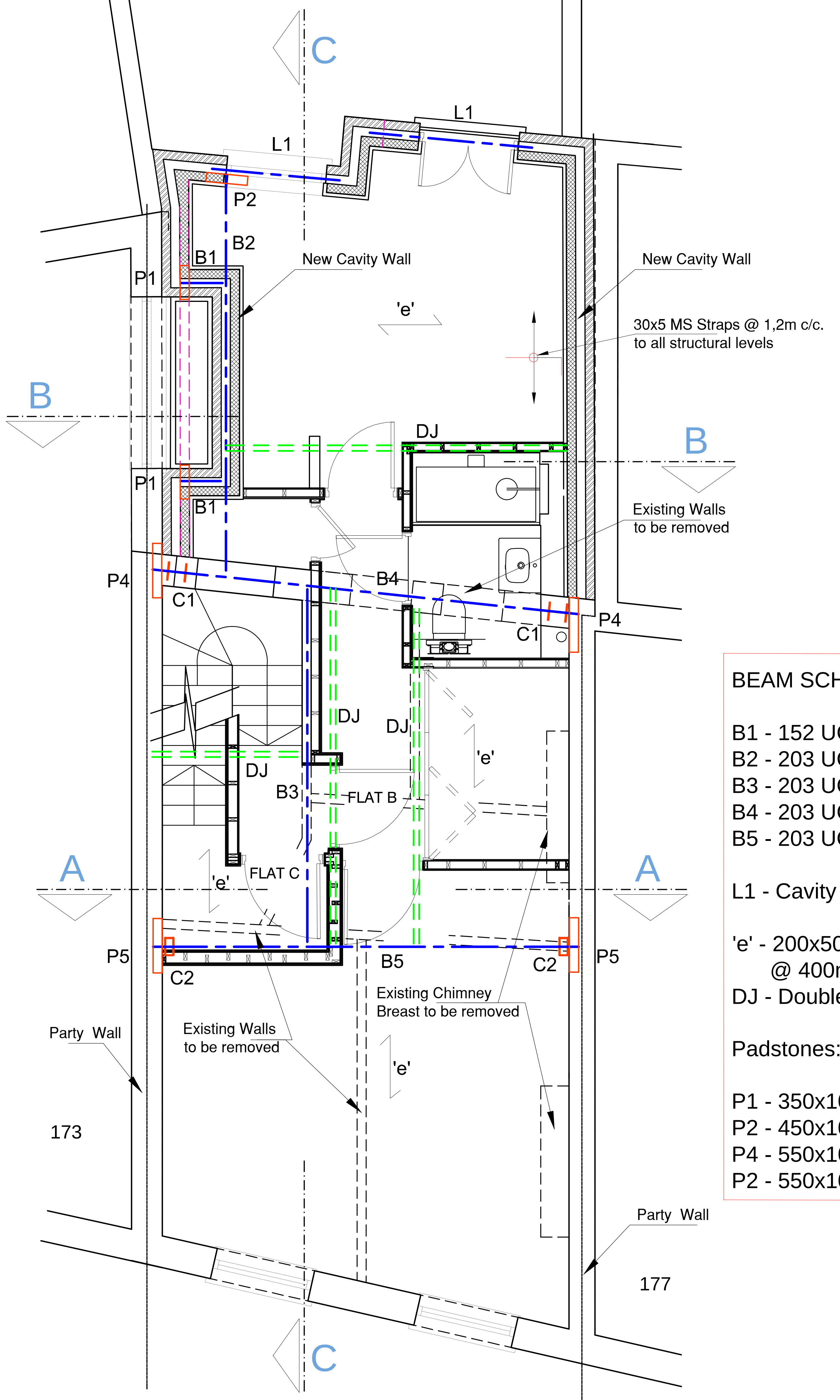


BEAM SCHEDULE:

B1 - 152 UC 23 + 5mm plate on top
B2 - 203 UC 46 + 5mm plate on top
B3 - 203 UC 46
B4 - 203 UC 52
B5 - 203 UC 52

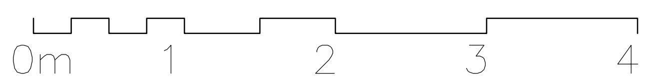
L1 - Cavity Lintel

'e' - 200x50 Timber Joists
@ 400mm c/c.
DJ - Double Joists

Padstones:

P1 - 350x100x10mm St. Plate
P2 - 450x100x15mm St. Plate
P4 - 550x100x20mm St. Plate
P2 - 550x100x20mm St. Plate

First Floor Plan



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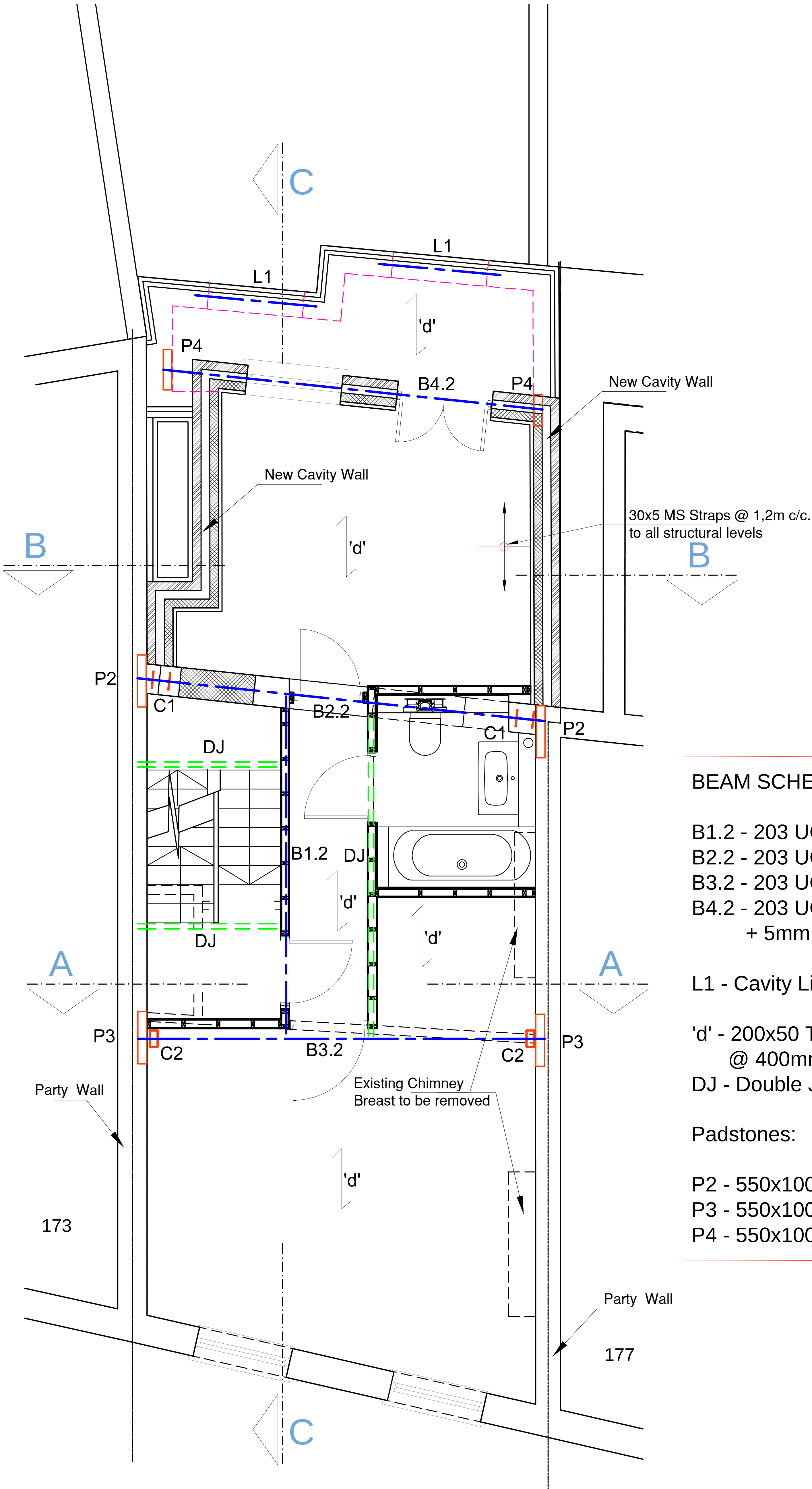

Second Floor Plan

0m 1 2 3 4

BEAM SCHEDULE:

B1.2 - 203 UC 46
B2.2 - 203 UC 52
B3.2 - 203 UC 52
B4.2 - 203 UC 52
+ 5mm St. Plate on top.

