Providing Ground Solutions

Form Structural Design Limited

79 Avenue Road, London Borough Of Camden

Basement Impact Assessment - Revision 2

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1. NON-TECHNICAL SUMMARY

Card Geotechnics Limited (CGL) has been instructed by Form Structural Design Limited ("the Client") to undertake a Basement Impact Assessment (BIA) for a proposed development.

1. The site is located at 79 Avenue Road, the site location is shown in Figure 1.

2. The site is rectangular in shape, covering an area of approximately 1,393m^2 and is approximately 60.9m long and 22.5m wide. The site is currently occupied by a two-storey building which is approximately 11.1m long and 20.1m wide. The existing building has no below ground levels. The ground level is approximately 45.5mOD to 46mOD. The current site layout is shown as Figure 2. The existing site topographical survey is included in Appendix A.

3. To the north of the site there is the neighbouring property, 81 Avenue Road. This property is approximately 1.3m from the building structure at 79 Avenue Road and 0.6m from the party wall at its closest point. To the south of the property is No. 77 Avenue Road which is some 2.9m from the building at 79 Avenue Road and 1m from the party wall at its closest point.

4. Neither of the two neighbouring properties have an existing basement level, however planning permission has been granted to redevelop each neighbouring site to include basement levels. For the purpose of this report it has been assumed that the proposed development at 79 Avenue Road will be constructed first. To the north-east of the site there is a pedestrian footpath and highway, with a sewer running below the highway.

5. It is proposed to demolish the existing building on the site and construct a new residential property with a mixed single and double level basement. The proposed basement will increase the footprint of the existing building by 5m at the front of building and 7m at the rear. The proposed development drawings are included in Appendix B.

6. It is proposed to retain the basement excavation with a contiguous pile retaining wall. The basement will be constructed using the 'top down' construction method; with the ground floor slab constructed initially to provide a very stiff box during construction to control ground movements.

7. An intrusive site investigation has been conducted by CGL. The ground and groundwater conditions beneath the site comprise of a limited thickness of Made Ground/Topsoil (up to

0.5m) overlying up to 2.15m of Head Deposits and in turn the London Clay Formation to a proven depth of 15m below ground level.

8. No groundwater was encountered by CGL during the drilling of boreholes at the site. Later monitoring visits recorded shallow water within the Head Deposits, however levels varied substantially across the site and it is anticipated that groundwater is locally perched. The soils on site are generally cohesive and substantial groundwater ingress during excavation is not anticipated.

9. A Ground Movement Assessment has been carried out to assess the impacts of the proposed development on the neighbouring structures and infrastructure. This has been carried out using PDISP and WALLAP software with reference to CIRIA C760. The predicted building damage is Category 1 (Very Slight) on the Burland scale. This is subject to the provision of a temporary prop at basement level along the northern elevation of the retaining wall, opposite the property at 81 Avenue Road. The risk to the nearby footpath, highway and sewer is also considered negligible.

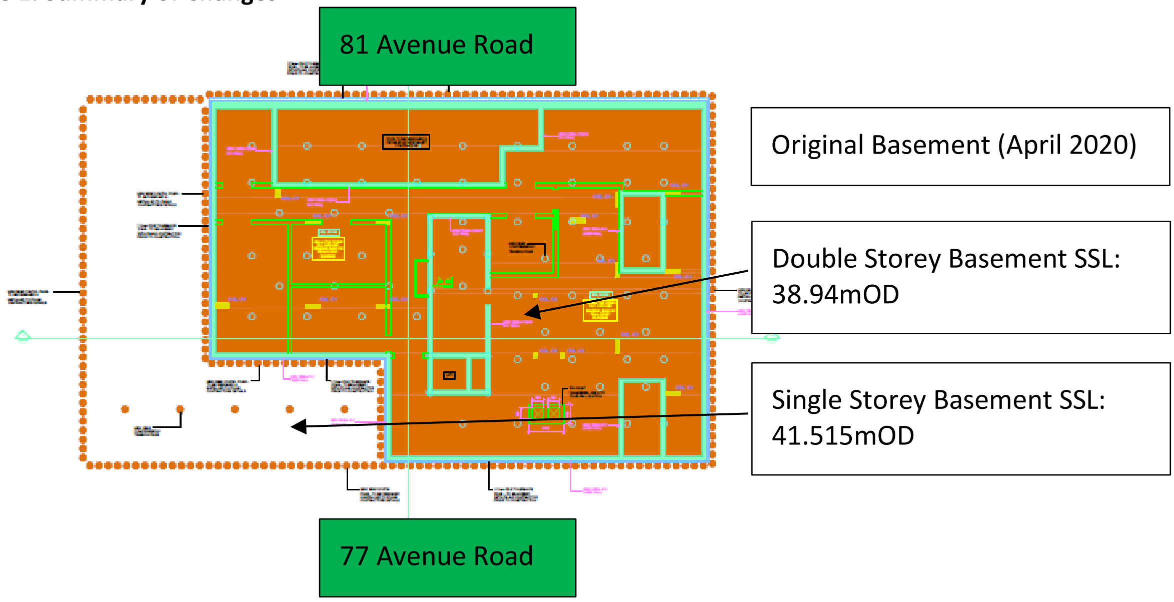
10. A structural monitoring strategy is recommended to control the works and impact to the neighbouring structures. Prior to construction commencing, baseline survey readings should be established, and a condition survey should be undertaken of adjacent buildings with any cracks and defects recorded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

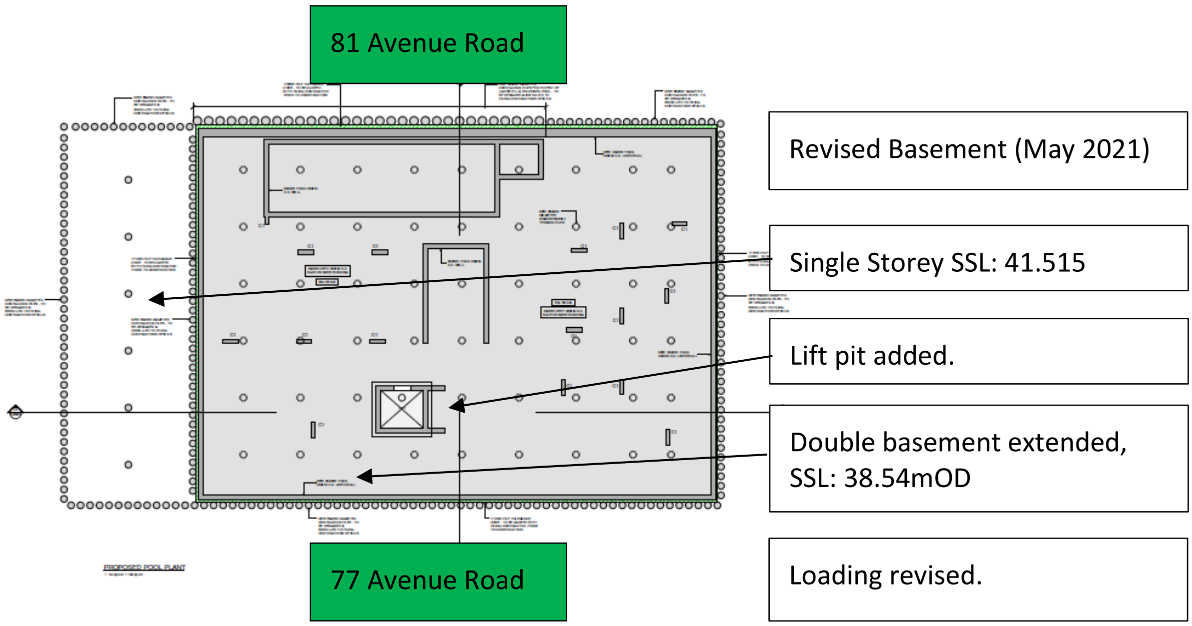
11. The BIA has identified no significant potential hydrogeological impacts and no impact to the wider hydrogeological environment.

12. 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.

This BIA has been updated to allow for revised pile loading, a revised extent of double basement, and slight deepening of the double storey basement level. In addition, a lift pit is to be excavated, this is remote from party wall structures.

Plate 1: Summary of Changes





2. INTRODUCTION

Card Geotechnics Limited (CGL) has been instructed by Form Structural Design Limited ("the Client"), hereafter referred to as Form SD, to undertake a Basement Impact Assessment (BIA) for the proposed development at 79 Avenue Road, Primrose Hill, NW8 6JD. It is proposed to demolish the existing two-storey residential building, to be replaced with a three storey residential property with a mixed single and double level basement.

The BIA approach follows the current planning procedure for basements adopted by the London Borough of Camden[1]. The report comprises the following elements:

- Desk Study;
- Screening;
- Scoping;
- Site Investigation;
- Ground Movement Assessment;
- Impact Assessment;
- Monitoring Strategy.

2.1 Sources of Information

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

- Site topographical survey (see Appendix A)
- Proposed development drawings (see Appendix B)
- Historic Ordnance Survey Maps[2]
- Second World War bomb damage records[3]

[1] London Borough of Camden. (2018). *Camden Planning Guidance – Basement*. March 2018.
[2] Old-Maps. (2019). [Online] Available at: https://www.old-maps.co.uk/#/Map/526895/183868/13/100765. [Accessed 12 November 2019].
[3] London Topographical Society. (2005). The London County Council Bomb Damage Maps 1939-1945.

- Geological mapping[4,5] and historical borehole records (see Plate 3 and Appendix C)
- Hydrogeological data[6]
- Lost Rivers of London[7]
- London Borough of Camden, Strategic Flood Risk Assessment[8]
- Site specific borehole and boundary wall foundation records (see Appendix D)
- Geotechnical laboratory results (see Appendix F)
- Structural loads (see Appendix G)

2.2 Site Layout

The site is located at 79 Avenue Road, within the London Borough of Camden. The site location is shown in Figure 1.

The site is rectangular in shape and measures approximately 60.9m long and 22.5m wide. The existing building is approximately 20.7m wide and 16.9m long and comprises a two-storey building with ground and first floor. The property is set back approximately 14.9m from the pedestrian footpath along the front of the site. The building is assumed to rest on shallow strip footings.

At the front of the site there is a driveway off Avenue Road and a garden area with a lawn. At the rear of the site is a garden with garden borders around the edge of the site and a lawn in the centre. The ground level at the front of the property ranges approximately from 45.6mOD to 45.7mOD. At the rear of the site the ground level varies from approximately 45.8mOD next to the building to 46.8mOD at the back of the rear garden.

There is a boundary wall which separates the site from 81 Avenue Road to the north and 77 Avenue Road to the south. The property at 81 Avenue Road is approximately 1.3m away from the existing building at 79 Avenue Road, has two storeys, and is approximately 17m wide and 15m long, with a garage attached to the north which is approximately 6.5m wide. The property at 77 Avenue Road is

[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] British Geological Survey. (2019). Geology of Britain viewer. [Online] available at: http://mapapps.bgs.ac.uk/geologyofbritain/home.html. [Accessed 29 October 2019].
[6] Natural England. (2019) Magic Map Application. [Online] Available at https://magic.defra.gov.uk/MagicMap.aspx. [Accessed 29 October 2019].
[7] Barton, N.J. (1992). The Lost Rivers of London: A Study of Their Effects Upon London and Londoners, and the Effects of London and Londoners on Them.
[8] URS (2014). *London Borough of Camden SFRA – Strategic Flood Risk Assessment*. July 2014.

approximately 2.9m away from 79 Avenue Road, has two-storeys and a converted roof space, and is approximately 16m wide and 15.5m long. It is understood that both neighbouring sites have had planning permission granted to redevelop them. The redevelopment of 81 Avenue Road includes a single-storey basement and at 77 Avenue Road a two-storey basement.

It is understood that the King's Scholars' Pond Sewer runs along Avenue Road, some 25.7m from the existing building on the site. The elevation of the sewer is not known.

2.3 Proposed Development

It is proposed to demolish the existing residential building on the site and replace this with the construction of a new residential building, comprising of two basement levels and three above ground storeys on piled foundations. The footprint of the proposed development will sit over the existing building.

The upper basement level (level B1) occupies the same footprint as the above ground levels and extends towards the front of the site (towards the east) by approximately 5m and towards the rear of the site (towards the west) by approximately 7m. The lower basement level (level B2) will occupy the same footprint as the ground level plan and extends towards the front of the site (towards the east) by approximately 5m. Figure 3 shows this layout graphically.

The proposed basement levels will be constructed using the 'top down' method, which will involve installing two contiguous pile walls around each basement level and installing the internal slab floors as excavation progresses. This creates a stiff box during construction to limit ground movements.

Proposed development drawings are provided in Appendix B.

3. DESK STUDY

3.1 Site History

The historical development of the site and the surrounding area has been traced from extracts of Ordnance Survey maps dating from 1850 to 1996[2].

The earliest map from 1850 shows the present-day carriageway along Avenue Road present and no houses built at this time in the general area.

The map from 1871-1872 shows a detached building on the present-day site with landscaping to front and rear of the building. The house present is not the same shape as the one currently on the site and is therefore likely to be an earlier building constructed on the site.

A post war map from 1953-1954 shows a number of ruins to the west of the site. The bomb damage map record[3] in Plate 2 shows that these houses were damaged, in some cases beyond repair, due to bomb damage. The property on the site sustained minor blast damage. The neighbouring property at 77 Avenue Road sustained general blast damage, while the neighbouring property at 81 Avenue Road sustained minor blast damage.

The 1960-1966 map which shows the present-day building present on the site. The ruined buildings to the west have been demolished. The map from 1967-1972 shows the present-day flats and houses along Queens Mead to the present to the west of the site were the ruined buildings were present.

The history of the site does not indicate the potential for substantial contamination. Areas of Made Ground may be present below and around the footprint of the former building on the site. The Made Ground is likely to comprise of demolition waste and hardcore from the former building.

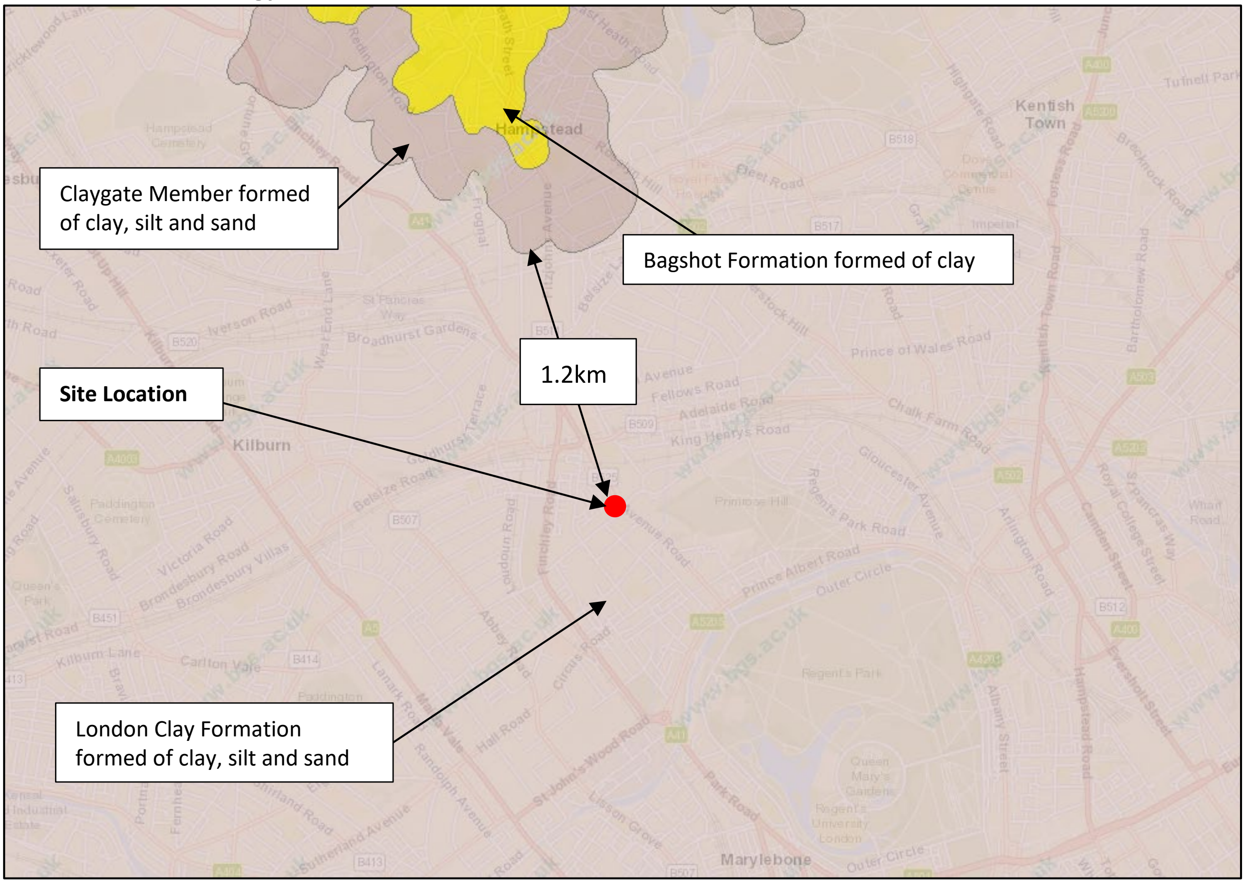
Plate 2. WW2 bomb damage record [black – total destruction, purple – damaged beyond repair, dark red – seriously damaged (doubtful if repairable at cost), light red – seriously damaged (but repairable at cost), orange – general blast damage, yellow – blast damage minor in nature]

3.2 Published and Unpublished Geology

The British Geological Survey (BGS) geological sheet[4] indicates that the site is underlain by the bedrock geology of the London Clay Formation, which is characteristically formed of overconsolidated clay deposits, with minor constituents of silt and sand. While the map shows no superficial deposits beneath the site, the map does show Head Deposits in close proximity to the site.

