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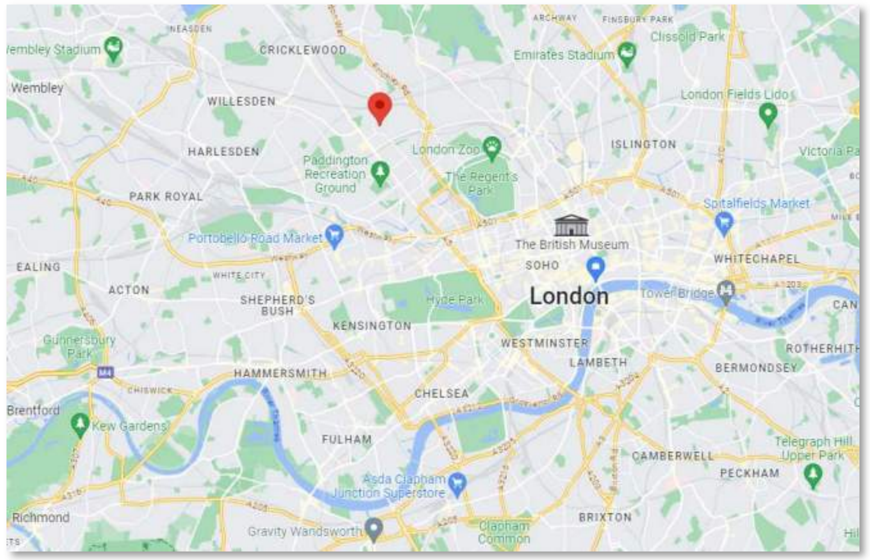

9 WOODCHURCH ROAD, WEST HAMPSTEAD, LONDON NW6 3PL

Construction Method Statement for Subterranean Development

10014-MBP-September 2023

CONTENTS

Revision	Issued For	Description	Date	By
P01	Planning	First Issue	25/07/23	NM
P02	Planning	Report updated following receipt of SI & GMA report.	04/09/23	NM

PREAMBLE

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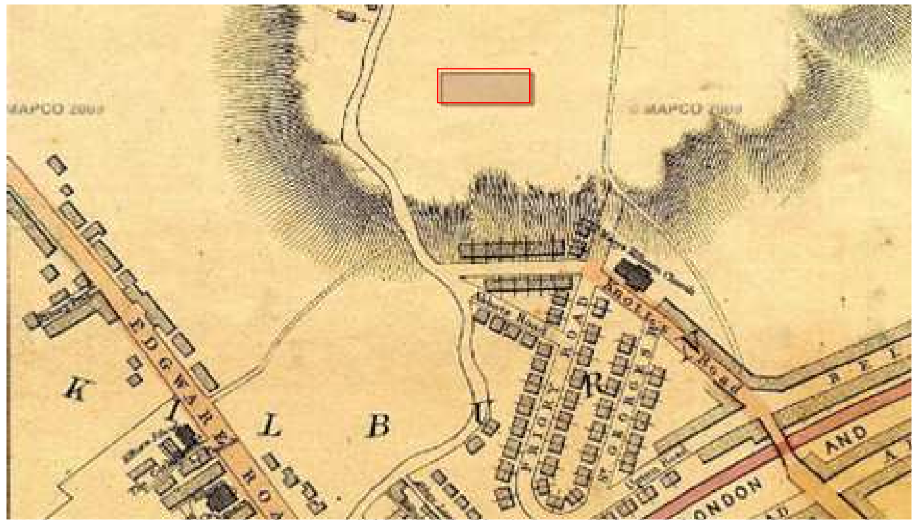


Map of London 1868 by Edward Weller

Ordnance Survey map surveyed 1866, published 1874

Ordnance Survey map Revised: 1893, Published: 1895

1. PREMISE

9 Woodchurch Road is a large 2 storey double fronted detached house with loft in West Hampstead, London, built in circa 1880s and as other London Victorian residential developments at the time is of load bearing London stock brick walls supporting timber floor joists and timber roof, boarded and clad with slate tiles. The walls are founded on corbelled masonry footing. The house has been split into flats and bedsits arranged over the three floor levels. A modern single storey extension is present at the rear and is used as a living room for the ground floor flat. The building has a cellar underneath part of the ground floor flat, most likely used originally for storing coal. There is no access to this cellar from inside the house.