L1 - Cavity Lintel

'd' - 200x50 Timber Joists
@ 400mm c/c.
DJ - Double Joists

Padstones:

P2 - 550x100x20mm St. Plate
P3 - 550x100x20mm St. Plate
P4 - 550x100x20mm St. Plate

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SECOND FLOOR PLAN

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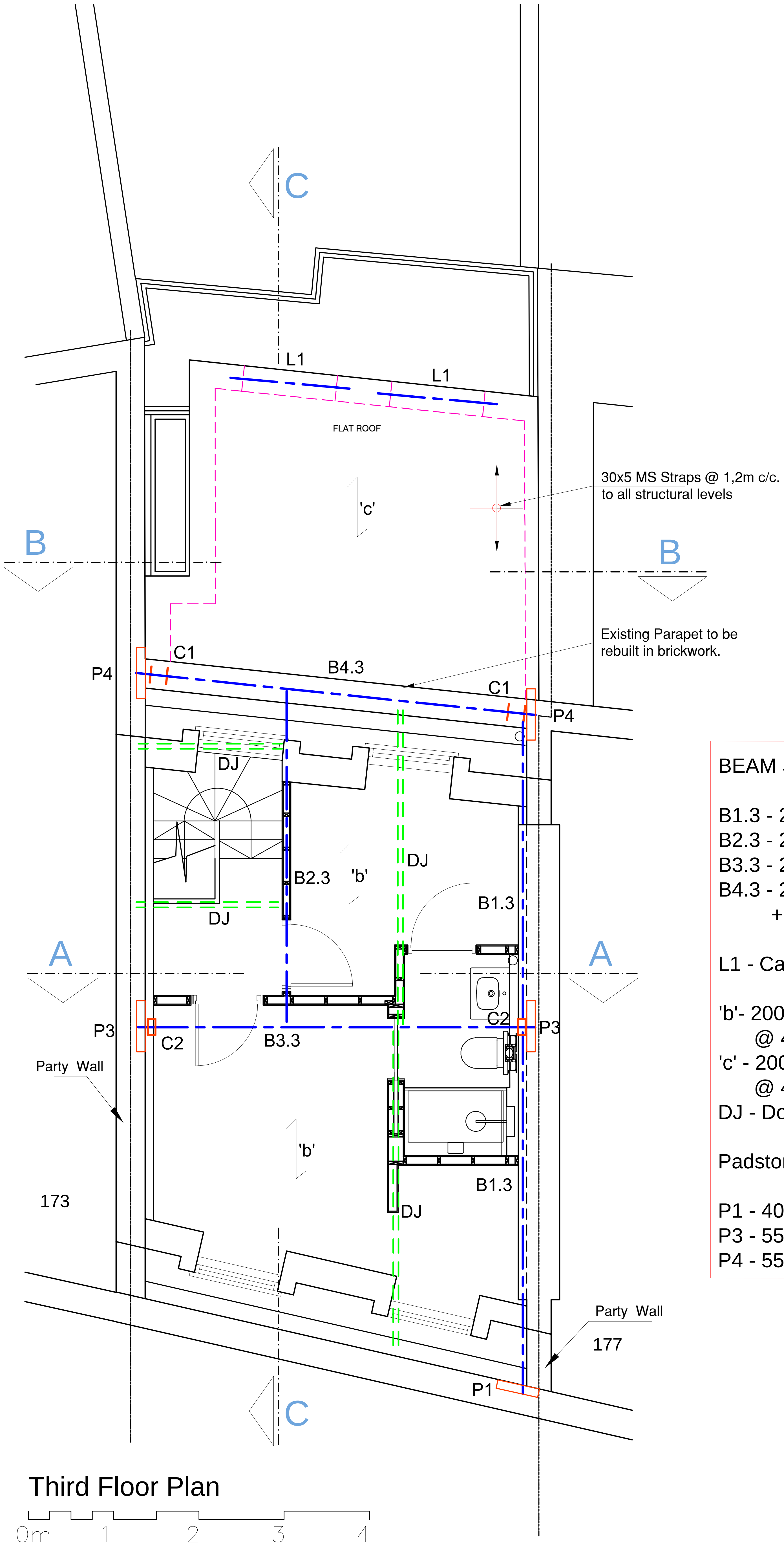

Third Floor Plan

BEAM SCHEDULE:

B1.3 - 203 x 133 UB 25
B2.3 - 203 UC 46
B3.3 - 203 UC 52
B4.3 - 203 UC 52
+ 5mm St. Plate on top.

L1 - Cavity Lintel

'b'- 200x75 Timber Joists
@ 400mm c/c.
'c' - 200x50 Timber Joists
@ 400mm c/c.
DJ - Double Joists

Padstones:

P1 - 400x100x15mm St. Plate
P3 - 550x100x20mm St. Plate
P4 - 550x100x20mm St. Plate

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THIRD FLOOR PLAN

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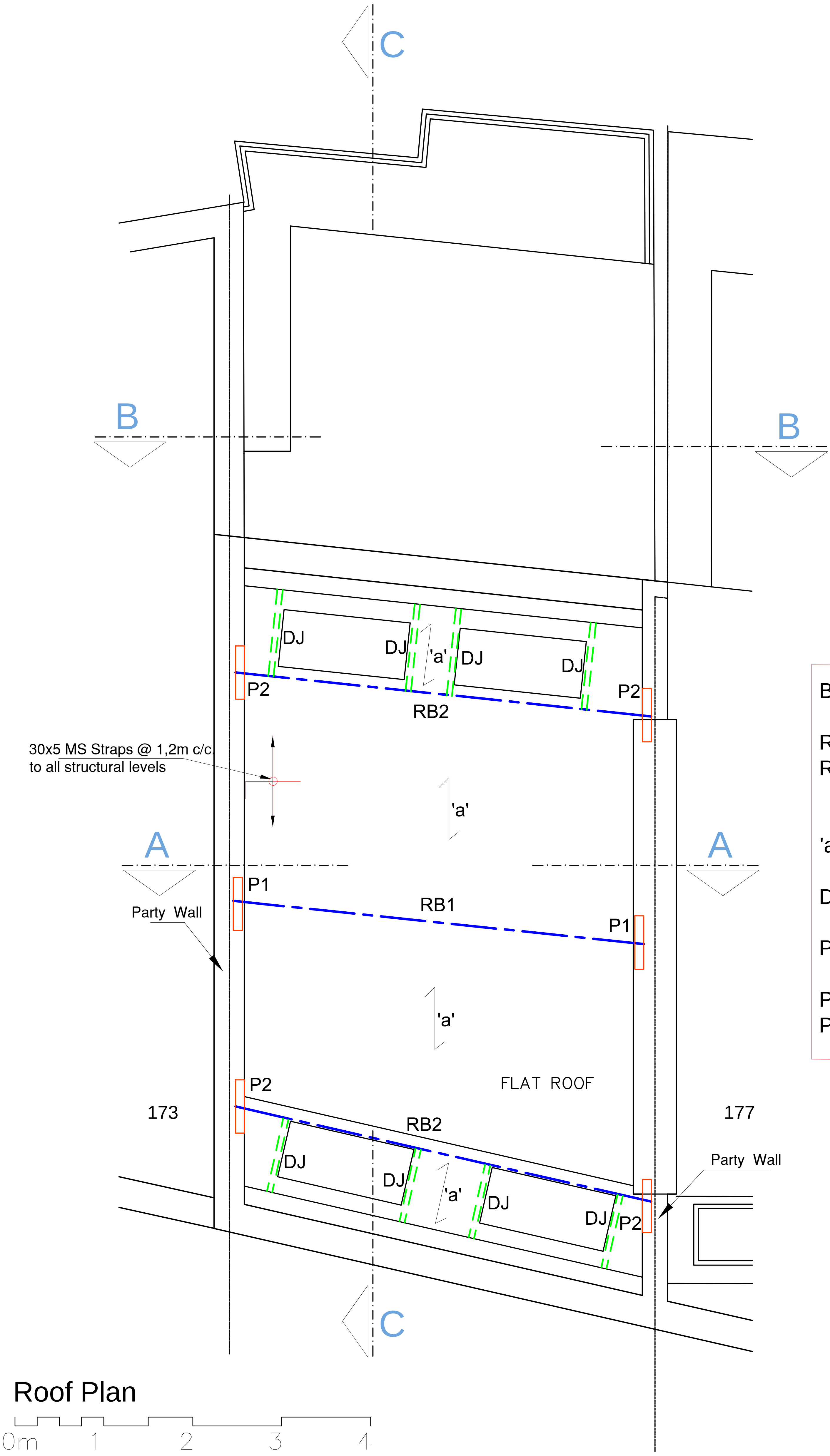

Roof Plan

BEAM SCHEDULE:

RB1 - 203 UC 52
RB2 - 203 UC 46

'a' - 200x50 Timb. Joists @ 400mm c/c.
DJ - Double Joists.