Extracts from the BGS Geology of Britain viewer[5] for the bedrock geology are presented in Plate 3.

Plate 3. Bedrock Geology



A review has been undertaken of ground conditions encountered in historic BGS borehole records. A summary is presented in Table 1.

Table 1. BGS historic borehole logs

Borehole	***Depth (mbgl)***	***Coordinates (m)***	***Depth to Top of Stratum (mbgl)***				
			Made Ground	***Topsoil***	***Head Deposits***	***Weathered London Clay Formation***	***London Clay Formation***
TQ28SE353	9.14	526720E, 183790N	0	-	0.3	0.76	
TQ28SE255/A	6.1	526540E, 183810N	0	-	-	0.23	
TQ28SE255/A-I	7.62	526540E, 183810N	-	-	0	0.23	
TQ28SE590	15.24	526740E, 184090N	0	-	-	0.45	10.66
TQ28SE591	9.14	526720E, 184070N	-	0	-	0.5	
TQ28SE592	11.12	526740E, 184040N	0	-	-	0.45	10.7
TQ28SE593	13.72	526690E, 184090N	-	0	-	0.5	11.32
TQ28SE594	12.19	526690E, 184040N	-	0	-	0.61	10.31
TQ28SE595	15.24	526660E, 184060N	0	-	-	1.22	11.4
TQ28SE596	15.24	526670E, 184010N	-	0	-	0.45	10.19

3.3 Hydrogeology

The groundwater conditions within these boreholes are presented in Table 2. The table shows that there were no recorded groundwater strikes in the boreholes listed. The natural strata encountered (Head Deposits, Weathered London Clay Formation and London Clay Formation) in these boreholes are characteristically of a very low permeability and therefore do not contain a continuous body of groundwater, however these strata can contain isolated limited lenses of perched water. The records are therefore in line with what is expected.

Table 2. BGS historic borehole logs – Groundwater strikes

Borehole	*Depth (mbgl)*	*Coordinates (m)*	*Groundwater Strike (mbgl)*	*Standing Water Level (mbgl)*
TQ28SE353	9.14	526720E, 183790N	-	-
TQ28SE255/A	6.1	526540E, 183810N	-	-
TQ28SE255/A-I	7.62	526540E, 183810N	-	-
TQ28SE590	15.24	526740E, 184090N	N/R[a]	-
TQ28SE591	9.14	526720E, 184070N	N/R[a]	-
TQ28SE592	11.12	526740E, 184040N	N/R[a]	-
TQ28SE593	13.72	526690E, 184090N	N/R[a]	-
TQ28SE594	12.19	526690E, 184040N	N/R[a]	-
TQ28SE595	15.24	526660E, 184060N	N/R[a]	-
TQ28SE596	15.24	526670E, 184010N	N/R[a]	-

a. N/R – Not Recorded

The aquifer designation for the area around the site is presented in Plate 4, which shows that London Clay Formation below the site is classed as unproductive.

Plate 5 shows that the site is located in a groundwater source protection zone, however this is likely to be related to the groundwater present at depth within the deep aquifer below the London Clay Formation. The BGS geological sheet[4] for the area shows chalk present below the site at approximately -60mOD. The sheet also indicates that the London Clay is approximately 50m thick below the site. Given the thickness of the London Clay Formation and depth to the chalk, the basement will not have an impact on the deep aquifer.

3.4 Hydrology, Drainage and Flood Risk

The nearest surface water feature is the Regents Canal, located 780m to the south-east. The nearest lost river is the River Tyburn which originally ran some 200m to the east of the site[7]. It is understood that the River Tyburn has been incorporated into the King's Scholar Sewer[8] which is located some 30m northeast from the existing building onsite, running along Avenue Road. The depth of the sewer is unknown.

A review of the London Borough of Camden Strategic Flood Risk Assessment (SFRA) shows the site is located within Critical Drainage Area 'Group3_005' and is not located within a Local Flood Risk Zone. The SFRA also shows that the surface water flood risk for the site is classed as low (1 in 1000 year) to medium (1 in 100 year), with the medium flood risk relating to the area located in the rear garden. The 1 in 1,000-year flood event hazard risk across the majority of the site is low (flood water <0.75m). The SFRA shows the road at the front of the site, Avenue Road, flooded during the 2002 floods however, the SFRA shows that none of the properties along the road flooded. Additionally, the SFRA shows that there has been one recorded internal sewer flooding events and zero external sewer flooding events occurring in the NW8 6 postcode area between 2004 and 2014.

The proposed basement B1 level extends out towards the rear garden which has a slope of <1°. While the SFRA shows that the rear garden has a medium surface water flood risk, the proposed development will still retain at least 50% of the rear garden with soft landscaping. The proposed development will also have groundwater protection measures in place to prevent groundwater infiltration into the basement.

Plate 4. Aquifer designation map

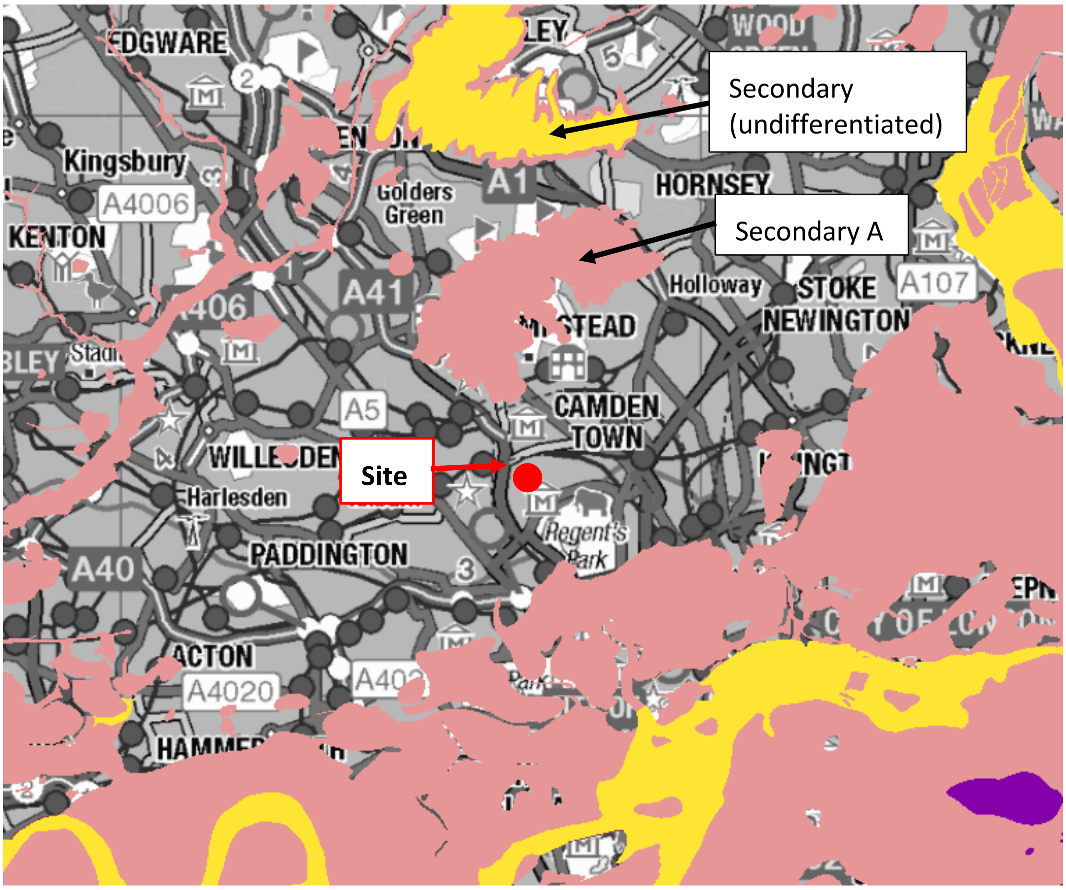
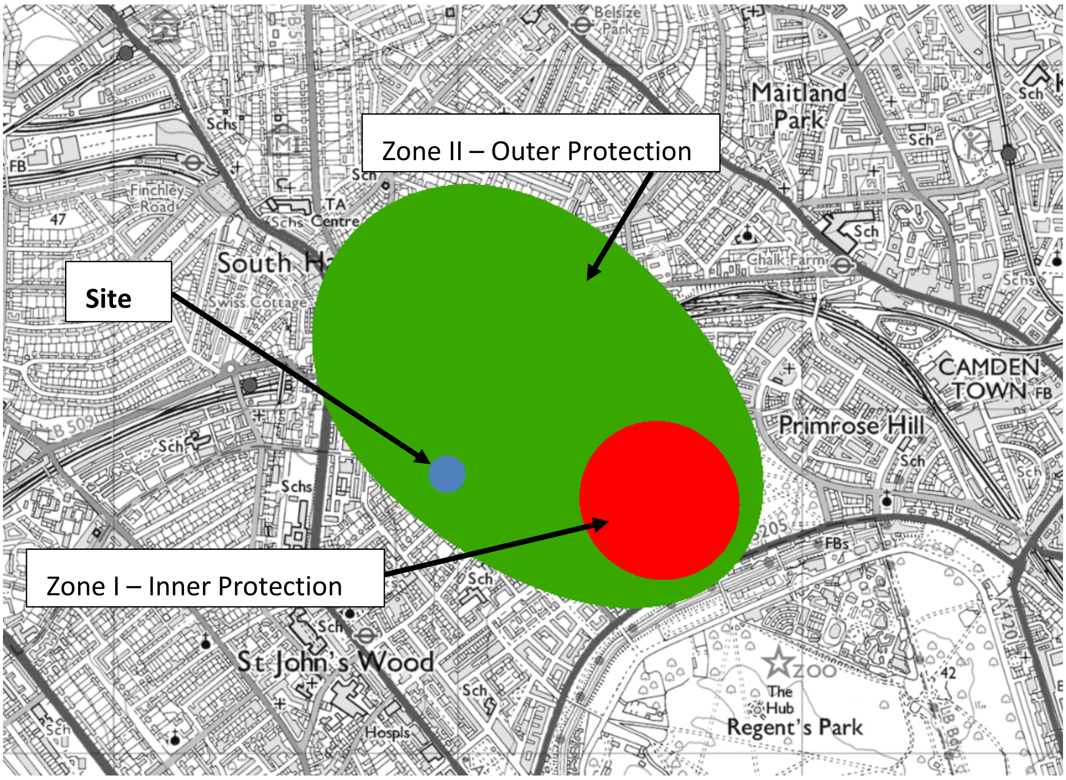


Plate 5. Groundwater source protection zones



4. SCREENING

4.1 Introduction

A screening assessment has been carried out 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 construction. Where 'yes' or 'unknown' can be simply answered with no analysis, these answers have been provided.

4.2 Subterranean (groundwater) flow

This section answers questions relating to subterranean (groundwater) flow in Table 3 below.

Table 3. Subterranean (groundwater) flow

Question	Response	Action Required
1a. Is the site located directly above an aquifer?	No. The site is not located above an aquifer.	None
1b. Will the proposed basement extend beneath the water table?	No. The London Clay Formation 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 Formation.	None
2. Is the site within 100m of a watercourse, well (used/discussed) or potential spring line?	Yes. The King's Scholars' Pond Sewer which culverts the River Tyburn is located some 27.5m northeast from the existing building onsite, running along Avenue road.	Assessment
3. Is the site within the catchment of the pond chains on Hampstead Heath?	No. The ponds are located approximately 2.7km north of the site.	None
4. Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	Yes. It is understood that the proposed development will increase the overall area of hardstanding across the site. However, this is unlikely to have a significant impact on surface water runoff, as the site is underlain by the London Clay Formation which typically has a very low permeability.	Third party Flood Risk Assessment[9] to consider the impact on surface water infiltration.
5. As part of the site drainage, will more water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via soakaways and/or SUDS)?	No. The ground conditions for the area the site is located are unsuitable for discharging water to the ground. However alternative forms of SUDS may be considered.	None
6. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on the Hampstead Heath) or spring line.	No.	None

In summary, the ground conditions expected across the site are of very low permeability and therefore only very low groundwater flow is anticipated. It is noted that the King's Scholars' Pond Sewer which culverts the River Tyburn is located some 27.5m northeast of the existing building. It is expected that

[9] Form Structural Design Limited. (2019). 79 Avenue Road, London, NW8 6JD – Flood Risk Assessment. October 2019.

the proposed development will increase the total area of hardstanding, however the impact on the surface water infiltration will be considered by a third party report[9].

4.3 Slope/Land Stability

This section answers questions relating to slope/land stability in Table 4.

Table 4. Slope/Land Stability

Question	*Response*	*Action Required*
1. Does the site include slopes, natural or manmade, greater than 7° (approximately 1 in 8)?	No. The gradient across the site is approximately <1°. The gradient along the south-west rear garden wall is 5°.	None
2. Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7° (approximately 1 in 8)?	No. The proposed development will not alter the existing landscaping.	None
3. Does the development neighbour land, including railway cuttings and the line, with a slope greater than 7° (approximately 1 in 8)?	No.	None
4. Is the site within a wider hillside setting in which the slope is greater than 7° (approximately 1 in 8)?	No.	None
5. Is the London Clay the shallowest strata at the site?	Yes. The stratum will need to be confirmed with an intrusive investigation.	Investigation
6. Will any tree/s 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?	Yes. To enable the proposed development to be undertaken it will be necessary to remove some of the existing trees on the site. Given that the proposed development foundations will be below the depth of the tree roots, it is expected that this will not have an impact on the proposed development. However, it is noted that trees felled on the site boundary may have impact on neighbouring properties.	Assessment
7. Is there a history of seasonal shrink-swell subsidence in the local area, and/or evidence of such effects at the site?	Unknown. The site is situated on the London Clay Formation which typically has a high shrink-swell capacity. However, the proposed development will be designed to accommodate this.	None
8. Is the site within 100m of a watercourse or potential spring line?	Yes. The nearest water feature is the King's Scholars' Pond Sewer which incorporates the River Tyburn some 27.5m northeast of the existing site buildings.	Assessment
9. Is the site within an area of previously worked ground?	No.	None
10. Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction.	No. The site is within the London Clay Formation, which is defined as an unproductive aquifer.	None
11. Is the site within 50m of the Hampstead Health ponds?	No. The ponds are located approximately 2.7km north of the site.	None
12. Is the site within 5m of a highway or pedestrian right of way?	No. The proposed development is set back 14.9m from the footpath.	None
13. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Yes. It is understood that the existing neighbouring properties do not have an existing below ground levels. However, both neighbouring properties have planning permission granted to redevelop the sites which will include one to two basement levels.	Impact Assessment
14. Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?	No. The nearest tunnel is located approximately 290m north of the site.	None

The ground and groundwater conditions on site require investigation and the impact of ground movements caused by the proposed development will require assessment. The impact of tree removal could cause the ground around the trees to swell. An impact assessment therefore needs to be undertaken.

4.4 Surface Flow and Flooding

This section answers questions relating to surface flow and flooding in Table 5.

Table 5. Surface water and flooding

Question	*Response*	*Action Required*
1. Is the site within the catchment of the pond chains on Hampstead Health?	No. The ponds are located approximately 2.7km north of the site.	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?	Unknown. While the overall area of hardstanding will increase, it is not known if this will significantly impact on the amount of surface water runoff, as the site is underlain by the London Clay Formation. The stratum typically has a very low permeability which limits surface water infiltration.	Third party Flood Risk Assessment[9] to consider the impact on surface water runoff.
3. Will the proposed basement development result in a change in the proportion of hard surfaced/paved external areas?	Yes. It is understood that the proposed development will increase the overall area of hardstanding across the site. However, this is unlikely to have a significant impact on surface water runoff, as the site is underlain by the London Clay Formation which typically has a very low permeability.	Third party Flood Risk Assessment[9] to consider the impact on surface water infiltration.
4. Will the proposed basement result in a change to the profile of the inflows (instantaneous and long-term) of surface water being received by adjacent properties or downstream watercourses?	No. The proposed development will not result in a significant change in the existing area of hardstanding. Therefore, a change in surface water received by neighbouring properties is not expected.	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 quality of the surface water is not expected to be impacted by the proposed development.	None
6. Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby water features?	No. The site is located outside of a local flood risk zone. While the road at the front of the property flooded in 2002, none of the properties along the road, including the site, have flooded. Additionally, the proposed basement does not neighbour any surface water features.	None

In summary, the proposed development is not expected to result in a change in surface water infiltration, however a third party report will need to be undertaken to consider the impact on surface water flow[9]. The site is not located within a flood risk area.