The proposed development includes the excavation of a new lower ground floor level underneath the full length of the west side of the house and half of the east side to create a three-bedroom flat with accommodation arranged over the ground and lower ground floor levels. In addition to this, a new two-bedroom house is to be built where the current garage is, between the gable walls of no. 7 and no. 9.

This report describes the likely structural solution for constructing this development, the interaction of the subterranean structure with the local geology and hydrogeology and its impact on surrounding buildings. Construction techniques are highlighted along with particular requirements for temporary works and excavations.

2. EXECUTIVE SUMMARY

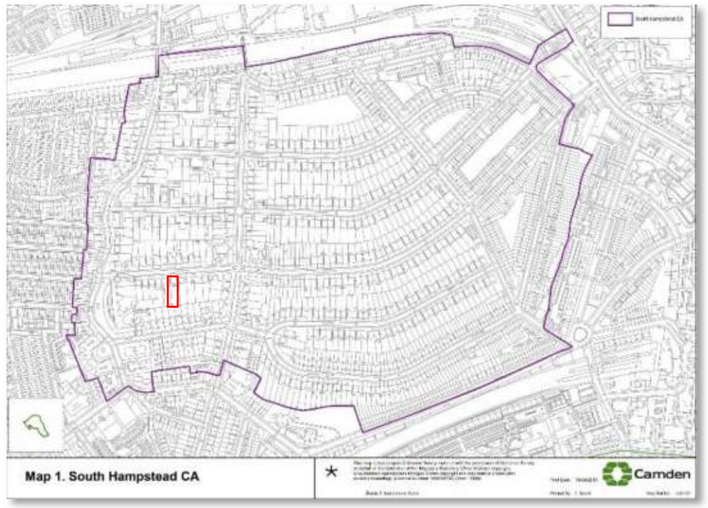
This preliminary report addresses some of the planning requirements imposed by London Borough of Camden as described in their published **Camden Planning Guidance for Basements, January 2021** and **Camden Local Planning Policy A5 for Basements** and provides a preliminary set of information for planning stage, in particular:

- The Desk Study can be found in Section 3.
- The ground and water table information can be found in section 4 of this report and in detail as a separate document by GEA in Appendix F.
- Flood Risk is discussed within the Local Geology & Hydrology in section 4.
- A description of the existing structural form can be found in section 5 of this report.
- Our engineering design for the basement is discussed in detail within section 6 of this report and the relevant drawings are contained within Appendix A.
- Drainage & SuDS are discussed in section 7.
- The proposed construction method & sequence and risk & impact to surrounding buildings are described in sections 9 and 8 respectively.

3. THE SITE AND AREA

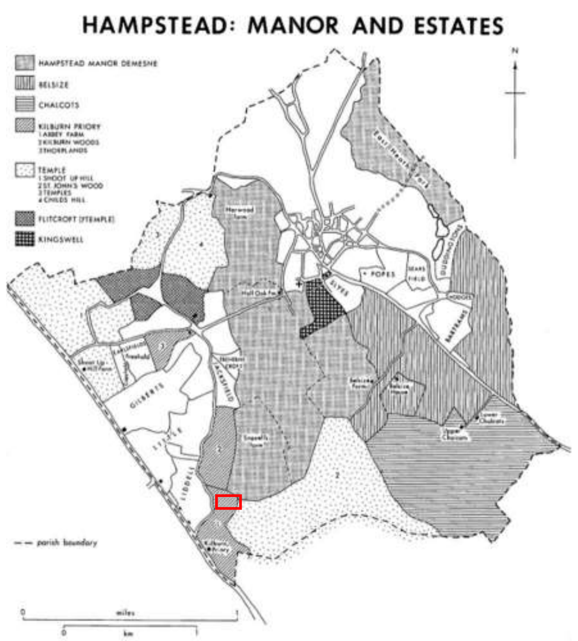
The site is located on Woodchurch Road within the London Borough of Camden and although not a listed building it is within the South Hampstead conservation area. It is approximately 710m northeast of Kilburn High Road overground station, 390m south of West Hampstead underground station, 785m west of South Hampstead station and 2.1km northwest of Primrose Hill. The site is roughly rectangular in shape approximately 15.5m by 39m at its widest and longest points.