Padstones:

P1 - 550x100x20 St. Plate
P2 - 550x100x20 St. Plate

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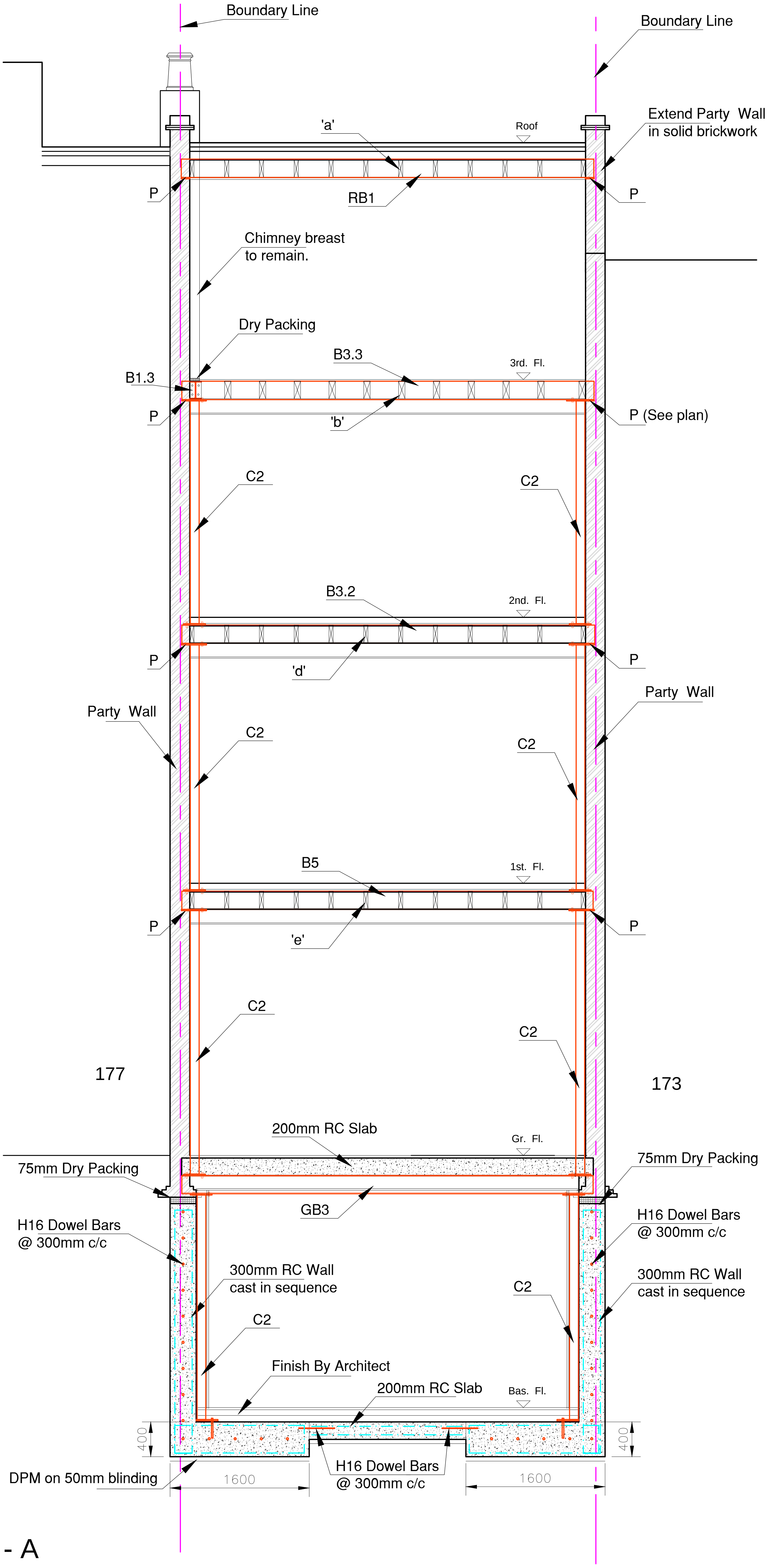

SECTION A - A

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Notes:
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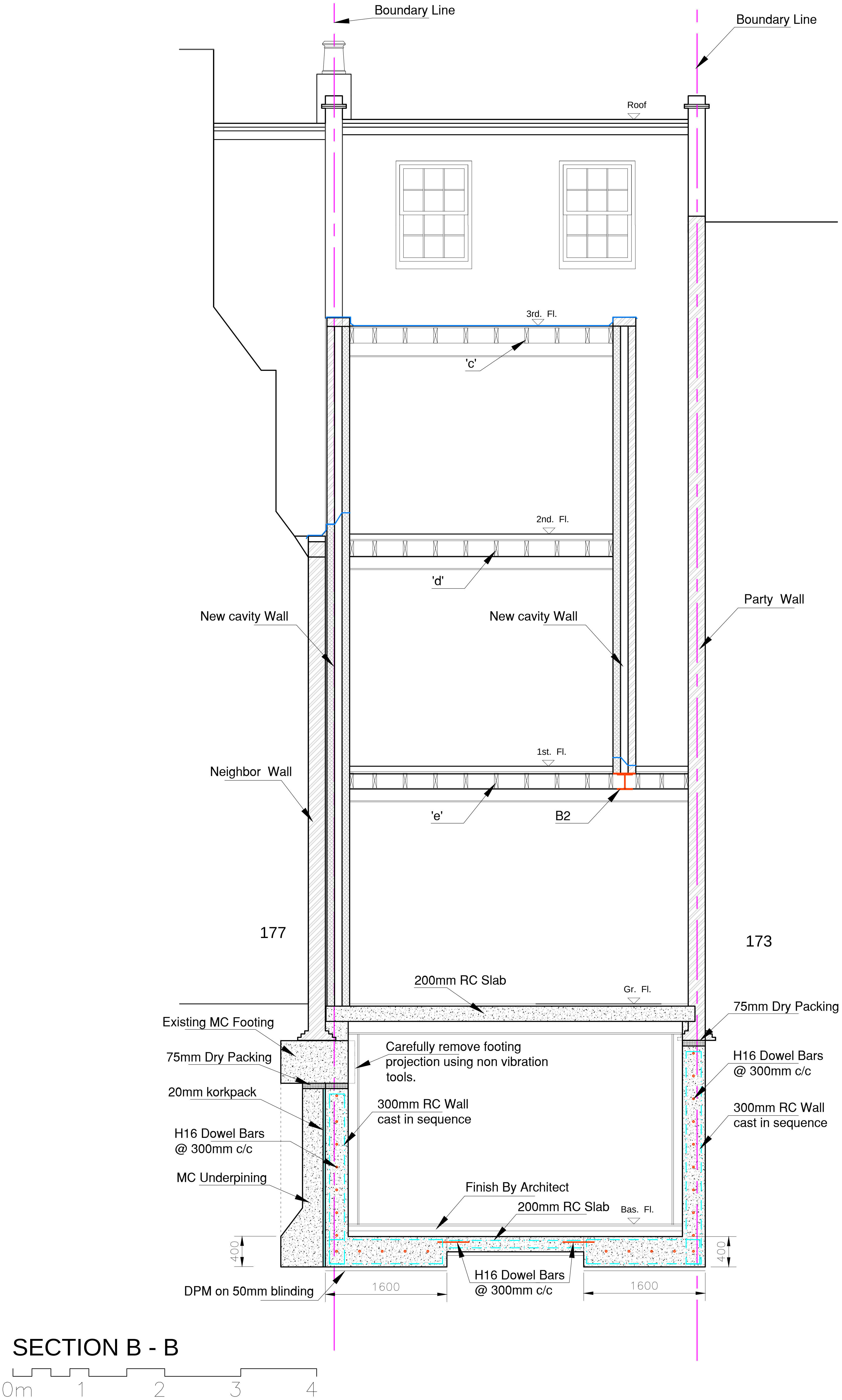