4.5 Non-technical Summary of Screening Process

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

- Slope/Land Stability – The site is understood to be underlain by deposits from the London Clay Formation, confirmation is required through ground investigation.

- Groundwater flow/flooding – Whilst significant groundwater is not anticipated on site, a ground investigation should be carried to record water levels and determine the influence of the King's Scholar sewer on the groundwater level.

- Slope/Land Stability – The proposed development will involve the removal of some trees on the site. While the removal of the trees is not expected to impact on the proposed development, it may impact on the neighbouring property foundations.

- Slope/Land Stability – The proposed development will increase the differential foundation depth the existing properties, the impact of ground movements caused by the proposed development on the neighbouring properties should be assessed.

- Groundwater flow/surface water and flooding – It is understood that the proposed development will increase the overall area of hardstanding across the site. The impact from this has been assessed in a third party Flood Risk Assessment[9], which also sets out the drainage strategy for the proposed development. No further action has been undertaken on this issue in this report.

5. SCOPING

5.1 Introduction

Based on the findings of the screening process, the following considerations have been brought forward to scoping for further assessment.

5.2 Slope/Land Stability – Presence of London Clay Formation on the site

It is understood from the desk study that the London Clay Formation is likely to be present across the site. This should be confirmed by undertaking investigation works. The investigation should also record groundwater levels present at the site.

5.3 Slope/Land Stability – Removal of trees

The proposed development will involve the removal of some trees within the site boundary. The removal of trees within cohesive soil causes the ground to swell and heave. If this occurs within the vicinity of a foundation it can cause differential movements which can damage the supported structure. Given that the proposed development is to be supported by pile foundations, it is not expected to be impacted. However, the removal of the trees may impact the neighbouring properties if they lie within the influence zone of the removed trees.

The nearest tree is some 8m high and located approximately 7m from 79 Avenue Road (see Appendix A). Using the NHBC[10] guidance, the influence zone of the tree is 10m radius from the tree trunk (assuming a neighbouring foundation depth of 1m and the tree is a broad leaf, high water demand species). Given that the building is located towards the outside of the influence zone, it is expected that the removal of the tree will cause negligible ground movements.

5.4 Slope/Land Stability – Differential foundation depth

The proposed development will increase the differential foundation depth between the two existing neighbouring properties at 81 Avenue Road and 77 Avenue Road. This could cause ground movements which could result in an unacceptable level of damage to the neighbouring properties. Therefore, a Ground Movement Assessment (GMA) will be undertaken using empirical and numerical methods. The results of which will be used to determine building damage category for each neighbouring property.

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

6. SITE INVESTIGATION

6.1 Fieldwork

An intrusive site investigation was undertaken by CGL on 5th September and 21st October 2019. The site investigation was undertaken broadly in accordance with the principals set out in BS 5930:2015[11]. The investigation comprised the drilling of one 15m Cable Percussion (CP) borehole, one 5m Window Sample (WS) borehole, and four Foundation Inspection Pits (FIPs) to 1.2mbgl. The FIPs were excavated below the party wall, in order to locate the extent of the wall foundations. All the borehole and FIP records can be found in Appendix D. The locations of the exploratory holes are presented in Figure 2.

In-situ testing was undertaken in the CP borehole in the form of Standard Penetration Tests (SPTs). Disturbed and undisturbed (U100) were collected for geotechnical and chemical testing.

Two of the FIPs (TP1 and TP2) were unable to confirm the base of the wall due to layer of concrete at the base of the hole. The concrete layer was encountered close to the invert level of the nearby foul water drain.

Following the completion of the two boreholes, monitoring wells were installed to monitor the groundwater level in the Head Deposits. A monitoring visit was completed on 21st October 2019 the monitoring record is provided in Appendix E.

6.2 Laboratory Testing

A total of 15 soil samples were tested by i2 Analytical Limited (UKAS and MCERTS accredited) for classification, strength and chemical testing. The following tests were carried out:

- Moisture content;
- Atterberg Limits testing including Plastic Limit (PL), Liquid Limit (LL) and Plasticity Index (PI);
- Quick undrained unconsolidated triaxial compression testing;
- pH and sulphate testing for buried concrete classification;
- Chemical testing, including asbestos screening and identification.

The laboratory results are provided in Appendix F.

[11] British Standards Institute. (2015). *BS 5930:2015 – Code of practice for ground investigations*. July 2015.

7. GROUND AND GROUNDWATER CONDITIONS

7.1 Ground Conditions

The ground conditions encountered during the CGL site investigation are summarised in Table 6.

Table 6. Summary of ground conditions encountered during the CGL site investigation

Stratum	*Depth to Top of Stratum (mbgl) [mOD]*	*Typical Thickness (m)*
Hardstanding and sub-base material or topsoil comprising very loose brown dark grey slightly clayey silty gravelly fine to coarse sand. [MADE GROUND]	0 [45.65]	0.35 to 0.5
Firm brown and grey mottled slightly gravelly slightly silty CLAY. Gravel is sub-rounded to rounded, fine to medium of flint. [HEAD DEPOSITS]	0.35 to 0.50 [45.54 to 45.30]	2.15
Firm to stiff orange brown occasionally mottled grey slightly silty CLAY. With frequent fine selenite crystals. (Present in BH1 only) [WEATHERED LONDON CLAY FORMATION]	2.5 [43.15]	6.2
Firm to stiff grey slightly silty CLAY. [LONDON CLAY FORMATION]	8.7 [36.95]	Not proven. Borehole terminated at 15mbgl (30.65mOD)

The observed strata are discussed separately in the following sections together with the results of the geotechnical tests. All information mentioned in the following sections is based on the results of the CGL site investigation. A conceptual site model is shown in Figure 4.

7.2 Made Ground/Topsoil

A limited thickness of Made Ground or Topsoil was encountered on site, typically between 0.35m to 0.5m. Where the Topsoil was encountered it comprised of very loose brown dark grey slightly clayey silty gravelly fine to coarse sand. The gravel was angular to rounded, fine to medium of flint, brick and ceramic. Fine to medium rootles were also encountered.

7.3 Head Deposits

Head Deposits were recovered in both boreholes from between 0.35mbgl and 0.5mbgl. The stratum comprised firm brown and grey mottled slightly gravelly slightly silty clay. The gravel was sub-rounded to rounded, fine to medium of flint.

A single SPT was carried out in the stratum which recorded an ‘N’ value of 18, corresponding to an undrained shear strength of 81kPa (assuming an f_1 value of 4.5N[12]). No other in-situ testing was undertaken in the stratum.

[12] Stroud, M.A. (1975). *The standard penetration test in incentive clay and soft rock.* Proceedings of the European Symposium of Penetration Testing, 2 p. 367-375.

Geotechnical laboratory testing carried out on the disturbed soil recorded the following:

- Moisture content values of 24 to 29%;
- Liquid Limit (LL) values of 63 to 78%;
- Plasticity Limit (PL) values of 26 to 28%;
- Plasticity Index values of 37 to 50%;
- Percentage passing 425µm sieve is 93 to 100%.

These results indicate the Head Deposits to be clay of 'medium to 'high' plasticity clay[10] and is therefore, this material is considered susceptible to volume change under the influence of trees or excavations.

7.4 Weathered London Clay Formation

The Weathered London Clay Formation was recovered in borehole BH1 and WS1. The stratum comprised firm to stiff orange brown occasionally mottled grey slightly silty clay with frequent fine selenite crystals.

SPTs were carried out in the stratum which recorded 'N' values of 17 to 18, corresponding to an undrained shear strength of 77kPa to 81kPa. Three undrained unconsolidated triaxial tests were carried out on undisturbed soil samples from the stratum. The recorded undrained shear strength, c_u ranged from 69kPa to 148kPa, which corresponds to a clay of 'medium high' to 'high' strength[11].

The geotechnical laboratory testing carried out on the disturbed soil sample produced the following classification parameters:

- Moisture content of 26 to 36%;
- Liquid Limit (LL) of 75 to 84%;
- Plasticity Limit (PL) of 30 to 36%;
- Plasticity Index of 45 to 48%;
- Percentage passing 425µm sieve is 100%.

These results indicate the Weathered London Clay Formation to be a clay of 'high' plasticity[10] and therefore susceptible to volume changes under the influence of trees or excavations.

7.5 London Clay Formation

The London Clay formation was encountered in borehole BH1 only. The stratum was found to comprise of firm to stiff slightly silty clay.

SPTs were carried out in the stratum with recorded 'N' values of 18 to 26, corresponding to an undrained shear strength of 81kPa to 117kPa. Two undrained unconsolidated triaxial tests were carried out on undisturbed soil samples from the stratum. The recorded undrained shear strength, c_u ranged from 106kPa to 137kPa, which corresponds to a clay of 'high' strength[11].

The geotechnical laboratory testing carried out on the disturbed soil sample produced the following classification parameters:

- Moisture content of 23%;
- Liquid Limit (LL) of 72 to 76%;
- Plasticity Limit (PL) of 27 to 30%;
- Plasticity Index of 45 to 46%;
- Percentage passing 425µm sieve is 83 to 100%.

These results indicate the London Clay Formation to be a clay of 'high' plasticity[10] and therefore susceptible volume change under the influence of trees or excavations.

7.6 Groundwater

No groundwater was encountered during the intrusive site investigation. However, during the subsequent monitoring visit undertaken on 21st October 2019 groundwater was encountered in both monitoring wells at 2.22mbgl (43.49mOD) in BH1 within the Head Deposits and 4.16mbgl in WS1 (41.88mOD) within the Weathered London Clay.

It was noted after the monitoring visit the contractor had added water to the bentonite seal around the monitoring pipe while installing the monitoring well in WS1. This was done to activate the bentonite seal. It is therefore considered that perched water is only present on the eastern side of the site and not across the entire site. This is supported by the significant difference in groundwater elevation between the two monitoring wells, despite the fact that both holes have the same response depths and the site being relatively level. Additionally, the borehole record for WS1 shows that the Weathered London Clay contains no granular material or fissures to enable significant permeability within the stratum.

It is not known if the groundwater encountered within BH1 is influenced by the King's Scholar sewer, however given that the sewer is some 27.5m to the east of the existing building with a significant layer of clay from the Head Deposits and London Clay Formation present, it is considered unlikely.

A copy of the monitoring record can be found in Appendix E.

7.7 Geotechnical Assessment Parameters

The geotechnical parameters for this report have been derived from the in-situ and laboratory testing carried out. The derived parameters are presented in Table 7 and a plot of undrained cohesion vs elevation is presented in Plate 6.

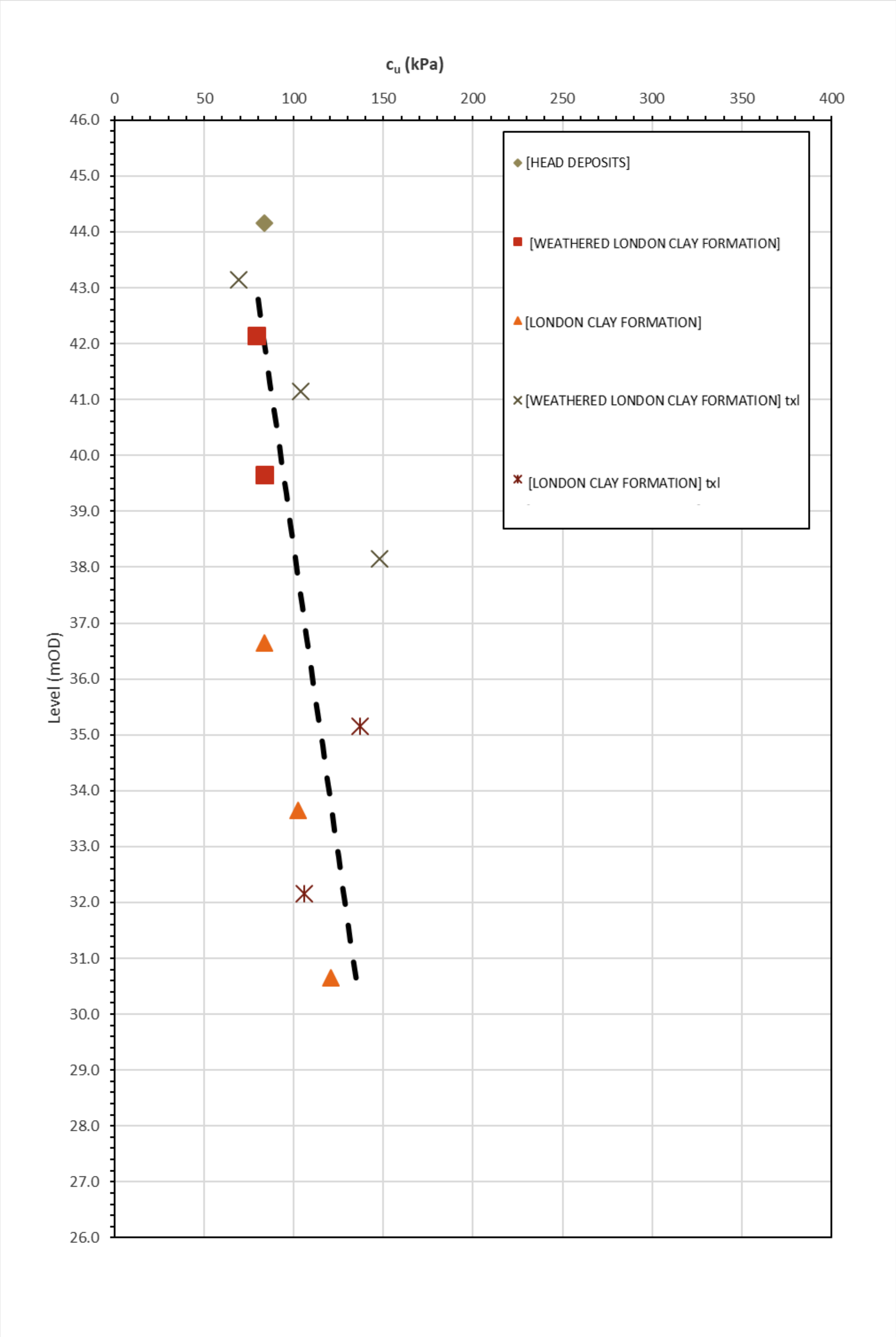
Table 7. Geotechnical parameters

Stratum	Depth to Top of Stratum (mbgl) [mOD]	Bulk Unit Weight, γ_b (kN/m^3)	Angle of Friction, φ (°)	Undrained Cohesion, c_u [c'] (kPa)	Young's Modulus, E_u [E'] (MPa)	
					Vertical	Lateral
Made Ground (granular)	0 [45.8]	18	30	-	13	13
Head Deposit	0.4 [45.4]	20	22^a	85 [0]	51^c [26]^d	85^e [63.75]^d
London Clay Formation	3.0 [42.8]	20	22^a	80 + 4.5z^b [5]	48 + 2.7z^{b,c} [36 + 2z^b]^d	80 + 4.5z^{b,e} [60 + 3.38z^b]^d

a. Based on BS 8002:2015 – Code of practice for earth retaining structures
b. z – depth below stratum level
c. Based on 600c_u – *Burland, Standing J.R., and Jardin F.M. (eds) (2001), building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.*
d. Based on 0.75E_u – *Burland, Standing J.R., and Jardin 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 1000c_u – *Burland, Standing J.R., and Jardin F.M. (eds) (2001), building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.*

Based on the results of the groundwater monitoring visit, as discussed in Section 7.6, a perched groundwater level of 43.49mOD has been assumed to be present along the eastern side of the site.

Plate 6. Undrained cohesion vs elevation



7.8 Concrete Aggressive Ground Classification

Soil samples from the strata encountered were sent for laboratory testing to determine the sulphate concentrations and pH in general accordance with the Building Research Establishment (BRE) SD1 guidance[13]. Table 8 presents a summary of the results.

Table 8. Design Sulphate (DS) classification for encountered soil strata

Stratum	Depth of Samples (m)	pH	Water Soluble Sulphate as SO_4 (2:1) mg/l	Total Sulphur (mg/kg)	Oxidisable Sulphides (OS % SO_4)	Design Sulphate (DS) Class [ACEC]
Head Deposit	2.75	7.9	95	120	0.003	DS-1 [AC-1]
Weathered London Clay Formation	5.0	7.7	3900	3400	0.16	DS-3 [AC-3s]

Pyritic soils are typically found in soils with an Oxidisable Sulphide (OS) percentage of greater than 0.3%. The results presented in Table 8 suggest that the Head Deposits and Weathered London Clay Formation are not pyritic.

[13] Building Research Establishment. (2005). *Special Digest 1 – Concrete in aggressive ground, third edition.*

8. CONSTRUCTION METHODOLOGY

8.1 Outline of Temporary and Permanent Works Proposals

Form SD have provided CGL with the proposed construction sequence. The construction sequence for the proposed development has been rationalised into four stages for the purpose of the GMA as follows:

Stage 1 - Demolition

- Demolish the existing building on the site.

Stage 2 – Installation of pile wall

- Install the contiguous pile wall around the upper (B1) level, followed by the lower (B2) basement level pile wall. Individual load bearing piles are installed after the pile walls are installed.