9 Woodchurch Road was built on grounds that was once part of Kilburn Woods of Kilburn Priory estate which used to extend, in modern terms, from West Hampstead station in the north to Abbey Road in the south and from West End Lane in the west to Priory Road on the east. Kilburn Priory was part of the Manor of Hampstead and was established in 1134 and endowed by Herbert, abbot of Westminster. Following the dissolution of the monasteries in 1536, the estate was sequestered by the crown and in 1547 was granted to John Dudley, Earl of Warwick, who sold it into private ownership. The remains of Kilburn Priory were demolished around 1790, following which the estate was split and changed hands several times with Kilburn Woods eventually passing to Colonel Henry Perry Cotton, of Quex Park, Isle



LB of Camden Conservation Area Map

Charles Booth's Poverty Map



of Thanet in 1849 who retained the land until 1874. Plans were first drawn-up in 1855 to develop the estate, which was still mainly farmland and pasture, but were delayed by uncertainty over the course of the railway. The earliest development began in the south, north of the existing London & North Western Railway (LNWR) line with new roads built in 1866 and named after places in Kent near the family's estate; Quex, Birchington and Mutrix Roads. A Roman Catholic Church and Wesleyan and Unitarian chapels were built in Quex Road in 1868-9 and at least 55 houses built on the estate between 1871-1885. Canfield (later Priory) Road was laid out on the boundary between Kilburn Woods and the Maryon Wilson estates and some 45 houses were built there between 1877-1882. Parallel roads were laid out to the north and 56 mostly detached and semi-detached houses were built in Acol Road, Woodchurch Road, Cleve Road and Chislett Road between 1874-1886. Woodchurch road was named after a hamlet bordering Quex Park.

There was a greater proportion of 'fairly comfortable, good ordinary earnings' category in Kilburn in the 1890s than in any other district of Hampstead. The most spacious and therefore high-class area was the Kilburn Woods estate, designated middle-class and well-to-do, with Cleve Road and Chislett Road classed as upper-middle and upper-class, wealthy.

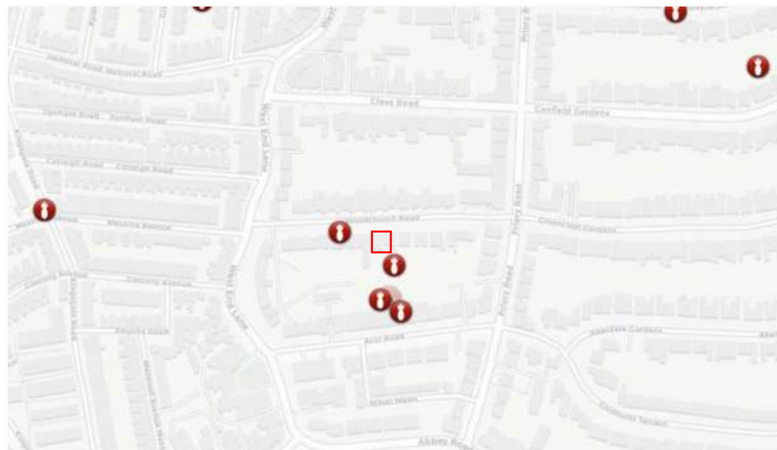
By the end of the 19th century, the estate was very well serviced by the railways, with a station to the north called Finchley Road opened on each of the three lines in 1860 (Hampstead Junction), 1869 (Midland) and 1879 (Metropolitan) respectively. Railway stations were also opened at Kilburn High Road in 1852 by the LNWR and Loudon Road (now South Hampstead) on the Metropolitan Line in 1879.

It was common when the railway network was built to disperse arisings from cutting excavations over adjacent land, which was often poorly compacted and led to settlement problems when that land was developed. West Hampstead and Kilburn High Road stations are approximately 400m north and 450m south of the property respectively which are sufficiently remote from the site that there is unlikely to be arisings beneath ground level due to the construction of these railway lines and stations.