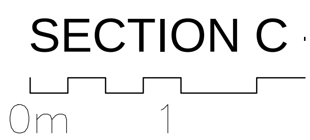

Issued For	Notes	Client	
Construction		-	

General Notes

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REPORT ALL DISCREPANCIES TO THE ARCHITECT/ENGINEER IMMEDIATELY.
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELATED ARCHITECT/ ENGINEERS DRAWINGS / DETAILS AND ALL OTHER RELEVANT INFORMATION.

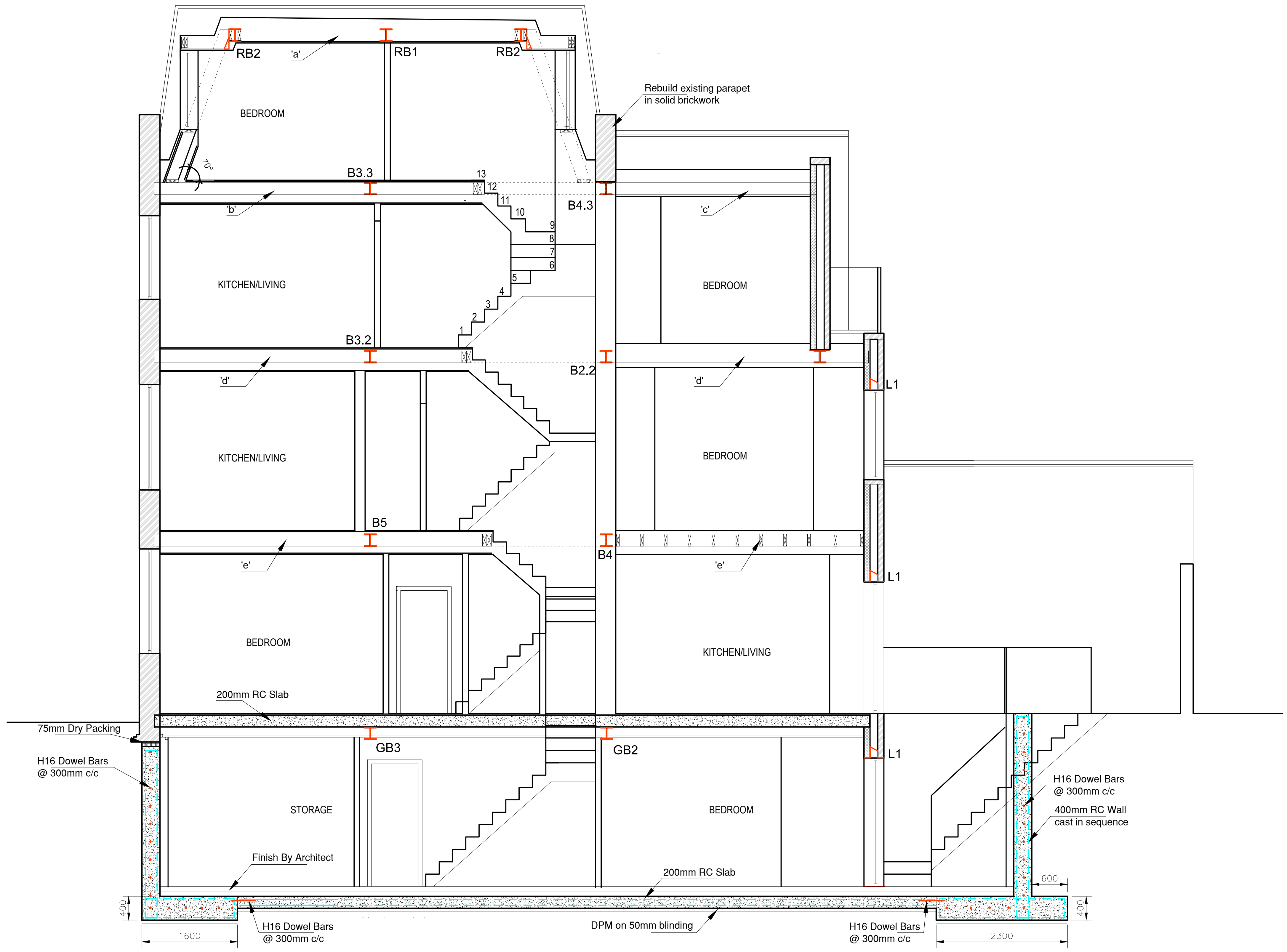
Project: 175 Arlington Road, London NW1 7EY

Drawing Title: SECTION B - B

Scale	Date	Job No.	Drawing No.
1:50@A3	Sep. 2023	1569	08

LIM ENGINEERING LTD
consulting engineers
15 Kinloch Drive, London, NW9 7LL
Tel: 020 8205 1427, Mob: 07775738210
Fax: 0208 205 1427

Notes:
For general notes see GN1



SECTION C - C

0m 1 2 3 4

Issued For	Notes	Client	
Building control		-	LIM ENGINEERING LTD consulting engineers 15 Kinloch Drive, London, Nw9 7LL - Tel: 020 8205 1427 Fax: 020 8205 1427

General Notes

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RELATED ARCHITECT/ ENGINEERS DRAWINGS / DETAILS AND ALL OTHER RELEVANT INFORMATION.

Project: 175 Arlington Road London NW1 7EY

Drawing Title: SECTION C - C

Scale	Date	Job No.	Drawing No.
1:75@A3	Sept. 2023	1569	09

175 Arlington Road, London NW1 7EY

GENERAL NOTES

1: All dimension to be verified on site.

2: All drawings to be read in conjunction with Architect Drawings.

3: All steelwork design and fabrication in accordance with BS 5950.

4: Apply 2 coats of red oxide primer to all steel prior to erection.

5. All structural steelwork to be Mild Steel Grade S275, designed, fabricated & erected in accordance with B5950 Part 1. Details of main connections are shown on drawings. All other connections to have a minimum of 2No. M20 8.8 grade bolts, sherardized or zinc plated (generally use 12mm end plate and 4No M20 8.8 grade bolts)

6: All welding to be min. 6 mm fillet welds.

6: All bolts to be grade 8.8. For Splice connection use HSFG Bolts.

7. Timber joists to be min. grade C16.

8: Double joist to be bolted together with M10 bolts + 63 dia. TP connectors and washers plate @ 400 mm c/c.

9: Connections:
Timber/Masonry: BAT SPH HANGERS
Timber/Timber: BAT JIFFY HANGERS or Framing Anchors.

10: Allow for Bat M305 Straps @ 1200 mm c/c for restraints to all structural levels.

11: Concrete Padstones to be grade C25 (1:2:4).

12: Temporary propping by Contractor.

13: All works to be approved by Building Control Officer.

14: Mass concrete foundation to be grade C20(SR). General RC to be grade C35.

15. Sulfate resistant concrete to be used below ground

16: All Waterproofing and Drainage to Architect specification.

17: New brickwork to be 21 N/mmsq. New blockwork to be min. 7 N/mmsq set in 1:1:6 mortar.

18. Underside of the foundation to be found on undisturbed ground and to be approved by Building Control Surveyor on site.