Stage 3 – Excavation of the basement

- Install ground floor capping beam.
- Install ground floor slab.
- Excavate to B1 level at 41.27mOD.
- Break down contiguous pile wall around B2 area to B1 level.
- Install capping beam to top of B2 contiguous pile wall at B1 level.
- Install temporary props across B2 capping beam level.
- Excavate to B2 level at 38.04mOD.

Stage 4 – Construction of below ground and above ground levels

- Install B2 level slab.
- Install B2 level structural walls and columns.
- Install B1 level slab.
- Remove temporary props across B2 capping beam level.

- Construct remaining above ground levels.

8.2 Applied Loads

The following sections outline the loads that will be applied during and after the construction of the proposed development. The discussed loads are present graphically in Figure 5.

8.2.1 Demolition Loads

As part of the proposed development the existing house of the site will be demolished, which will result in stress relief to the underlying ground.

The demolition load has been estimated by CGL assuming a typical two storey masonry clad timber frame residential structure. The estimated load is 10kPa.

8.2.2 Excavation Loads

The proposed development involves creating two basement levels as shown in Figure 4. The structural plans for the proposed development are provided in Appendix B. The top basement level (level -1) includes the same footprint of the above ground levels, and extends into the front and rear gardens. The bottom basement level (level -2) will cover much of the top basement level, however does not extend into the rear garden. The top level (level -1) will have a maximum excavation depth of 4.42m and the bottom level (level -2) will have a maximum excavation depth of 7.39m (this assumes a slab thickness of 450mm for the B1 slab and B2 slab). In addition, a localised lift pit will be constructed at bottom basement level, with a localised additional excavation depth of 1.5m.

The stress relief applied by the excavation has been calculated basement on the excavation depths of each stratum and the respective unit weights shown in Table 7. The excavation loads calculate for each level are presented in Table 9. The stratum thicknesses are based on those presented in Table 7.

Table 9. Excavation unloading pressures

	Level -1 Excavation		*Level -2 Excavation*		*Lift Pit*	
Stratum	***Thickness of Stratum Excavated (m)***	***Unloading Pressure from Stratum Excavation (kPa)***	***Thickness of Stratum Excavated (m)***	***Unloading Pressure from Stratum Excavation (kPa)***	***Thickness of Stratum Excavated (m)***	***Unloading Pressure from Stratum Excavation (kPa)***
Made Ground	0.4	7.2	0.4	7.2	0.4	7.2
Head Deposit	2.6	52	2.6	52	2.6	52
London Clay Formation	1.42	28.4	4.39	87.8	5.89	117.8
Total	4.42	87.6	7.39	147	8.89	177

8.2.3 Construction Loads

The loads from the proposed new building on site have been provided by Form SD and are presented in Appendix G. The line loads applied around the edge of the basement levels have been used to determine the length of the piles as described in Section 8.3 and the resulting end and shaft bearing loads as described in Section 8.4. It is understood that the structure will be supported on piles. Pile loads provided by Form SD (Appendix G) have been used to determine the length of the piles as described in Section 8.5 and the resulting end and shaft bearing loads are described in 8.6.

The construction loads as assumed in the CGL Ground Movement Assessment (GMA) are presented in Figure 5.

8.3 Preliminary Pile Wall Design

A contiguous pile wall has been proposed to retain the excavation. While groundwater has been encountered during the monitoring visit, this is likely to be associated with perched water contained within the Head Deposits on site. Given that the Head Deposit stratum is predominantly formed from slightly gravelly slightly silty clay, it is expected that the stratum will have a 'very low' permeability (1 x 10^{-6} to 1 x 10^{-9})[14]. Therefore, it is expected that a contiguous pile wall may be appropriate using sump pumps to control seepage into the basement excavation. It is understood that a contiguous piled wall is under construction for the basement development at 73 to 75 Avenue Road.

At this stage only a preliminary retaining wall design has been undertaken for the purpose of this report to estimate the length of the piles. The preliminary pile design has been based on the line loads applied to the contiguous pile wall as shown in Appendix G or approximately two times the excavation depth assuming high level propping will be provided, whichever is greater. Based on the proposed development plans (Appendix B), 350mm diameter piles at 500mm c/c spacing are proposed around the majority of the proposed retaining wall, with exception to the northern wall adjacent to the pool, which is proposed to be 450mm diameter piles at 600mm c/c spacing. The preliminary pile design has been based on this. An alternative design can used so long as it the resulting ground movements do not increase the Building Damage Category.

The design assumes the following:

- The pile retaining wall along the western, southern and eastern elevation will be constructed as a contiguous pile wall with 350mm diameter piles at 500mm c/c spacing. Along the northern

[14] CIRIA. (2016). *CIRIA C750 – Groundwater control: design and practice, second edition.*

elevation the contiguous pile wall will be constructed with 450mm diameter piles at 600mm c/c spacing;

- All piles will be cast in-situ, Continuous Flight Auger (CFA) or bored;
- Only the length of the piles below the excavation level of the proposed basement levels will contribute to the axial capacity of the pile wall;
- Top of pile wall level has been taken as being ground level (45.8mOD);
- The design has been calculated based on the Eurocode 7 Design Approach 1, Combination 1 and Combination 2 assuming no working or preliminary pile load testing has been scheduled or undertaken:
  - Combination 1 applies partial factors to the dead and live loads of 1.35 and 1.5 respectively, while the geotechnical parameters have a partial factor of 1.0 applied;
  - Combination 2 applies partial factors to the dead and live loads of 1.0 and 1.3, with geotechnical partial factors of 1.6 for the skin friction, 2.0 for the base capacity and 1.4 for the model factor (model factor value is based on the case of no working or preliminary pile load tests);
- The capacity calculation assumes an end bearing capacity factor (N_c) of 7.5 (reduced to allow for interaction effects on the pile wall), an adhesion value of 0.5 and a limiting skin friction of 110kPa.

A summary of the wall length design is presented in Table 11. The wall sections are shown in Figure 5.

Table 10. Contiguous pile wall initial design summary

Wall Section	*Max Total Unfactored Load (kN/m)*	*Max Combination 1 Load (kN/pile) [Combination 2]*	*Minimum Pile Length from ground level for axial load, Combination 1 [Combination 2] (m)*	*x2 Max Excavation Depth (m)*	*Design Length assumed in analysis (m)*	*Safe Working Load (kN/pile)*	*Base Resistance (%)*
P1	107.5	74 [56]	5.49 [6.13]	9.5	9.5	143	20
P2 (pool section)	205	204 [156]	9.77 [10.85]	15.4	15	328	20
P3	247.5	170 [130]	10.37 [11.41]	15.4	15	245	20
P4	150	104 [80]	8.91 [9.69]	15.4	15	245	20
P5	200	137 [105]	9.65 [10.57]	15.4	15	245	20
P6	192.5	132 [100]	6.97 [7.90]	9.5	9.5	143	20

Wall Section	Max Total Unfactored Load (kN/m)	Max Combination 1 Load (kN/pile) [Combination 2]	Minimum Pile Length from ground level for axial load, Combination 1 [Combination 2] (m)	x2 Max Excavation Depth (m)	Design Length assumed in analysis (m)	Safe Working Load (kN/pile)	Base Resistance (%)
P7	175	120 [91]	6.67 [7.54]	9.5	9.5	143	20
P8	240	165 [125]	7.78 [8.82]	9.5	9.5	143	20

8.4 Pile Wall Load Pressures

The pile walls will transfer the loads to the ground through the shaft (shaft friction) and toe (end bearing) of the piles. The loads transferred via the pile shaft and toe have been calculated based on the preliminary designs in the previous section, with the shaft friction loads modelled as acting uniformly at a level two-thirds down the pile (below the excavation level), assuming a 1:4 load spread down the pile, and the end bearing modelled at the toe of the pile with the load applied uniformly across area of the toe.

A summary of the skin friction and end bearing pressures is shown in Table 11.

Table 11. Pile wall end bearing and shaft pressures summary

Wall Section	Max Total Unfactored Load (kN/m)	Max End Bearing Load (kN/m)	Max Pile Shaft Load (kN/m)	Level at which Shaft Friction is Applied (mOD)	End Bearing Pressure (kPa)	Equivalent Pile Shaft Pressure (kPa)
P1	107.5	21.5	86	37.24	61	38
P2	205	49.5	198	33.22	110	70
P3	247.5	49.5	198	33.32	141	73
P4	150	30	120	33.32	86	44
P5	200	40	160	33.32	114	59
P6	192.5	38.5	154	37.24	110	68
P7	175	35	140	37.24	100	62
P8	240	48	192	37.24	137	70

8.5 Preliminary Pile Design

At this stage only a preliminary pile design has been undertaken for the purpose of this report to estimate the length of the piles. The preliminary pile design has been based on the loads applied to the piles as shown in Appendix G. Pile designs for 350mm 450mm and 550mm piles have been provided in and Plate 8 for B1 and B2 levels respectively, however the results of the 350mm piles have been used for this report.

The design assumes the following:

- 350mm diameter piles will be used.
- All piles will be cast in-situ, Continuous Flight Auger (CFA) or bored;

- Top of pile level has been taken as 41.06mOD for the piles at B1 level, and 38.49mOD for the piles at B2 level;

- The design has been calculated based on the Eurocode 7 Design Approach 1 Combination 2 assuming no working or preliminary pile load testing has been scheduled or undertaken;

  - Combination 2 applies partial factors to the dead and live loads of 1.0 and 1.3, with geotechnical partial factors of 1.6 for the skin friction, 2.0 for the base capacity and 1.4 for the model factor (model factor value is based on the case of no working or preliminary pile load tests);

- The capacity calculation assumes an end bearing capacity factor (N_c) of 9, an adhesion value of 0.5 and a limiting skin friction of 110kPa.

Plate 7: Preliminary Pile Safe Working Load Graph B1 Level

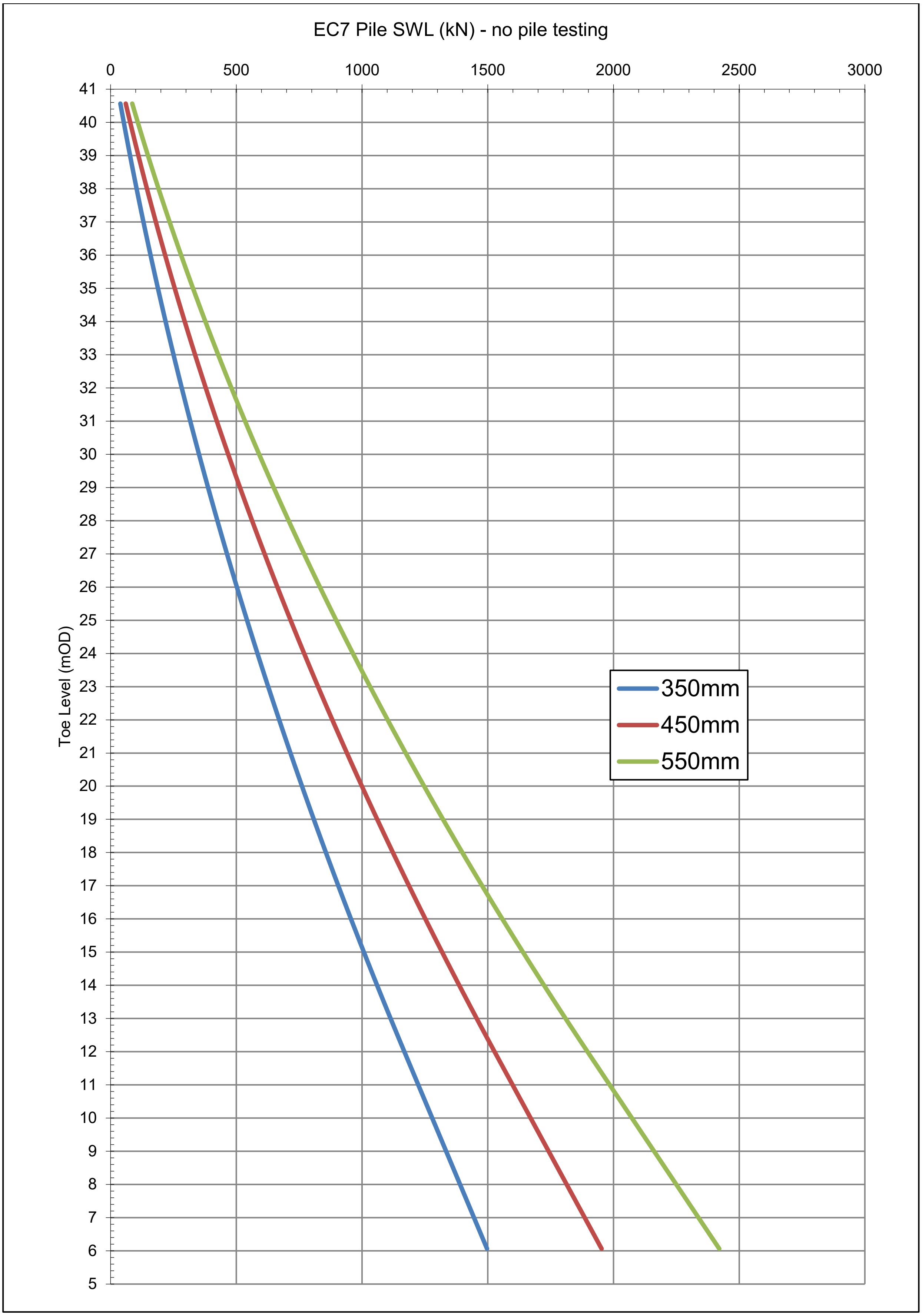
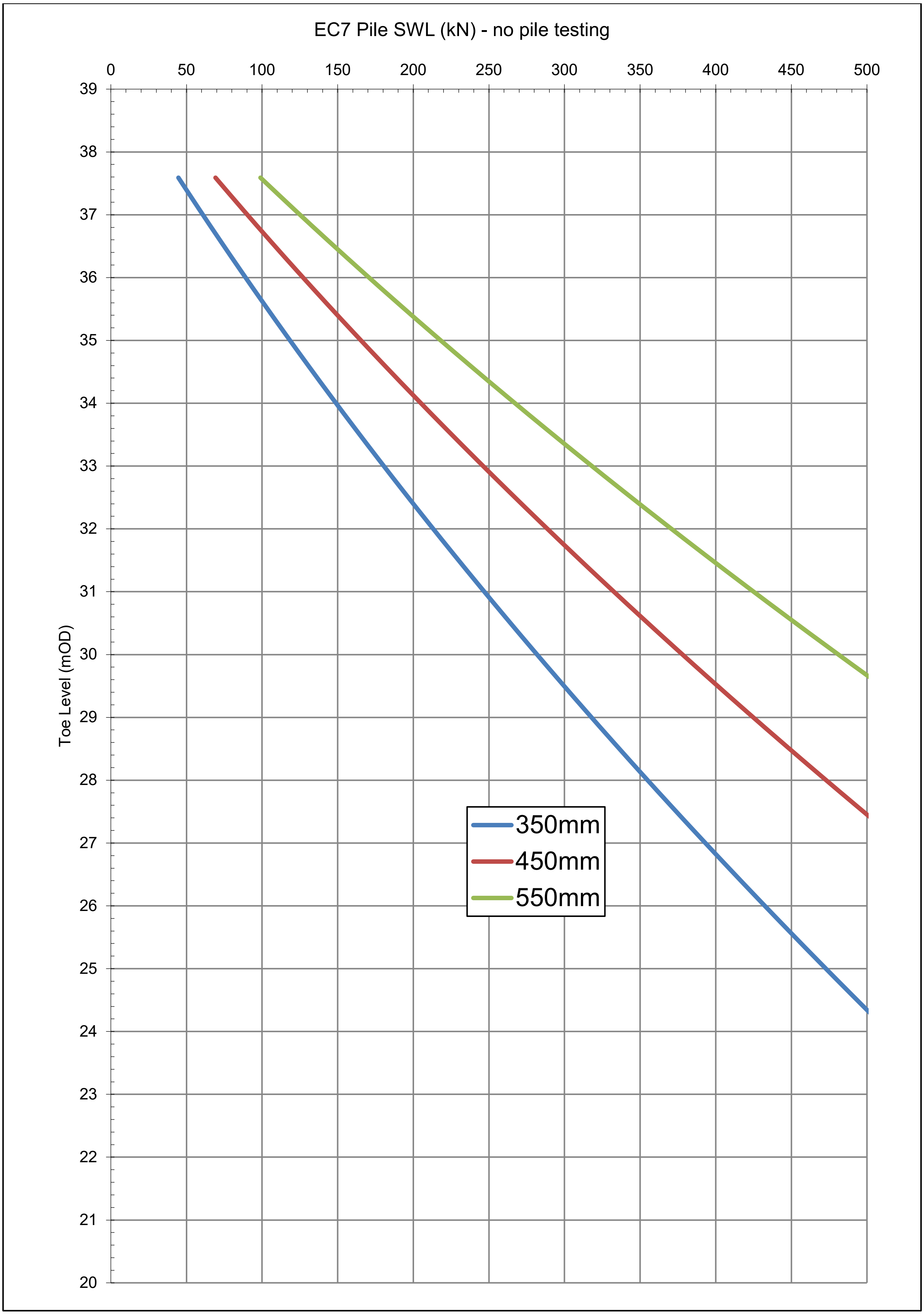


Plate 8: Preliminary Pile Safe Working Load Graph B2 Level.


EC7 Pile SWL (kN) - no pile testing
0
50
100
150
200
250
300
350
400
450
500
Toe Level (mOD)
39
38
37
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
21
20
350mm
450mm
550mm

8.6 Pile Pressures

It is assumed, for the PDisp modelling, that proposed piles on the same level will act as a pile group. The piles will transfer the loads to the ground through the shaft (shaft friction) and toe (end bearing) of the piles. The loads transferred via the pile shaft and toe have been calculated based on the preliminary designs in the previous section, with the shaft friction loads modelled as acting uniformly at a level two-thirds down the pile group (below the B2 slab level), assuming a 1:4 load spread down the pile group, and the end bearing modelled at the toe of the pile group with the load applied uniformly across area of the pile group toe.

A summary of the skin friction and end bearing pressures is shown in Table 11.

Table 12. Pile group end bearing and shaft pressures summary

Level	Total Unfactored Load (kN)	End Bearing Load (kN)	Shaft Load (kN)	Level at which Shaft Friction is Applied (mOD)	End Bearing Pressure (kPa)	Equivalent Pile Shaft Pressure (kPa)
B1	724.2	72.4	651.8	26.40	28.1	7.8
B2	618.5	61.9	556.7	26.42	3	15.5

9. GROUND MOVEMENT AND BUILDING DAMAGE ASSESSMENT

9.1 Ground Movement Assessment

A Ground Movement Assessment (GMA) has been carried out based on guidance from CIRIA C760[15] which is described by Burland, Standing, J.R. and Jardine F.M. (2001). The GMA considers ground conditions, construction methodology and existing structures/infrastructure present on and close to the site.

Possible ground movement mechanisms are outlined below:

- Installation settlement of the retaining wall. The installation of the pile wall can result in vertical and lateral ground movements behind the wall. These movements can be minimised with good construction control to avoid over-flighting.
- Deflection of the retaining wall: Piled retaining walls are relatively slender and are therefore prone to deflection under applied earth pressures if they are not rigidly propped, deflection can lead to ground movements behind the wall;
- Heave movement: The London Clay is susceptible to short-term and time dependant swelling after a change in overburden pressure, which will occur due to the demolition of the existing building on the site, the excavation of the basement, additional loads from the proposed building and installation movements from the retaining walls.

It is proposed to underpin the northern and southern garden walls to enable the construction of the pile capping beam. It is recommended that the underpins are constructed using the hit-and-miss construction methodology. This involves installing the underpins in 1m wide sections in five stages. After each section is formed, the subsequent section is formed three sections away as shown on drawing L(23)01 in Appendix B. By following this method, the lateral expansion of the London Clay Formation at each successive underpin section is very localised and therefore unlikely to impact on the strip footing of the adjacent property. Therefore, the ground movements and impact on the adjacent properties are expected to be negligible and have therefore not been considered in this assessment.

A number of key structures are within the zone of influence and have been assessed to determine the impact on them. These include the neighbouring property at 81 Avenue Road and the neighbouring property at 77 Avenue Road. Critical section lines have been placed through these properties to identify the critical ground movements that are predicted to occur as a result of the proposed

[15] CIRIA. (2017). *C760 – Guidance on embedded retaining wall design*.

development. The exact location of these critical section lines perpendicular to proposed basement, have been determined based on the worst case for vertical ground movements produced by the PDISP models. The distance between the pile retaining wall and each neighbouring property has been based on the minimum distances, as shown in Figure 2.

The following assumptions have been made within the GMA:

- The foundations for the neighbouring properties are at 1mbgl (44.8mOD);
- While the neighbouring properties have planning permission granted for redevelopment, which includes basement levels, it will be assumed that the proposed development will be constructed first. This is considered to be more conservative for the GMA.

The ground movements have been determined using a combination of empirical and numerical analysis methods. Ground movements relating to the pile wall have been determined using the guidance for pile installation and deflection due to excavation, as outlined CIRIA guidance C760. Ground movements relating to the demolition of the existing building, excavation of the basement and construction of the proposed development have been calculated using the Oasys PDISP (Pressure Induced Displacement) software. The software calculates the ground movements caused by vertical pressures, in an elastic half-space. It can use both linear elastic and non-linear soil conditions. In this analysis only elastic conditions were considered.

9.2 Stage 1 - Demolition

This stage models the ground movements which are generated from the demolition of the existing building on the site. It has been assumed that the existing building is founded at the ground surface at a level of 45.8mOD. It has been assumed that only vertical ground movements (heave) will occur.

The demolition loads specified in Section 8.2.1 have been applied across the footprint of the site. This will result in heave movements occurring. The resulting vertical ground movements at the formation level of the neighbouring properties have been calculated in PDISP and are shown in Figure 6 and Table 13.

Table 13. Maximum ground movements along critical section lines at Stage 1

	Critical Section Line 1 – 81 Avenue Road	*Critical Section Line 2 – 77 Avenue Road*
Maximum vertical ground movements across neighbouring building (mm)	-0.4	-0.2
Maximum horizontal ground movements across neighbouring building (mm)	0	0
Foundation formation level (mOD)	44.69	44.69

Note: Negative vertical movements indicate heave, positive vertical values indicate settlement. Negative horizontal movements indicate movements away from the excavation/basement, positive horizontal movements indicate movements towards excavation/basement.

9.3 Stage 2 – Installation of pile wall

Stage 2 involves the installation of the pile walls and individual load bearing piles, which will result in vertical and lateral ground movements that could affect the surrounding structures. Only the impact from the pile wall around the larger B1 basement level has been included, as the piles around the B2 level and the individual load bearing piles are expected to be installed after the B1 level pile wall has been installed.

Potential lateral and vertical ground movements have been calculated in accordance with CIRIA C760. The guidance states that for contiguous pile walls the resulting lateral and vertical ground movements are 0.04% of the total wall depth at the pile wall. The lateral ground movements reduce with distance from the pile wall to negligible at a distance of 1.5 times the total wall depth, while the vertical ground movements reduce to negligible at a distance of 2 times the total wall depth. Ball, Langdon and Creighton (2014[16]) showed that these movements could be halved if a good standard of workmanship was adopted and that the piles are installed in a hit-and-miss sequence. These recommendations have been used to reduce the lateral and vertical ground movements to 0.02%.

The resulting maximum installation ground movements are presented in Table 14.

Table 14. Maximum pile wall installation movements

Pile Wall	*Wall Depth (m)*	*Horizontal Movements*		*Vertical Movements*	
		Surface Movement at Wall (mm)	*Distance Behind Wall to Negligible Movement (m)*	*Surface Movement at Wall (mm)*	*Distance Behind Wall to Negligible Movement (m)*
North elevation	15	3	22.5	3	30
East elevation	15	3	22.5	3	30
South elevation	15	3	22.5	3	30
West elevation	9.5	1.9	14.25	1.9	19

Note: Negative vertical movements indicate heave, positive vertical values indicate settlement. Negative horizontal movements indicate movements away from the excavation/basement, positive horizontal movements indicate movements towards excavation/basement.

The resulting ground movements from this stage and Stage 1 have been combined and are presented in Table 15.

Table 15. Maximum ground movements along critical section lines at Stage 2

	Critical Section Line 1 – 81 Avenue Road	*Critical Section Line 2 – 77 Avenue Road*
Maximum vertical ground movements across neighbouring building (mm)	2.1	2.1
Maximum horizontal ground movements across neighbouring building (mm)	3	3
Foundation formation level (mOD)	44.7	44.7

Note: Negative vertical movements indicate heave, positive vertical values indicate settlement. Negative horizontal movements indicate movements away from the excavation/basement, positive horizontal movements indicate movements towards excavation/basement.

[16] Ball, R., Langdon, N., and Creighton, M. (2014). *Prediction of party wall movements using CIRIA C580.* September 2014.

9.4 Stage 3 – Excavation of the basement

The excavation to form the basement will result in lateral and vertical ground movements from the deflections of the pile wall, as well as heave movements from the excavation. The ground movements resulting from the deflection of the pile wall are expected to be limited as the basement will be constructed using 'top down' construction. It is expected that the contiguous pile wall will contain the elastic heave movement which will occur in the short-term, therefore heave movements have been limited to the extent of the basement excavation. It is also expected that only the ground movements from the pile wall around the B1 level will impact on the neighbouring properties, as they will contain the B2 level pile wall deflection movements. The ground movements have been modelled using a combination of the results from the soil-structure interaction software WALLAP (used to model the movements from the deflection of the pile wall), guidance from CIRIA C760[15] and the results from PDISP (used to model the heave movements within the excavation).

The WALLAP model has followed the Stage 3 construction sequence detailed in Section 8.1, with an additional stage included to model the deflection of the wall in the long-term post-consolidation. A surcharge has been applied to account for the loading by boundary wall and neighbouring buildings. The output from the WALLAP models is presented as Appendix H.

WALLAP cannot determine the vertical settlement profile behind the pile wall, therefore the semi-empirical method detailed in Section 6.2.2 of CIRIA C760[15] has been used. The method suggests that the vertical ground movement profile is equal to half the lateral ground movement profile along 1.5 times the height of the pile wall. Lateral ground movements behind the pile wall on the retained side have been calculated by assuming a parabolic reduction in lateral ground movements away from the pile wall.

The pressures modelled in PDISP have used the soil pressures in Section 8.2.1 and Section 8.2.2. The PDISP model results are shown in Figure 7.

The combined WALLAP and PDISP results across the two critical sections are presented in Table 16 below.

Table 16. Maximum ground movements along critical section lines at Stage 3

	Critical Section Line 1 – 81 Avenue Road	***Critical Section Line 2 – 77 Avenue Road***
Maximum vertical ground movements across neighbouring building (mm)	9.4	4.6
Maximum horizontal ground movements across neighbouring building (mm)	8.0	5.7
Foundation formation level (mOD)	44.7	44.7

Note: Negative vertical movements indicate heave, positive vertical values indicate settlement. Negative horizontal movements indicate movements away from the excavation/basement, positive horizontal movements indicate movements towards excavation/basement.

9.5 Stage 4 – Construction of below ground and above ground levels

This stage models the total ground movements in the long-term once the rest of the building has been constructed and the cohesive ground has been allowed to fully drain and consolidate. This has modelled by applying the structural loads presented in Figure 5 in the PDISP model, as well as using the drained soil parameters (φ', E') in Table 7. While it has been assumed that the short-term elastic heave movements will not extend outside the basement excavation, due to the confining effect of the contiguous pile wall, it has been assumed that the long-term heave will occur outside the basement excavation. The ground movements resulting from the pile wall deflection remain unchanged as long term movements have been included in Stage 3.

The ground movements resulting from the PDISP model are presented in Figure 8. The ground movements within the basement include both the short-term (elastic) and the long-term (plastic) heave movements. The ground movements outside the basement excavation include the short-term and long-term ground movements from the demolition of the existing building, together with the long-term ground movements caused by the excavation of the basement.

The combined results from the PDISP model in Figure 8 and the WALLAP model along the two critical section lines are presented in Table 17.

Table 17. Maximum ground movements along critical section lines at Stage 4

	Critical Section Line 1 – 81 Avenue Road	*Critical Section Line 2 – 77 Avenue Road*
Maximum vertical ground movements across neighbouring building (mm)	6.0	-0.83
Maximum horizontal ground movements across neighbouring building (mm)	8.0	5.7
Foundation formation level (mOD)	44.7	44.7

Note: Negative vertical movements indicate heave, positive vertical values indicate settlement. Negative horizontal movements indicate movements away from the excavation/basement, positive horizontal movements indicate movements towards excavation/basement.

9.6 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 Wroth[17] and later supplemented by the work of Boscardin and Cording[18] has been used, as described in CIRIA Special Publication 200[19] and CIRIA C760[15]. General categories are summarised below in Table 18.

[17] 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.

[18] Boscardin, Standing J.R., and Cording, E.G. (1989). Building response to excavation induced settlement. J Geotech Eng ASCE, 115(1), pp 1-21.

[19] 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.

Table 18. Classification of damage visible to walls (reproduction of Table 6.4, CIRIA C760)

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 and horizontal displacement lines across the two critical sections have been plotted and the results are shown in Plate 9 to Plate 12. The length of the critical section lines and the distance to boundaries of the neighbouring properties have been discussed in Section 9.1.These plots include the displacement due to installation of the piles and the deflection of the retaining wall. The results indicate the maximum angular distortions across 81 Avenue Road and 77 Avenue Road are 1/1750 and 1/3460 respectively. These are conservative values as the calculations do not include stiffness of the foundation slab of the properties and assume fully flexible loaded zones. The angular distortions across the width of the neighbouring properties are within the limits identified by Skempton and MacDonald[20] 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.

[20] 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.

Plate 9. Critical section line 1 (81 Avenue Road) - Vertical displacement profile

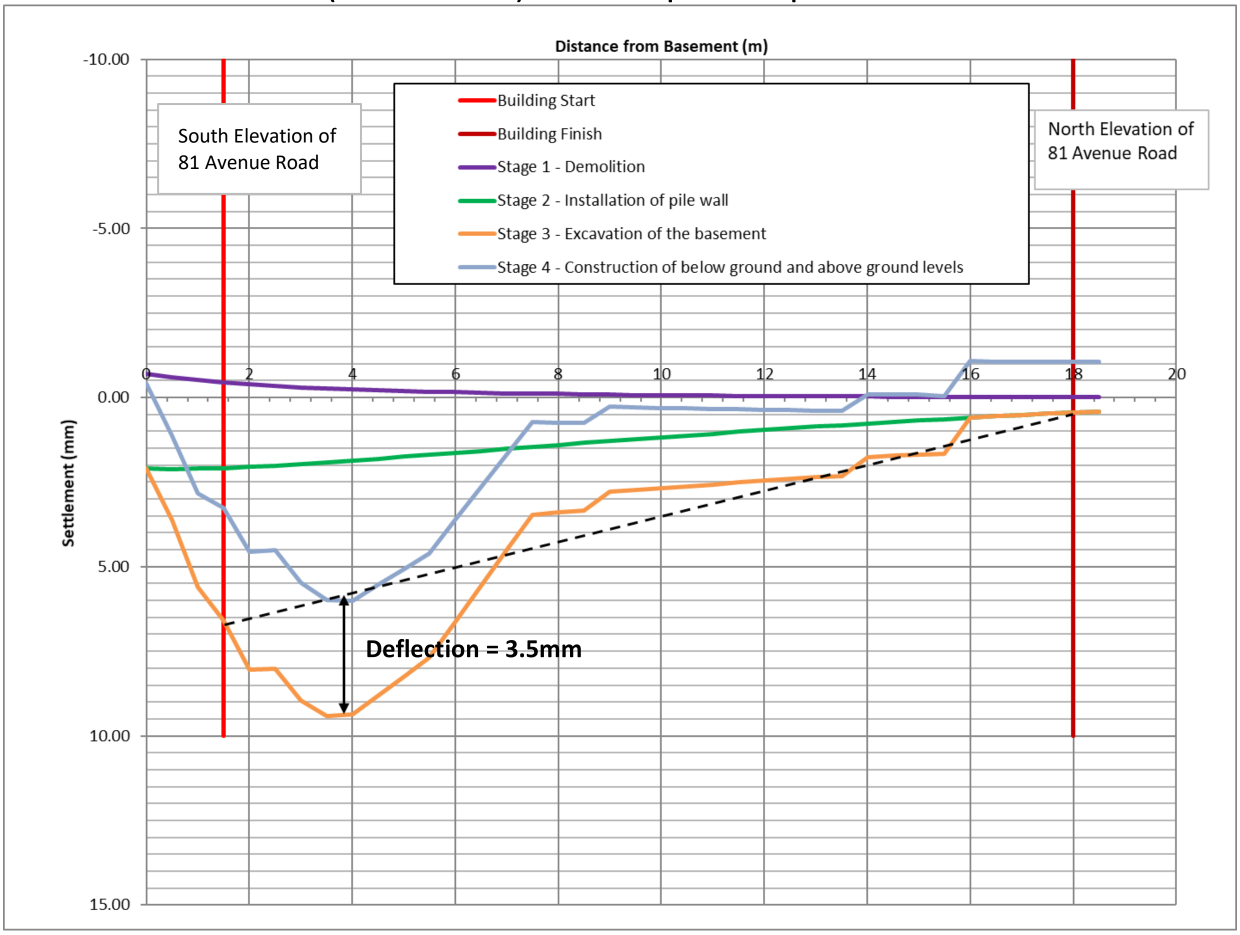


Plate 10. Critical section line 1 (81 Avenue Road) – Horizontal displacement profile