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Registered Office: 103 Clifford Gardens, London NW10 5JG, Registration No: 2804841

UNDERPINNING NOTES:

175 Arlington Road, London NW1 7EY – Basement

Excavation for underpinning is to be carried out with care and must not damage or disturb the existing footings. Sides of excavations are to be vertical and square with propping and boarding installed to maintain the stability of the adjoining ground. Boarding and propping is to be installed below the existing foundation where necessary to avoid disturbance thereto.

Underpinning is to be executed strictly in accordance with the sequence shown on the drawings, with not less than seven days lapsing between adjacent stages if sulphate resisting cement is used, or three days if rapid hardening sulphate resisting cement is used.

Excavation and construction of underpinning (including dry packing) is to be completed at each numbered stage prior to commencing work on the next section in the sequence.

The width of underpinning is to be no less than that of the existing foundation and is to be cast in bays not exceeding 1000mm in length. The head of each bay of underpinning is to be dry packed using 1:3 sharp sand and cement rammed in not earlier than 24 hours after the concrete has been cast to provide a dense, uniform and unvoided bearing surface.

Underpinning is to be carried out in accordance with the requirements of the adjoining owner's party-wall Surveyor and the Contractor is to give him the appropriate notice prior to commencing work.

The Contractor is to take precautions to control noise in accordance with the Control of Pollution Act 1974, Noise Abatement Act 1960 plus all amendments thereto and specific requirements of the Local Authority.

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REFURBISHMENT TO

175 Arlington Road
London, NW1 7EY

CALCULATIONS

Job No: 1569 **Date: October 2023**

General

These calculations cover the design of the structural elements for the proposed retaining walls.

The design is based on:

BS 6399-Loading
BS 8110-Concrete
BS 5950-Steel
BS 5628-Masonry
BS 5268-Timber

General loading

Roof:

- Dead load: $1.0\,\frac{\mathrm{kN}}{\mathrm{m}^2}$
- Imposed load: $0.75\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

Ground, floors:

- Dead load: $0.85\,\frac{\mathrm{kN}}{\mathrm{m}^2}$
- Imposed load: $1.5\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

Walls:

220 mm Brick wall

- $4.4\,\frac{\mathrm{kN}}{\mathrm{m}^2}$

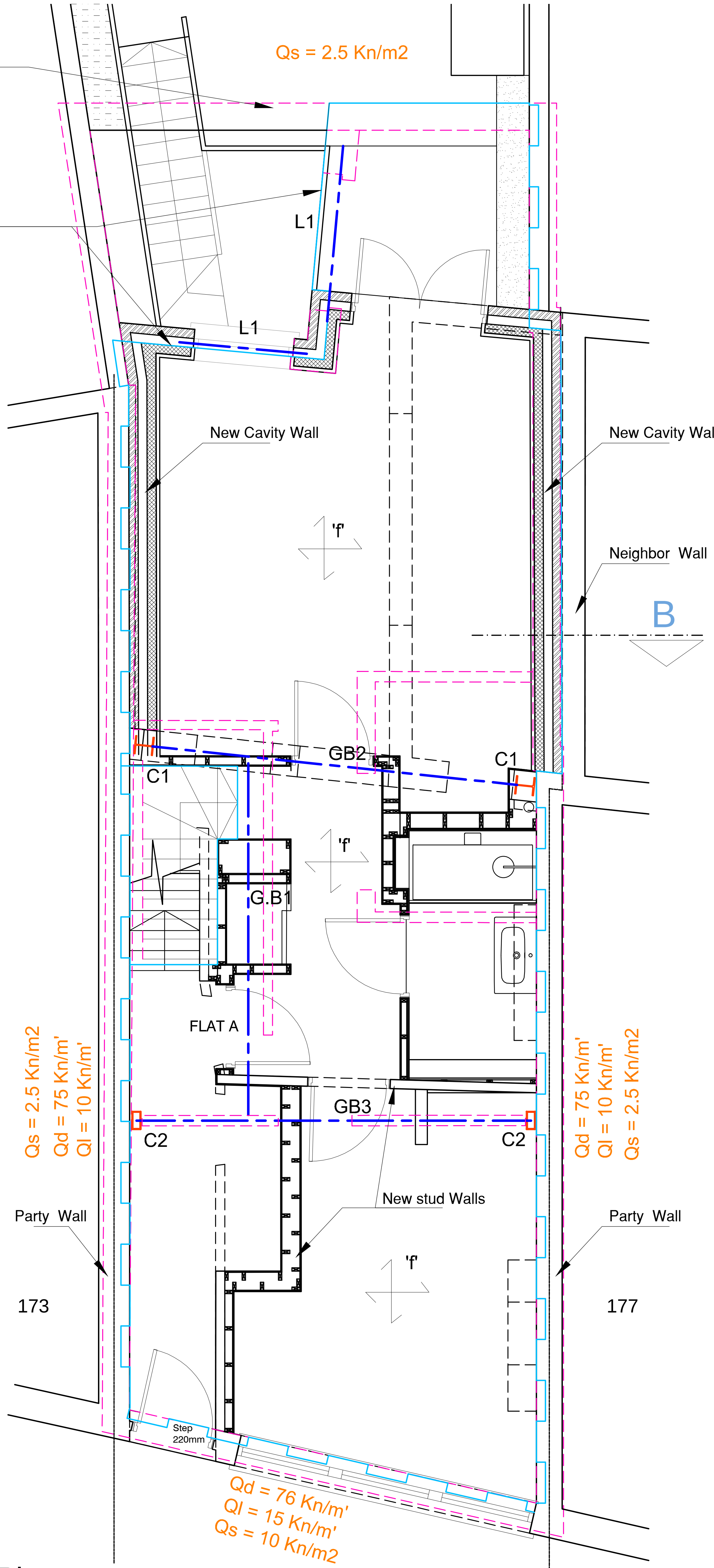
120 mm Brick wall

- 2.4

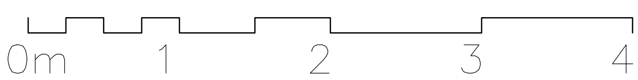
120 mm Stud wall

- 0.6

Notes:
For general notes see GN1



Ground Floor Plan



LOADING AT GROUND FLOOR LEVEL

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Construction		- -	LIM ENGINEERING LTD consulting engineers 15 Kinloch Drive, London, NW9 7LL Tel: 020 8205 1427, Mob: 07775738210 Fax: 0208 205 1427
General Notes		Project	
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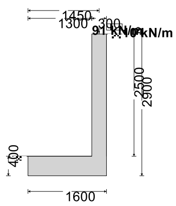
		Drawing Title	
		LOADING AT GR. FL. LEVEL	

Scale	Date	Job No.	Drawing No.
1:50@A3	Sep. 2023	1569	LL1

RETAINING WALL ANALYSIS (BS 8002:1994) - FRONT RETAINNING WALL

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type;	**Unpropped cantilever**
Height of retaining wall stem;	h_{stem} = **2500** mm
Thickness of wall stem;	t_{wall} = **300** mm
Length of toe;	l_{toe} = **1300** mm
Length of heel;	l_{heel} = **0** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1600** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1200** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2900** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **0** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\ \text{mm})$ = **0** mm
Density of wall construction;	γ_{wall} = **24.0** kN/m^3
Density of base construction;	γ_{base} = **24.0** 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;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2900** mm

Retained material details

Mobilisation factor;	M = **1.5**
Moist density of retained material;	γ_m = **21.0** kN/m^3
Saturated density of retained material;	γ_s = **23.0** kN/m^3

Design shear strength; $\phi' = \mathbf{25.8}$ deg

Angle of wall friction; $\delta = \mathbf{19.9}$ deg

Base material details

Soft clay

Moist density; $\gamma_{mb} = \mathbf{18.0}$ kN/m^3

Design shear strength; $\phi'_b = \mathbf{24.2}$ deg

Design base friction; $\delta_b = \mathbf{18.6}$ deg

Allowable bearing pressure; $P_{bearing} = \mathbf{150}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \surd(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))]^2) = \mathbf{0.347}$$

Passive pressure coefficient for base material

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

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 – \sin(\phi') = \mathbf{0.565}$

Loading details

Surcharge load on plan; Surcharge = **10.0** kN/m^2

Applied vertical dead load on wall; $W_{dead} = \mathbf{76.0}$ kN/m

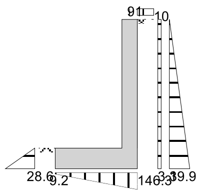
Applied vertical live load on wall; $W_{live} = \mathbf{15.0}$ kN/m