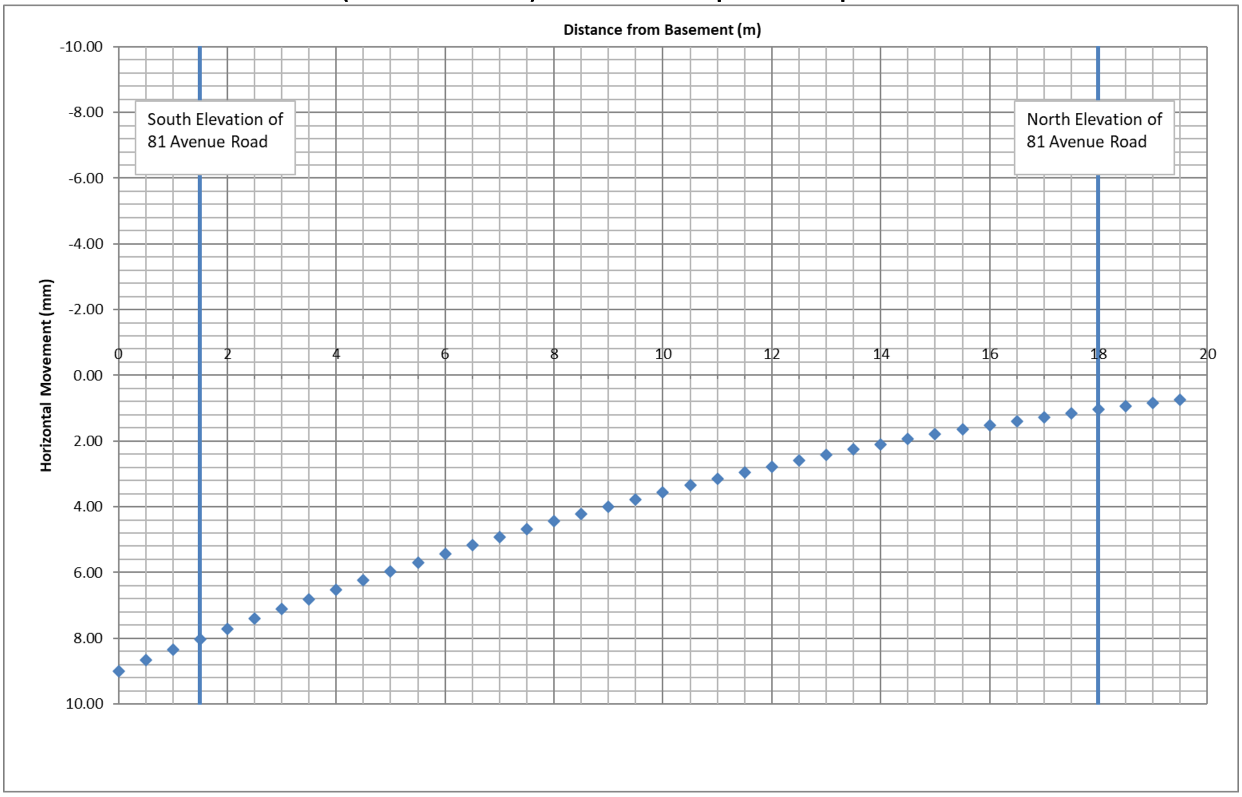


Plate 11. Critical section line 2 (77 Avenue Road) - Vertical displacement profile

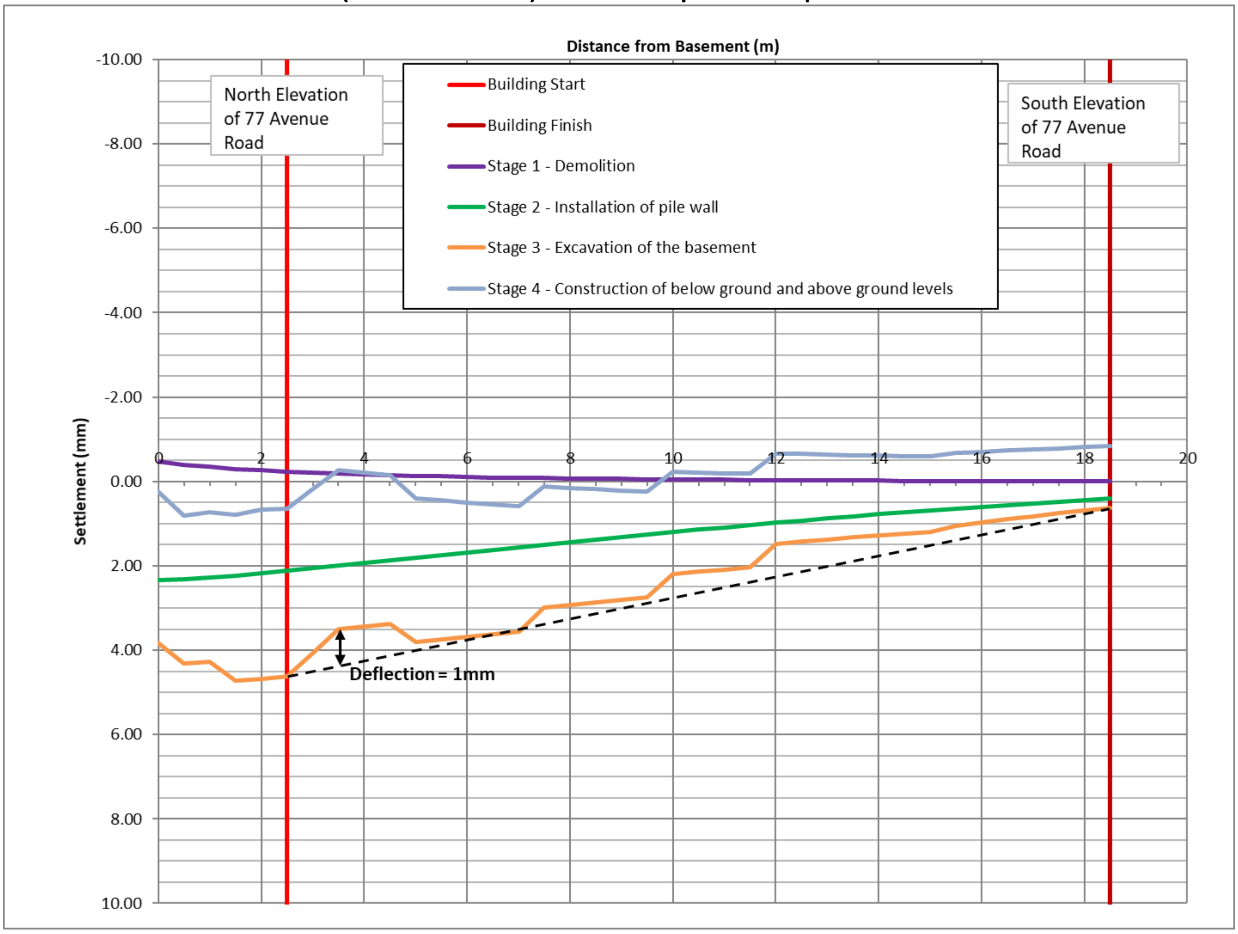
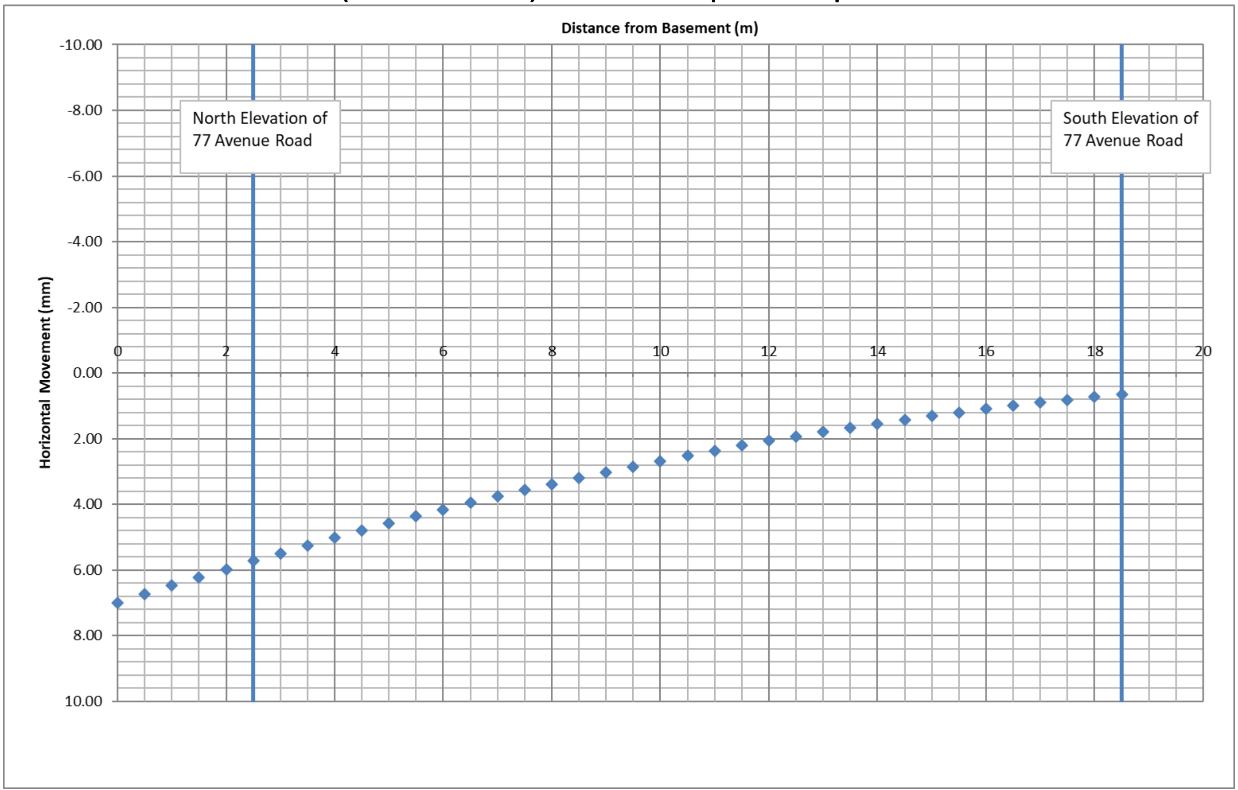


Plate 12. Critical section line 2 (77 Avenue Road) - Horizontal displacement profile



The Damage Category for each of the neighbouring properties has been determined by plotting the horizontal strain and deflection ratio values as summarised in Table 19 and presented graphically in Plate 13 and Plate 14. The damage category limits have been based on the slenderness (length/height) of each structure and the assumed structural material used to support each structure (timber-masonry)

The results show that the anticipated damage category for both neighbouring structures is Category 1 'very slight' damage including fine cracks that are easily treated of <1mm. This is based on a good standard of workmanship, adopting a hit and miss construction sequence when installing the pile wall, constructing the basement using 'top down' construction and providing temporary propping during construction along the 450mm diameter pile retaining wall section at B1 slab level. Regular monitoring of the retaining wall should be undertaken during construction to confirm these values are not exceeded and to manage risk.

Table 19. Summary of ground movements and corresponding Damage Category

Critical Section	*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*
Line 1	Stage 4	7.0	3.5	0.042	0.021	Category 1
Line 2	Stage 4	5.1	1	0.032	0.006	Category 0

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 13. Building Interaction Chart - Critical section line 1 (81 Avenue Road)

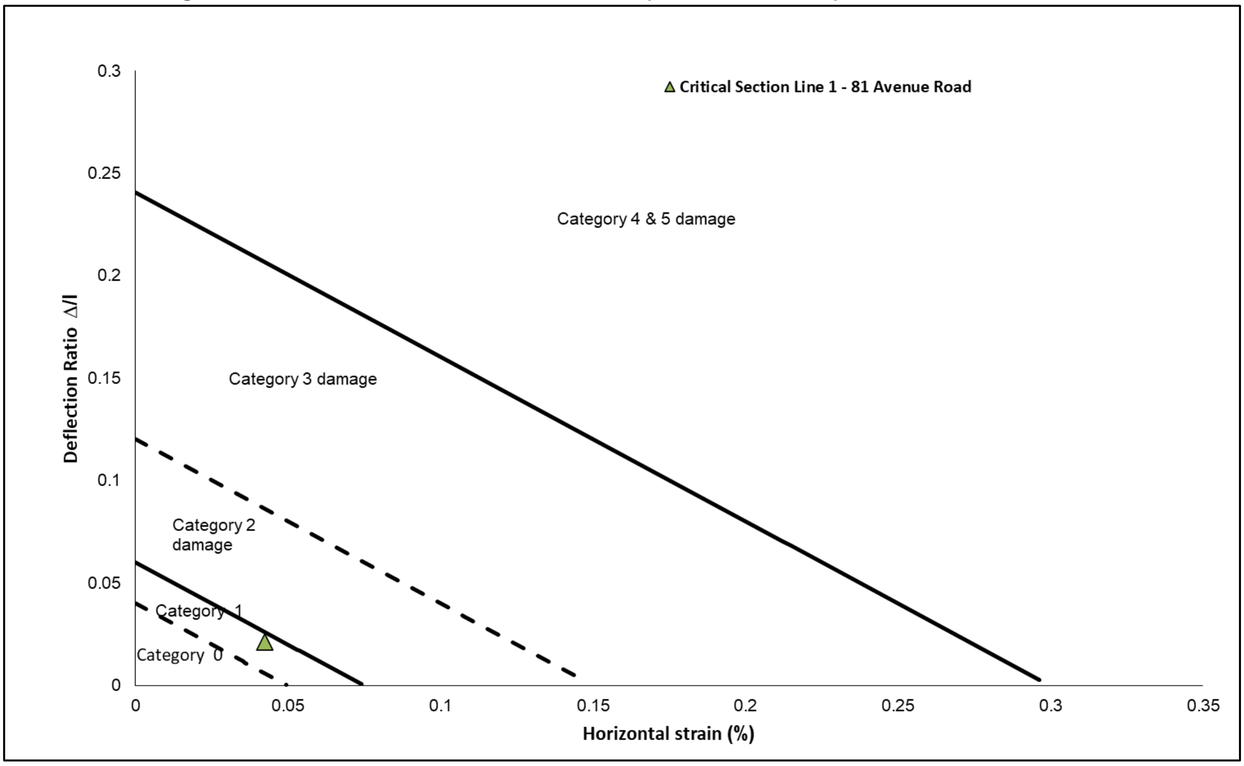
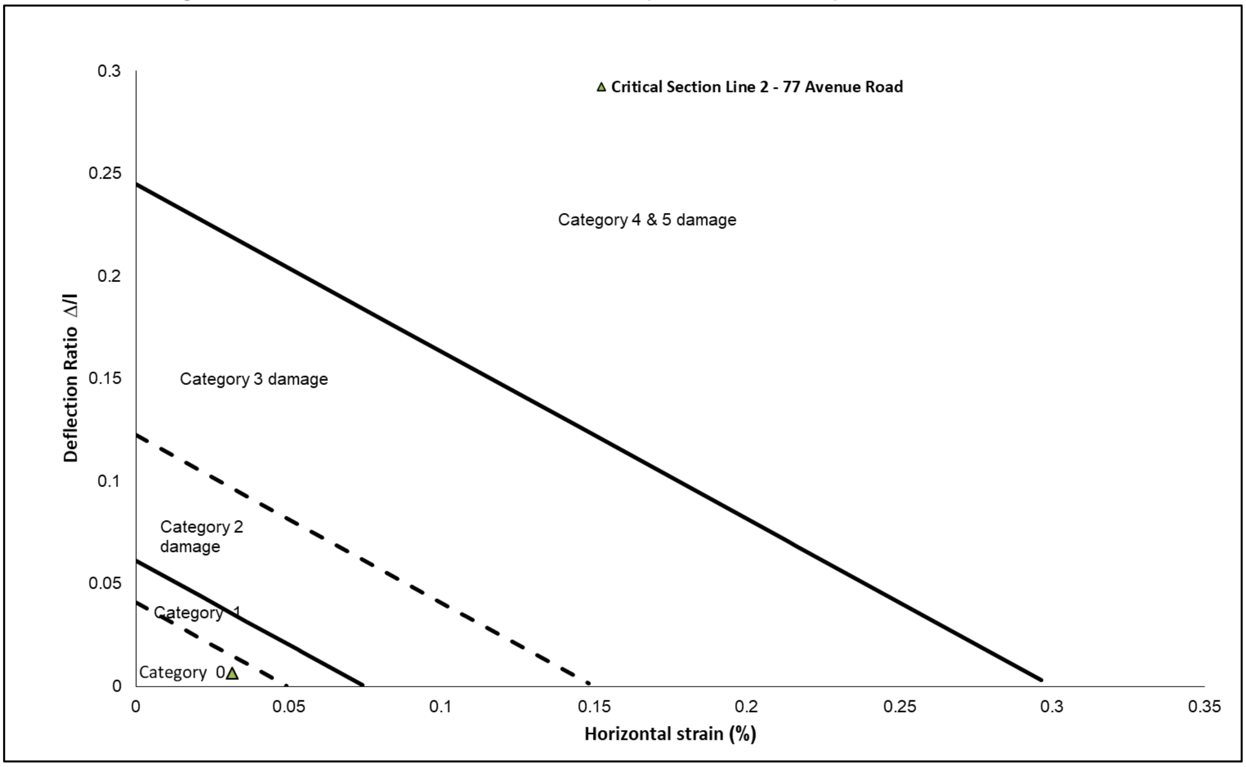


Plate 14. Building Interaction Chart - Critical section line 2 (77 Avenue Road)



9.7 Control of Construction Works (Monitoring Strategy)

The results of the ground movement analysis suggest that with good construction, maximum damage to adjacent structures generated by the assumed construction methods and sequence is likely to be within Category 1 'very slight' damage. The predicted damage category is dependant on adopting a good standard of workmanship, adopting a hit-and-miss construction sequence when installing the contiguous pile walls using the method described by Ball, Langdon and Creighton (2014[16]), constructing the basement using the top down construction method and propping the 450mm diameter pile wall section at B1 slab level.