Position of applied vertical load on wall; $l_{load} = \mathbf{1450}$ mm

Applied horizontal dead load on wall; $F_{dead} = \mathbf{0.0}$ kN/m

Applied horizontal live load on wall; $F_{live} = \mathbf{0.0}$ kN/m

Height of applied horizontal load on wall; $h_{load} = \mathbf{0}$ mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{18}$ kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base} = \mathbf{15.4}$ kN/m

Applied vertical load; $W_v = W_{dead} + W_{live} = \mathbf{91}$ kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + W_v = \mathbf{124.4}$ kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff} = \mathbf{9.5}$ kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2 = \mathbf{28.8}$ kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a} = \mathbf{38.3}$ kN/m

Calculate stability against sliding

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = \mathbf{5.7}$ kN/m

Resistance to sliding; $F_{res} = F_p + (W_{total} - W_{live}) \times \tan(\delta_b) = \mathbf{42.5}$ kN/m

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = \mathbf{13.7}$ kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = \mathbf{27.9}$ kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a} = \mathbf{41.6}$ kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) = \mathbf{26.1}$ kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2 = \mathbf{12.3}$ kNm/m

Design vertical dead load; $M_{dead} = W_{dead} \times l_{load} = \mathbf{110.2}$ kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{dead} = \mathbf{148.6}$ kNm/m

Check stability against overturning

Total overturning moment; $M_{ot} = \mathbf{41.6}$ kNm/m

Total restoring moment; $M_{rest} = \mathbf{148.6}$ kNm/m

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Design vertical live load; $M_{live} = W_{live} \times l_{load} = \mathbf{21.8}$ kNm/m

Total moment for bearing; $M_{total} = M_{rest} - M_{ot} + M_{live} = \mathbf{128.7}$ kNm/m

Total vertical reaction; $R = W_{total} = \mathbf{124.4}$ kN/m

Distance to reaction; $x_{bar} = M_{total} / R = \mathbf{1035}$ mm

Eccentricity of reaction; $e = \text{abs}((l_{base} / 2) - x_{bar}) = \mathbf{235}$ mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) = \mathbf{9.2}$ kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) = \mathbf{146.3}$ kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **25.2** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **21.5** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **130.4** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **177.1** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **26.2** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **69.8** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **96** kN/m

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **8** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **38** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **67.5** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **105.5** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **36.5** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **17.2** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **189.1** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **242.8** kNm/m

Factored bearing pressure

Total moment for bearing; $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **137.3** kNm/m

Total vertical reaction; $R_f = W_{total\_f}$ = **177.1** kN/m

Distance to reaction; $x_{bar\_f} = M_{total\_f} / R_f$ = **775** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **25** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **120.9** kN/m^2

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **100.5** kN/m^2

Rate of change of base reaction; $\text{rate} = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **12.75** kN/m^2/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **104.3** kN/m^2

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2)$ = **102.4** kN/m^2

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2)$ = **100.5** kN/m^2

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

Material properties

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

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

Base details

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

Cover to reinforcement in toe; c_{toe} = **40** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **146.4** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **17.5** kN/m

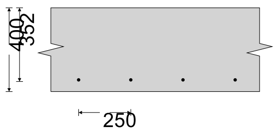
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **128.9** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **120.6** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **14.1** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **106.5** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **352.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.025**

Compression reinforcement is not required

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

$z_{toe} =$ **334** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **732** mm²/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base} =$ **520** mm²/m

Area of tension reinforcement required; $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) =$ **732** mm²/m

Reinforcement provided; **16 mm dia.bars @ 250 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov} =$ **804** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe}) =$ **0.366** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 =$ **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; $v_{c\_toe} =$ **0.446** N/mm²

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

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; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **22.6** kN/m

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **51.9** kN/m

Calculate shear for stem design

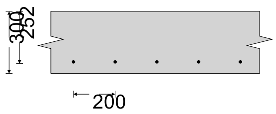
Shear at base of stem; $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f}$ = **74.5** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **32.8** kNm/m

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **53.6** kNm/m

Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a}$ = **86.4** kNm/m



Check wall stem in bending

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

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **252.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.039**

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} = **239** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **829** mm²/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **390** mm²/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **829** mm²/m

Reinforcement provided; **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov}$ = **1005** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.296** N/mm²

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

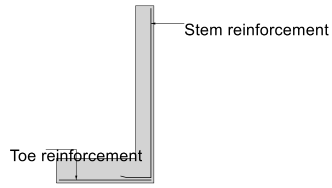
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.584** N/mm²

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

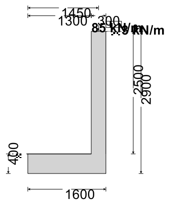
Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 250 mm centres - (804 mm^2/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994) - RETAINNING WALL UNDER PW NO 173 AND 177 ARLINGTON RD.

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type;	**Unpropped cantilever**
Height of retaining wall stem;	h_{stem} = **2500** mm
Thickness of wall stem;	t_{wall} = **300** mm
Length of toe;	l_{toe} = **1300** mm
Length of heel;	l_{heel} = **0** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1600** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1200** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2900** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **0** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **0** mm
Density of wall construction;	γ_{wall} = **24.0** kN/m^3
Density of base construction;	γ_{base} = **24.0** 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;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2900** mm

Retained material details

Mobilisation factor;	M = **1.5**
Moist density of retained material;	γ_m = **19.0** kN/m^3

Saturated density of retained material; γ_s = **21.5** kN/m^3

Design shear strength; ϕ' = **21.1** deg

Angle of wall friction; δ = **16.1** deg

Base material details

Firm clay

Moist density; γ_{mb} = **21.0** kN/m^3

Design shear strength; ϕ'_b = **12.2** deg

Design base friction; δ_b = **18.6** deg

Allowable bearing pressure; $P_{bearing}$ = **150** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.416}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{2.299}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 - \sin(\phi') = \mathbf{0.640}$

Loading details

Surcharge load on plan; Surcharge = **2.5** kN/m^2

Applied vertical dead load on wall; W_{dead} = **75.0** kN/m

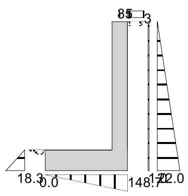
Applied vertical live load on wall; W_{live} = **10.0** kN/m

Position of applied vertical load on wall; l_{load} = **1450** 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/m^2

Vertical forces on wall

Wall stem;	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **18** kN/m
Wall base;	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **15.4** kN/m
Applied vertical load;	$W_v = W_{dead} + W_{live}$ = **85** kN/m
Total vertical load;	$W_{total} = w_{wall} + w_{base} + W_v$ = **118.4** kN/m

Horizontal forces on wall

Surcharge;	$F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **2.9** kN/m
Moist backfill above water table;	$F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **31.9** kN/m
Total horizontal load;	$F_{total} = F_{sur} + F_{m\_a}$ = **34.8** kN/m

Calculate stability against sliding

Passive resistance of soil in front of wall;	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.7** kN/m
Resistance to sliding;	$F_{res} = F_p + (W_{total} - W_{live}) \times \tan(\delta_b)$ = **40.1** kN/m

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge;	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **4.2** kNm/m
Moist backfill above water table;	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **30.9** kNm/m
Total overturning moment;	$M_{ot} = M_{sur} + M_{m\_a}$ = **35.1** kNm/m

Restoring moments

Wall stem;	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **26.1** kNm/m
Wall base;	$M_{base} = w_{base} \times l_{base} / 2$ = **12.3** kNm/m
Design vertical dead load;	$M_{dead} = W_{dead} \times l_{load}$ = **108.8** kNm/m
Total restoring moment;	$M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **147.1** kNm/m

Check stability against overturning

Total overturning moment;	M_{ot} = **35.1** kNm/m
Total restoring moment;	M_{rest} = **147.1** kNm/m