A formal monitoring strategy should be implemented across the site to observe and control 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 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 movement, vertical movement and tilting has occurred.

[21] Nickolson, D., Tse, Che-Ming., Penny, C. *The Observational Method in ground engineering: principals and applications,* CIRIA report R185, 1999.

Mitigation strategies should be prepared prior to construction and implemented if unacceptable movements occur. Mitigation strategies could include revising the pile installation sequence, installing additional temporary props and installing temporary/permanent casing.

Monitoring data should be checked against predefined trigger limits and review regularly to assess and manage the damage category of the adjacent buildings as construction progresses. The data could also potentially be used to undertake back analysis calculations and value-engineer certain elements of the construction, such as prop design.

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 or structural defects should be carefully recorded, documented and regularly inspected as construction progresses.

10. BUILDING IMPACT ASSESSMENT – NON-TECHNICAL SUMMARY

10.1 Land Stability

The site investigation has identified that the natural soil below the site comprises of Head Deposit and London Clay Formation. No groundwater was encountered during the site investigation; however, groundwater was encountered during the subsequent monitoring visit in the Head Deposit stratum. Given that no groundwater was encountered while drilling the boreholes and the predominately cohesive ground conditions which typically have a very low permeability, it is expected that only isolated perched water will be encountered during construction which can be managed using sump pumps.

The site does have a gentle slope, which dips towards the north-east with an angle less than 7°. The proposed development will be founded on a combination of a pile wall and raft slab. The toe of the piles will be below the formation level of the neighbouring properties.

The nearest pedestrian footpath is some 14.9m from the existing building and the nearest sewer (King's Scholars' Pond Sewer) is some 27.5m from the existing building. The proposed basement will be some 10.1m from the footpath and some 20.9m from the sewer. The ground movements at both the footpath and sewer are expected to be negligible.

The building damage category for the neighbouring properties at 81 and 77 Avenue Road can be controlled to within Category 1 'Very Slight' damage. This assumes a good standard of workmanship, installing the piles in a hit-and-miss construction sequence, constructing the basement using the 'top down' construction method and providing temporary propping along the 450mm diameter pile wall section at B1 slab level.

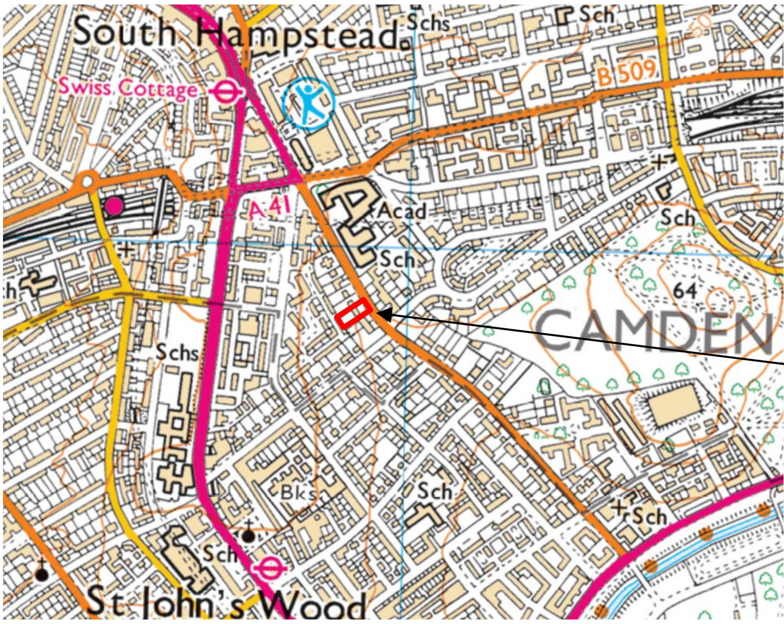
10.2 Hydrogeology and Groundwater Flooding

The BIA has concluded that there is a 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 final design to mitigate against possible groundwater intrusion into the proposed basement.

10.3 Hydrology, Surface Water Flooding and Sewer Flooding

The BIA has concluded there is a very 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].

FIGURES



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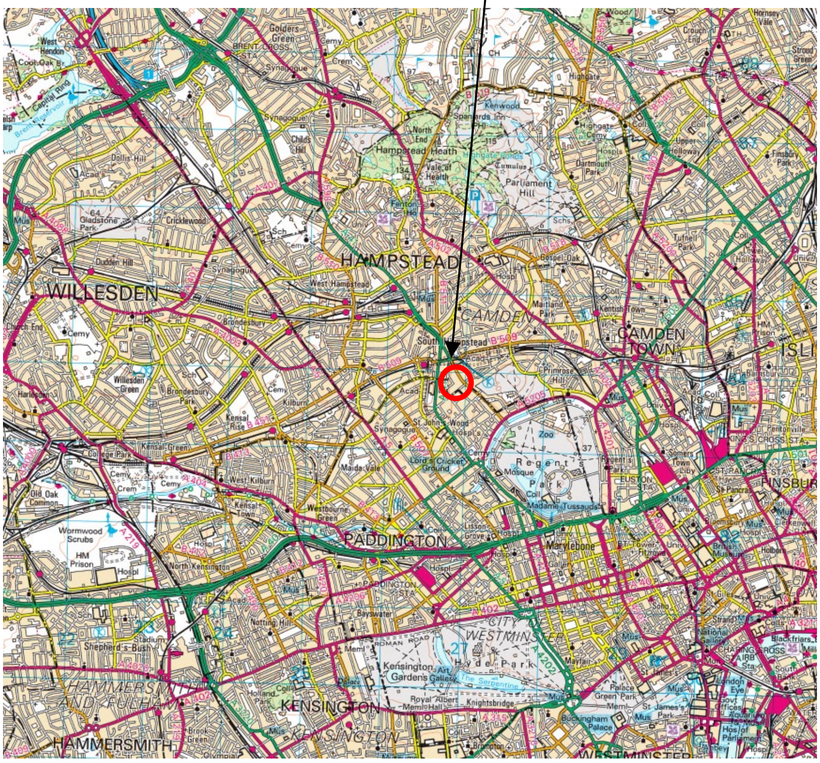
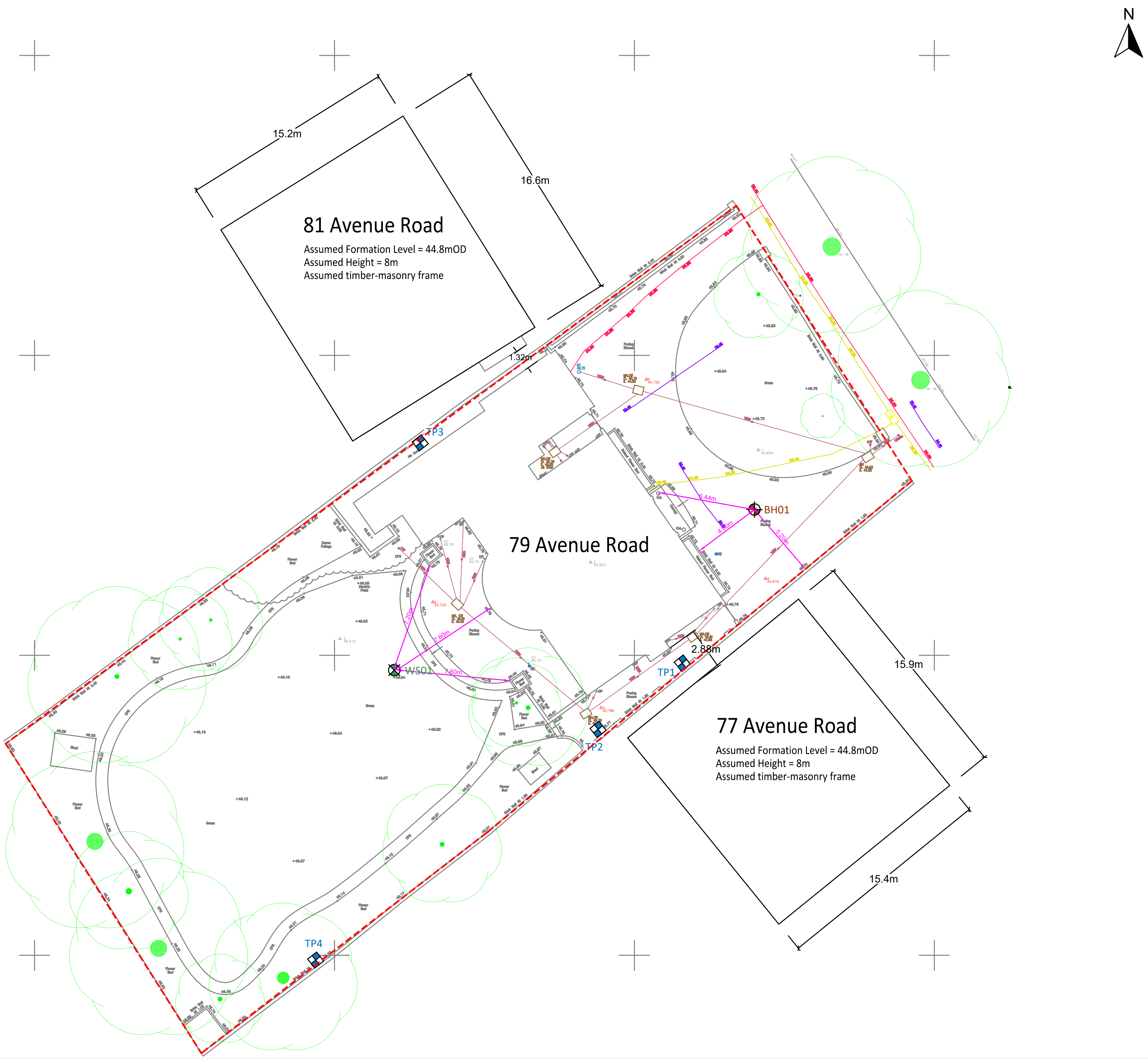
Site



Client	Project	Job No
Form Structural Design Limited	**79 Avenue Road, London Borough of Camden**	**CG/38181A**
CGL	Title **Site Location Plan**	**Figure 1**



KEY

- Borehole
- Window Sample
- Trial Pit
- Site Boundary
- Underground Foul Drainage
- Underground Storm Drainage
- Underground Combined Drainage
- Underground Unidentified Drainage
- Underground Electric
- Underground Telecoms
- Underground CATV
- Underground Water
- Underground Chamber
- Underground Cable Ducts
- GPR Profile
- GPR Reflection

Notes

1. Do not scale from drawing.
2. Topographic survey taken from Laser Surveys drawing 'G9399/5' Rev 0 (July 2019).

Rev	Date	Comments
0	01/11/19	

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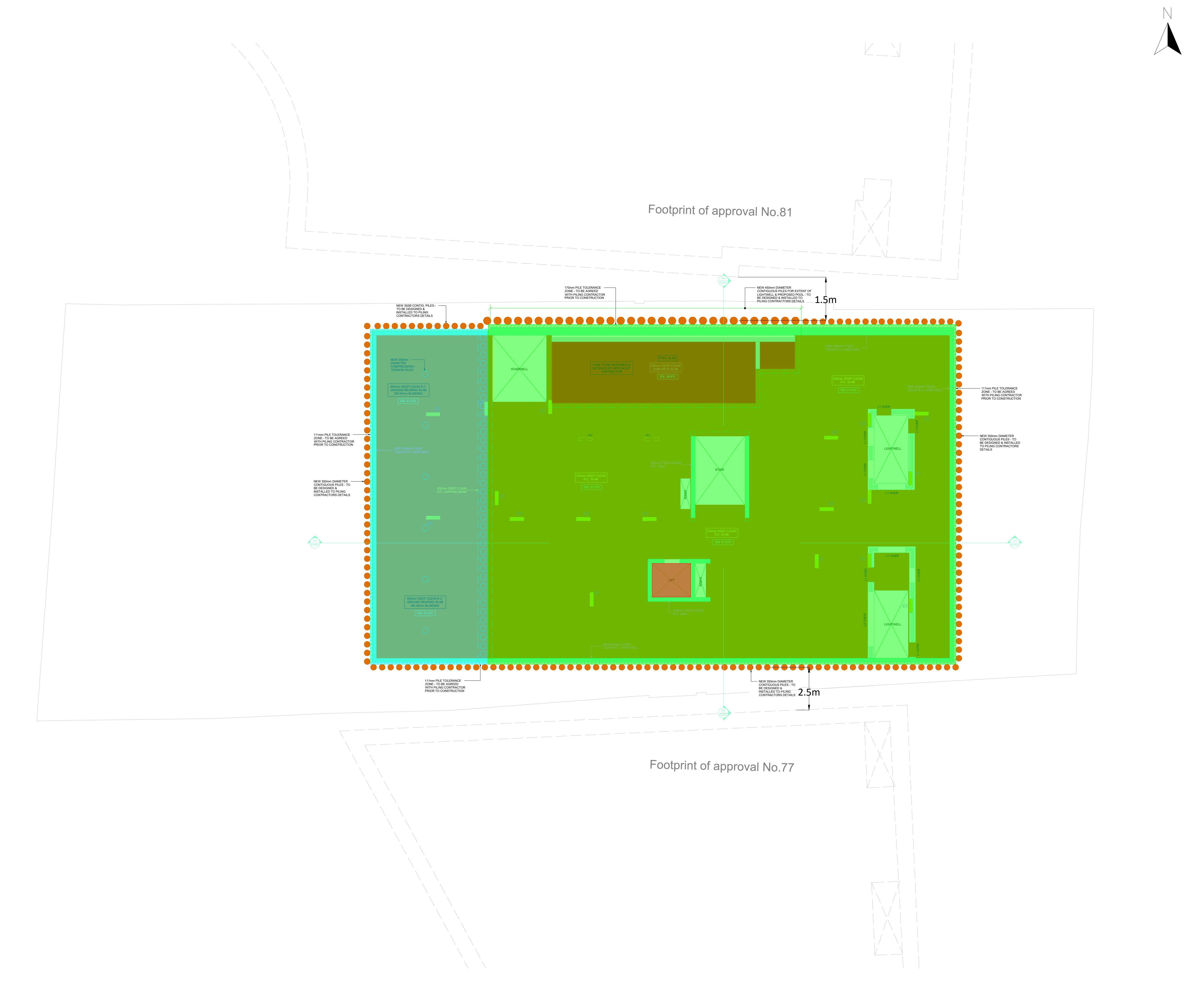
Project: **79 Avenue Road, London Borough of Camden**