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Design vertical live load;	$M_{live} = W_{live} \times l_{load}$ = **14.5** kNm/m
Total moment for bearing;	$M_{total} = M_{rest} - M_{ot} + M_{live}$ = **126.6** kNm/m
Total vertical reaction;	$R = W_{total}$ = **118.4** kN/m
Distance to reaction;	$x_{bar} = M_{total} / R$ = **1069** mm
Eccentricity of reaction;	$e = \text{abs}((l_{base} / 2) - x_{bar})$ = **269** mm

Reaction acts outside middle third of base

Bearing pressure at toe;	$p_{toe} = 0\ \text{kN/m}^2$ = **0** kN/m^2
Bearing pressure at heel;	$p_{heel} = R / (1.5 \times (l_{base} - x_{bar}))$ = **148.7** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **25.2** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **21.5** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **121** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **167.7** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times h_{eff}$ = **7.4** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **71.6** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **79** kN/m

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.1** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **10.8** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **69.2** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **80** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **36.5** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **17.2** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **175.5** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **229.2** kNm/m

Factored bearing pressure

Total moment for bearing; $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **149.2** kNm/m

Total vertical reaction; $R_f = W_{total\_f}$ = **167.7** kN/m

Distance to reaction; $x_{bar\_f} = M_{total\_f} / R_f$ = **890** mm

Eccentricity of reaction; $e_f = abs((l_{base} / 2) - x_{bar\_f})$ = **90** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **69.5** kN/m²

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **140.1** kN/m²

Rate of change of base reaction; $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **-44.14** kN/m²/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{heel\_f} + (rate \times (l_{heel} + t_{wall})), 0\ kN/m^2)$ = **126.9** kN/m²

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{heel\_f} + (rate \times (l_{heel} + t_{wall} / 2)), 0\ kN/m^2)$ = **133.5** kN/m²

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{heel\_f} + (rate \times l_{heel}), 0\ kN/m^2)$ = **140.1** kN/m²

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

Material properties

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

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

Base details

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

Cover to reinforcement in toe; c_{toe} = **40** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **127.7** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **17.5** kN/m

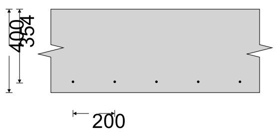
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **110.2** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **95.5** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **14.1** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **81.4** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **354.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.019**

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} = **336** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **556** mm²/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base} =$ **520** mm²/m

Area of tension reinforcement required; $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) =$ **556** mm²/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov}$ = **565** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe}) =$ **0.311** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2} =$ **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} = **0.396** N/mm²

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

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; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) =$ **6.4** kN/m

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 =$ **53.2** kN/m

Calculate shear for stem design

Shear at base of stem; $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} =$ **59.6** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 =$ **9.3** kNm/m

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 =$ **55** kNm/m

Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a} =$ **64.3** kNm/m

Check wall stem in bending

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

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **254.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.028**

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} = **241** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **612** mm^2/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **390** mm^2/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **612** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.235** N/mm^2

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

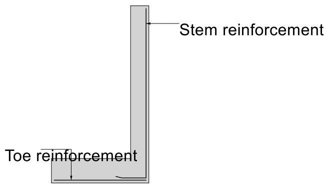
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.528** N/mm^2

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

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

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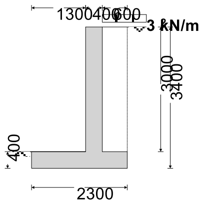
	ma	October 2023				

RETAINING WALL ANALYSIS (BS 8002:1994) - REAR RETAINNING WALL

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type;	**Unpropped cantilever**
Height of retaining wall stem;	h_{stem} = **3000** mm
Thickness of wall stem;	t_{wall} = **400** mm
Length of toe;	l_{toe} = **1300** mm
Length of heel;	l_{heel} = **600** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2300** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1100** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3400** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **0** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **0** 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;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3400** mm

Retained material details

Mobilisation factor;	M = **1.2**
Moist density of retained material;	γ_m = **18.0** kN/m^3

Saturated density of retained material; γ_s = **21.0** kN/m^3

Design shear strength; ϕ' = **29.3** deg

Angle of wall friction; δ = **14.2** deg

Base material details

Moist density; γ_{mb} = **18.0** kN/m^3

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

Design base friction; δ_b = **18.6** deg

Allowable bearing pressure; $P_{bearing}$ = **150** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.311}$$

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; $K_0 = 1 - \sin(\phi') = \mathbf{0.511}$

Loading details

Surcharge load on plan; Surcharge = **2.5** kN/m^2

Applied vertical dead load on wall; W_{dead} = **0.0** kN/m

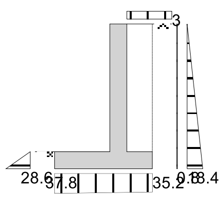
Applied vertical live load on wall; W_{live} = **0.0** kN/m

Position of applied vertical load on wall; l_{load} = **0** 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/m^2

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.3** kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **21.7** kN/m

Surcharge; w_{sur} = Surcharge $\times l_{heel}$ = **1.5** kN/m

Moist backfill to top of wall; $w_{m\_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **32.4** kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + w_{sur} + w_{m\_w}$ = **83.9** kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times$ Surcharge $\times h_{eff}$ = **2.6** kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **31.3** kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a}$ = **33.9** kN/m

Calculate stability against sliding

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.7** kN/m

Resistance to sliding; $F_{res} = F_p + W_{total} \times \tan(\delta_b)$ = **34.0** kN/m

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **4.3** kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **35.5** kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a}$ = **39.8** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **42.5** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **25** kNm/m

Surcharge; $M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2)$ = **3** kNm/m

Moist backfill; $M_{m\_r} = (w_{m\_w} \times (l_{base} - l_{heel} / 2) + w_{m\_s} \times (l_{base} - l_{heel} / 3))$ = **64.8** kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{sur\_r} + M_{m\_r}$ = **135.2** kNm/m

Check stability against overturning

Total overturning moment; M_{ot} = **39.8** kNm/m

Total restoring moment; M_{rest} = **135.2** kNm/m

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Total moment for bearing; $M_{total} = M_{rest} - M_{ot}$ = **95.4** kNm/m

Total vertical reaction; $R = W_{total}$ = **83.9** kN/m

Distance to reaction; $x_{bar} = M_{total} / R$ = **1137** mm

Eccentricity of reaction; $e = \mathrm{abs}((l_{base} / 2) - x_{bar})$ = **13** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **37.8** kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **35.2** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **39.6** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **30.4** kN/m

Surcharge; $w_{sur\_f} = \gamma_{f\_l} \times \text{Surcharge} \times l_{heel}$ = **2.4** kN/m

Moist backfill to top of wall; $w_{m\_w\_f} = \gamma_{f\_d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **45.4** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{m\_w\_f}$ = **117.8** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **6.9** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **74.4** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **81.3** kN/m

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **8**
kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **11.8** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **84.3** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **96.1** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **59.5** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **35** kNm/m

Surcharge; $M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2)$ = **4.8** kNm/m

Moist backfill; $M_{m\_r\_f} = (w_{m\_w\_f} \times (l_{base} - l_{heel} / 2) + w_{m\_s\_f} \times (l_{base} - l_{heel} / 3))$ = **90.7**
kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{m\_r\_f}$ = **189.9** kNm/m

Factored bearing pressure

Total moment for bearing; $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **93.9** kNm/m

Total vertical reaction; $R_f = W_{total\_f}$ = **117.8** kN/m

Distance to reaction; $x_{bar\_f} = M_{total\_f} / R_f$ = **797** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **353** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **98.4** kN/m^2

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **4** kN/m^2

Rate of change of base reaction; $\text{rate} = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **41.05** kN/m^2/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **45.1** kN/m^2

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2)$ = **36.9** kN/m^2

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2)$ = **28.6** kN/m^2

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

Material properties

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

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

Base details

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

Cover to reinforcement in toe; c_{toe} = **40** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2$ = **93.3** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **17.2** kN/m

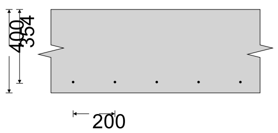
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base}$ = **76.1** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6$ = **87.6** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **14.9** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base}$ = **72.8** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **354.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.017**

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} = **336** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **497** mm²/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base}$ = **520** mm²/m

Area of tension reinforcement required; $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **520** mm²/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov}$ = **565** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.215** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2}$ = **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} = **0.396** N/mm²

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

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

Material properties

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

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

Base details

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

Cover to reinforcement in heel; c_{heel} = **40** mm

Calculate shear for heel design

Shear from bearing pressure; $V_{heel\_bear} = (p_{heel\_f} + p_{stem\_heel\_f}) \times l_{heel} / 2$ = **9.8** kN/m

Shear from weight of base; $V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base}$ = **7.9** kN/m

Shear from weight of moist backfill; $V_{heel\_wt\_m} = w_{m\_w\_f}$ = **45.4** kN/m

Shear from surcharge; $V_{heel\_sur} = w_{sur\_f}$ = **2.4** kN/m

Total shear for heel design; $V_{heel} = - V_{heel\_bear} + V_{heel\_wt\_base} + V_{heel\_wt\_m} + V_{heel\_sur}$ = **45.9** kN/m

Calculate moment for heel design

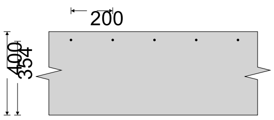
Moment from bearing pressure; $M_{heel\_bear} = (2 \times p_{heel\_f} + p_{stem\_mid\_f}) \times (l_{heel} + t_{wall} / 2)^2 / 6$ = **4.8** kNm/m

Moment from weight of base; $M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2)$ = **4.2** kNm/m

Moment from weight of moist backfill; $M_{heel\_wt\_m} = w_{m\_w\_f} \times (l_{heel} + t_{wall}) / 2$ = **22.7** kNm/m

Moment from surcharge; $M_{heel\_sur} = w_{sur\_f} \times (l_{heel} + t_{wall}) / 2$ = **1.2** kNm/m

Total moment for heel design; $M_{heel} = - M_{heel\_bear} + M_{heel\_wt\_base} + M_{heel\_wt\_m} + M_{heel\_sur}$ = **23.3** kNm/m



Check heel in bending

Width of heel; b = **1000** mm/m

Depth of reinforcement; $d_{heel} = t_{base} - c_{heel} - (\phi_{heel} / 2)$ = **354.0** mm

Constant; $K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu})$ = **0.005**

Compression reinforcement is not required

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

z_{heel} = **336** mm

Area of tension reinforcement required; $A_{s\_heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel})$ = **159** mm^2/m

Minimum area of tension reinforcement; $A_{s\_heel\_min} = k \times b \times t_{base}$ = **520** mm^2/m

Area of tension reinforcement required; $A_{s\_heel\_req} = \mathrm{Max}(A_{s\_heel\_des}, A_{s\_heel\_min})$ = **520** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_heel\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress; $v_{heel} = V_{heel} / (b \times d_{heel}) =$ **0.130** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; $v_{c\_heel} =$ **0.396** N/mm^2

$v_{heel} < v_{c\_heel}$ - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

Material properties

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

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

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; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) =$ **6.1** kN/m

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 =$ **57.9** kN/m

Calculate shear for stem design

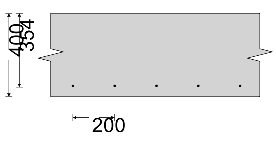
Shear at base of stem; $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} =$ **64** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 =$ **10.4** kNm/m

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 =$ **69.5** kNm/m

Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a} =$ **79.9** kNm/m



Check wall stem in bending

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

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **354.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.018**

Compression reinforcement is not required

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

$z_{stem} =$ **336** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **546** mm^2/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **520** mm^2/m

Area of tension reinforcement required; $A_{s\_stem\_req} = Max(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **546** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.181** 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/mm^2

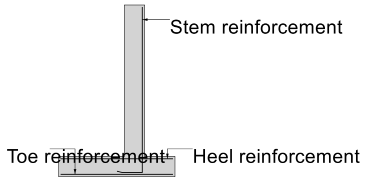
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.396** N/mm^2

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

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Heel bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

Project: ARLINGTON HOUSE, 220 ARLINGTON ROAD, CAMDEN, LONDON	Client: [redacted] Engineer: [redacted]	Boring Methods: LIGHT CABLE PERCUSSION 150 mm DIAMETER CASED 150 mm DIAMETER G.L. TO 1.50 m UNCASED 1.50 TO 10.00 m	Hole No. **BH1**
Ground Level: 26.66 m.A.O.D.	Coordinates: m.E. m.N.		Sheet 1 of 1
			Job No **10482**

WATER: Date/Time at Depth	Depth of Casing m	Depth to Water m	Inst.	STRATA: Description	Level m.A.O.D.	Depth m	SAMPLING/IN SITU TEST: Depth m	Type & No.	Blows/ Strength	LAB TESTING: % < 425	W %	W_P %	W_L %	ρ Mg/m³	C_u kN/m²	OTHER TESTS AND NOTES
				Made Ground (Reinforced concrete surfacing)	26.56	0.10										Hand excavated from ground level to 1.20m (120mins)
				Made Ground (Dark grey subangular fine-coarse gravel size ash, clinker, brick and mortar fragments with occasional pieces of glass, plastic, wood and pottery)			0.40	D1								CLEA screen with speciated polyaromatic hydrocarbons, total petroleum hydrocarbons and asbestos screen (D1)
					25.96	0.70	0.80	D2								
				Soft extremely closely fissured grey brown slightly sandy [silty] CLAY with some blue gleying and occasional partings of silt and sand (London Clay)			1.20-1.65	U1	(35)	100	31	22	69	1.96	34	
							1.65	D3								
				- occasional selenite crystals recorded at some depths			1.90	D4								
							2.10-2.60	U2	(35)		32			1.91	48	pH and Water Soluble Sulphate One-dimensional Consolidation
				- rare partially decayed root traces recorded to 2.60m approximately			2.60	D5								
							2.90	D6								
				- becoming firm with increasing depth			3.10-3.60	U3	(50)	100	31	23	73	1.95	54	
							3.60	D7								
							3.80	D8								
					22.46	4.20	4.10-4.20	U4	(100)							Undisturbed sample attempted at 4.10m - failed on recovery
				Very weak-weak brown CLAYSTONE [recovered as subangular fine-coarse gravel size fragments] (London Clay)	22.36	4.30	4.10-4.50	B1								Chisel in use between 4.20 and 4.30m (30mins)
				Firm extremely closely fissured dark grey brown [silty] CLAY (London Clay)			5.00-5.45	S1	N=12							
							5.45	D9								
				- occasional partings of ironstained red brown silt and fine sand recorded at some depths			6.00	D10								
							6.50-7.00	U5	(55)		31			1.95	71	
							7.00	D11								
							7.50	D12								pH and Water Soluble Sulphate
				- becoming stiff with increasing depth			8.00-8.45	S2	N=20							
																No groundwater recorded during drilling
							8.45	D13								
							9.00	D14								Borehole complete at 10.00m
							9.50-10.00	U6	(70)	100	31	26	74	1.97	130	On completion of drilling backfilled with bentonite/ cement grout from ground level to 10.00m
					16.66	10.00	10.00	D15								

Water Level observations during boring, depths below GL.

Strike	Depth Obs.	Depth after 5min	10 min	15 min	20 min

WATER
1 First Strike
2 Subsequent Strike
N - Overnight Depth
C- Completion Depth
S Seepage not rising

SAMPLE KEY
D Small disturbed sample
B Bulk disturbed sample
W Water sample
U Undisturbed sample
P Piston sample

TEST KEY
S Standard penetration test
C Cone penetration test
K Permeability test
V In situ vane test

BLOWS / STRENGTH
N = N value
26/150 blows, for 150mm, drive after seating
26*, blows for part or whole of seating drive only
(26) U sample blow count
V = Vane Strength - kN/m²

Fieldwork By	MW
Dates	02/10/06
Log	AJS

Sheet 1 of 1

BH1

RSA GEOTECHNICS LTD (01449) 723723

EXPLORATORY HOLE LOCATION PLAN

PROPOSED EXTENSION OF
ARLINGTON HOUSE, 220 ARLINGTON ROAD, CAMDEN, LONDON NW1

RSA GEOTECHNICS LIMITED

NOTE: All locations are approximate
Date 12 DECEMBER 2006
Scale NOT TO SCALE
Drawing No 10482/2