Client: **FORM Structural Design Limited**

Drawing title: **Figure 2 - Site Layout and Exploratory Hole Location Plan**

Scale(s)	Job No.
NTS	CG/38181

			Dwg No.	Rev.
Drawn	ZAH	01/11/19	CG/38181-002	0
Checked				
Approved				



KEY

B2 Basement Level
Excavation level = 38.09mOD

B1 Level Basement Level
Excavation level = 41.065mOD

Lift Pit
Excavation level = 36.59mOD

Note: Existing ground level assumed to be 45.8mOD

Rev	Date	Comments
*	*	*

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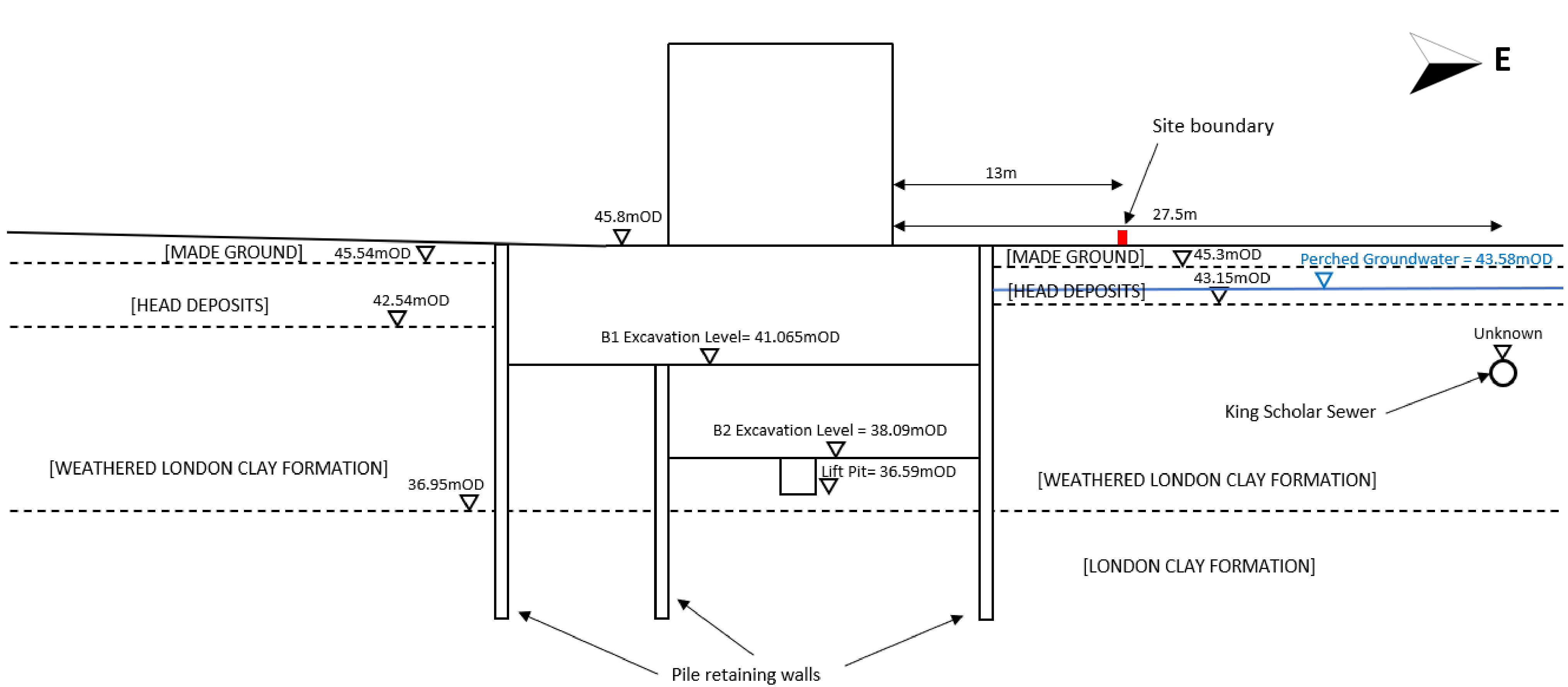
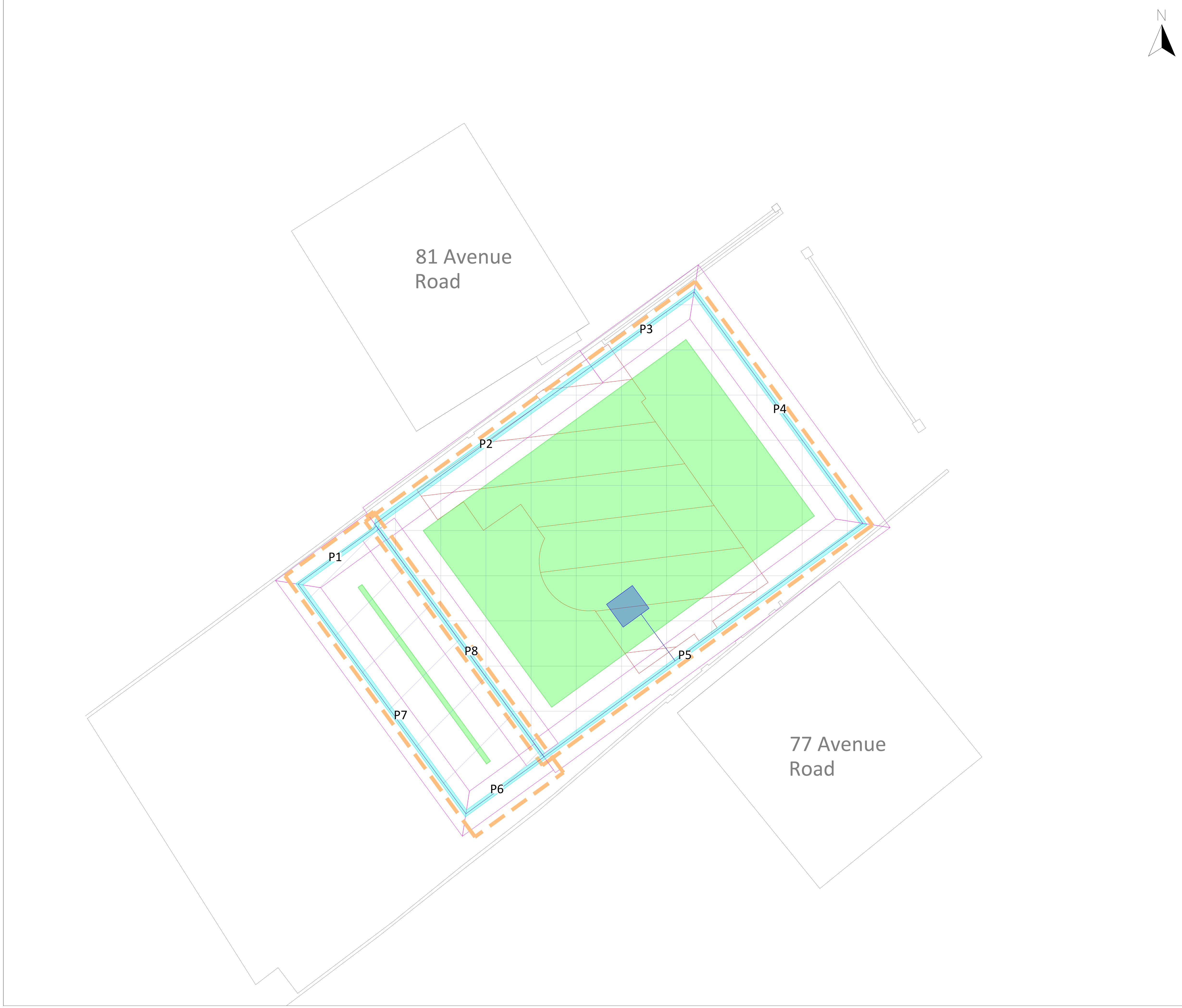
Project	79 Avenue Road, London Borough of Camden
Client	Form Structural Design Limited
Drawing title	Figure 3 - Basement Layout Plan
Scale(s)	NTS
Job No.	CG/38181A
Drawn	TBP 25/05/2021
Checked	
Approved	
Dwg No.	CG/38181A-003
Rev.	0



Client	Project	Job No
Form Structural Design Limited	**79 Avenue Road, London Borough of Camden**	**CG/38181A**
CGL	Title: **Conceptual Site Model**	**Figure 4**



KEY

B2 Basement Excavation = -144kPa
Applied at 38.54mOD

B1 Basement Excavation = -89kPa
Applied at 41.515mOD

Lift Pit = -174kPa
Applied at 37.040mOD

Existing Building Demoliton = -10kPa
Applied at 45.8mOD

Foundation Pile Shaft Bearing
B1 = 7.8kPa, Applied at 26.398mOD
B2 = 15.5kPa, Applied at 26.423mOD

Foundation Pile End Bearing
B1 = 28.1kPa, Applied at 19.07mOD
B2 = 3kPa, Applied at 20.59mOD

P1
Pile End Bearing, North Elevation = 61kPa
Applied at 35.330mOD
Pile Shaft Bearing, North Elevation = 38kPa
Applied at 37.24mOD

P2
Pile End Bearing, East Elevation = 110kPa
Applied at 30.93mOD
Pile Shaft Bearing, East Elevation = 70kPa
Applied at 33.22mOD

P3
Pile End Bearing, South Elevation = 141kPa
Applied at 30.93mOD
Pile Shaft Bearing, South Elevation = 73kPa
Applied at 33.32mOD

P4
Pile End Bearing, West Elevation = 86kPa
Applied at 30.93mOD
Pile Shaft Bearing, West Elevation = 44kPa
Applied at 33.32mOD

P5
Pile End Bearing, B2 = 114kPa
Applied at 30.93mOD
Pile Shaft Bearing, B2 = 59kPa
Applied at 33.32mOD

P6
Pile End Bearing, B2 = 110kPa
Applied at 35.33mOD
Pile Shaft Bearing, B2 = 68kPa
Applied at 37.24mOD

P7
Pile End Bearing, B2 = 100kPa
Applied at 35.33mOD
Pile Shaft Bearing, B2 = 62kPa
Applied at 37.24mOD

P8
Pile End Bearing, B2 = 137kPa
Applied at 35.33mOD
Pile Shaft Bearing, B2 = 70kPa
Applied at 37.24mOD

Rev	Date	Comments
0	25/05/2021	-

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Project: **79 Avenue Road, London Borough of Camden**

Client: **Form Structural Design Limited**

Drawing title: **Figure 5 - PDISP Model Layout and Applied Loads**

Scale(s): **NTS**

Job No.: **CG/38181A**

Drawn	TBP	25/05/2021
Checked		
Approved		

Dwg No.: **CG/38181A-005**

Rev.: **0**

KEY

- Critical Section Line 1
- Critical Section Line 2
- Outline of Basement Level
- PDISP Vertical Displacement Contour Plot

Rev	Date	Comments
0	25/05/2021	

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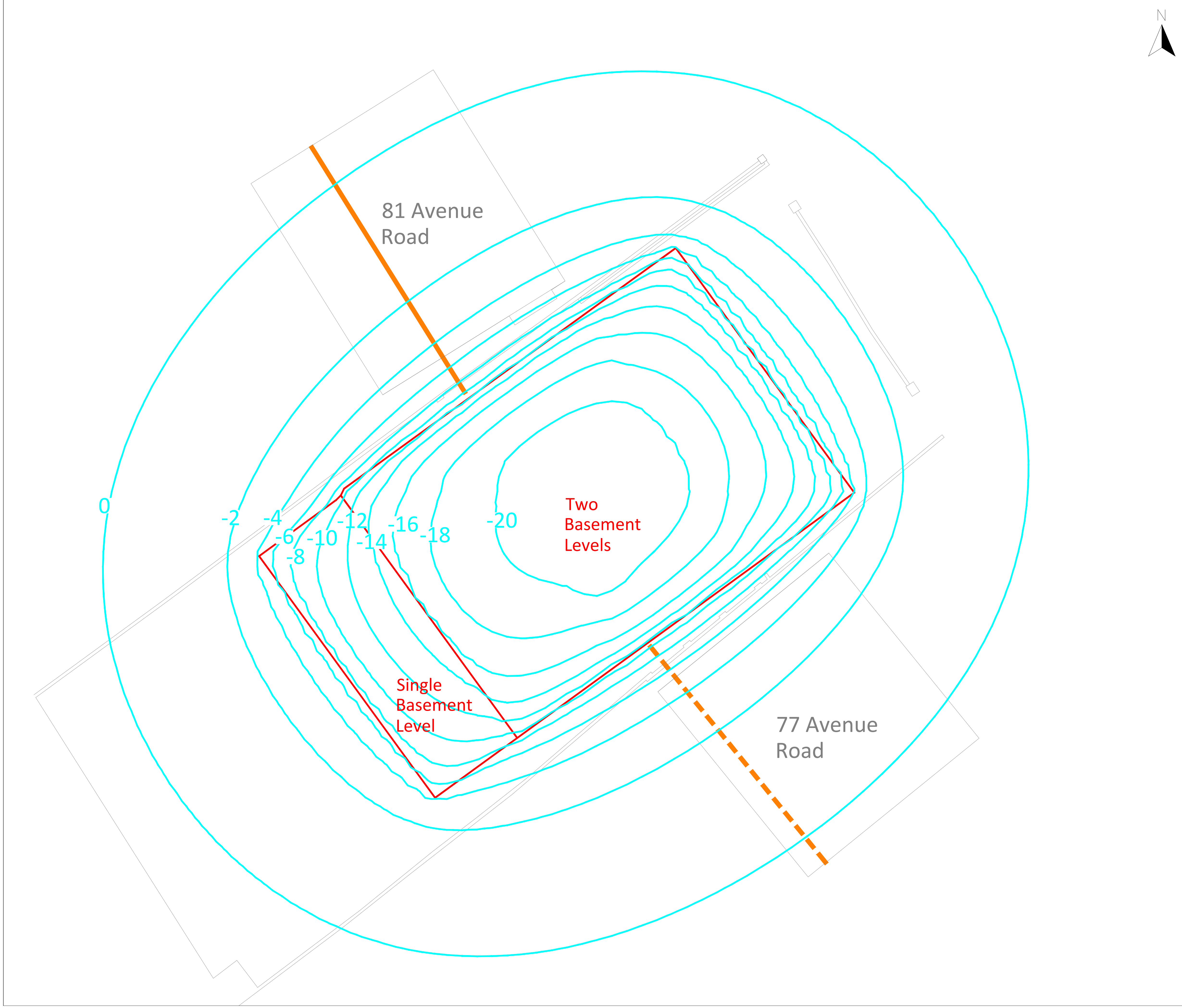
Project	**79 Avenue Road, London Borough of Camden**
Client	**Form Structural Design Limited**
Drawing title	**Figure 6 - Stage 1 (Demolition) PDISP Vertical Ground Movements**
Scale(s)	**NTS**
Job No.	**CG/38181A**
Drawn	TBP 25/05/2021
Checked	
Approved	
Dwg No.	**CG/38181A-006**
Rev.	**0**



KEY

- Critical Section Line 1
- Critical Section Line 2
- Outline of Basement Level
- PDISP Vertical Displacement Contour Plot

Rev	Date	Comments
0	25/05/2021	

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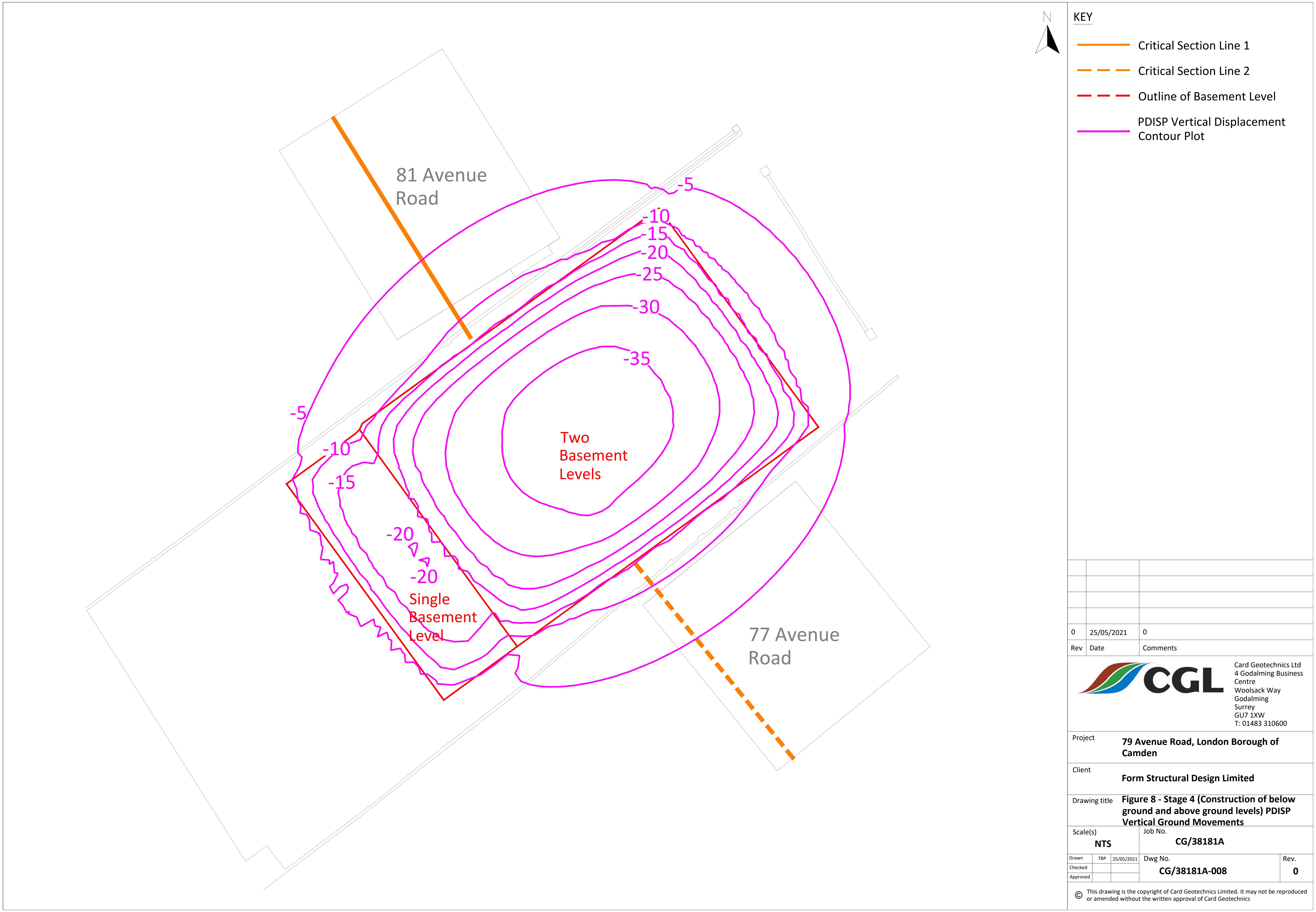
Project: **79 Avenue Road, London Borough of Camden**

Client: **Form Structural Design Limited**

Drawing title: **Figure 7 - Stage 3 (Excavation of the basement) PDISP Vertical Ground Movements**

Scale(s)	Job No.
NTS	CG/38181A

			Dwg No.	Rev.
Drawn	TBP	25/05/2021	CG/38181A-007	0
Checked				
Approved				



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0	25/05/2021	0

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Project: **79 Avenue Road, London Borough of Camden**

Client: **Form Structural Design Limited**

Drawing title: **Figure 8 - Stage 4 (Construction of below ground and above ground levels) PDISP Vertical Ground Movements**

Scale(s): **NTS**

Job No. **CG/38181A**

Drawn	TBP	25/05/2021
Checked		
Approved		

Dwg No. **CG/38181A-008**

Rev. **0**