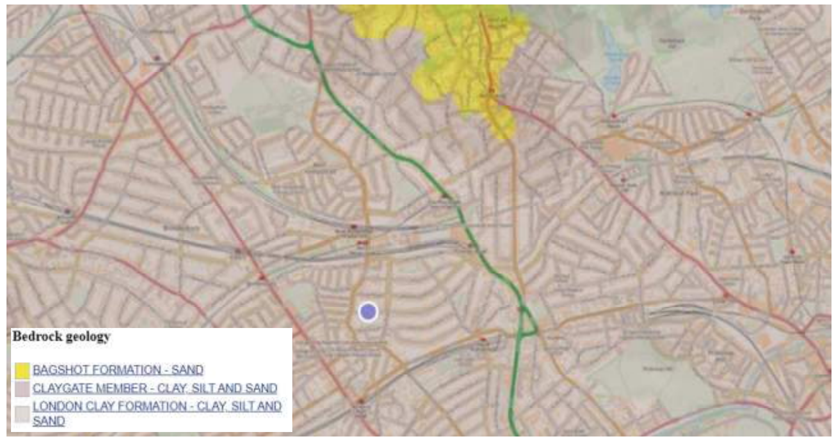
London was heavily bombed during WWII and many areas suffered ordnance damage. Four high explosive bombs were recorded to have fallen during the Blitz on Woodchurch Road, very near to no. 9, and on Acol Road. The LCC Bomb damage map recorded total destruction (black), damage beyond repair (purple) and seriously damaged (pink & dark red) to several buildings on West End Lane. Many properties on Acol Road and Priory Road also suffered blast damage ranging from minor in nature (yellow), general not structural (orange) to seriously damaged (pink & dark red) but 9 Woodchurch Road appear to have escaped damage.

There are a number of mature trees in the front garden of no 7 Woodchurch Road and the rear garden of 9 Woodchurch Road, all of which are capable of influencing and affecting the design of the proposed basement which, in turn, must be detailed to avoid distressing the trees or their roots. Although it is noted that the proposed lower ground floor plan area is a reasonable distance away from surrounding trees, an Arboriculture survey and report is recommended to record the types of trees, their respective root protection area and assess the impact they may have on the proposed new basement.

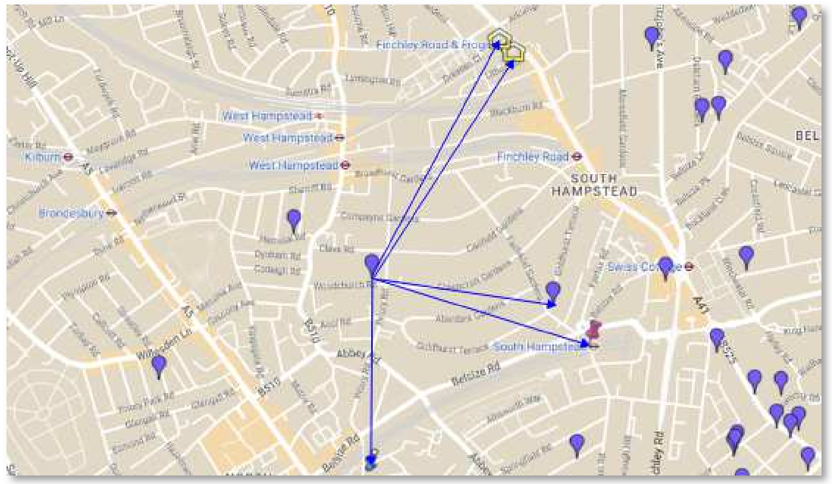
LCC Bomb Damage Map



Blitz Bomb Map



BGS Data of local bedrock geology



Nearby MBP Site Investigations

4. LOCAL GEOLOGY & HYDROGEOLOGY

The British Geological Survey Map indicates that the site is underlain by London Clay Formation of clay, silt and sand. A number of nearby investigations provide more detail:

1. **From an MBP site at Goldhurst Terrace, NW6 (680m-east of the site), two 5m deep window sample boreholes were sunk in September 2019:**
   - **0.5m to 0.8m of MADE GROUND over LONDON CLAY FORMATION described as firm weathered brown fissured silty clay with blue grey along fissure surfaces and rare sand partings. Below a depth of between about 2.2m and 2.5m the clay becomes darker brown in colour suggesting medium strength.**
   - **The boreholes were dry during initial SI, but water was found in both boreholes during monitoring at 0.92m and 1.39m at WS1 and 0.76m and 1.17m at WS2.**
2. **From an MBP site at Loudon Road, NW9 (760m south-east of the site), two cable percussion boreholes to 25m deep sunk in November 2008:**
   - **0.4m to 1.5m of MADE GROUND over LONDON CLAY FORMATION described as firm brown, orange brown and pale grey/greyish brown becoming stiffer with depth, found to a depth of 9m.**
   - **No ground water encountered apart from groundwater seepage at 4.7m associated with clay stones.**
3. **From an MBP site at Finchley Road, NW3 (0.84m north west of the site), two cable percussion boreholes to 30m deep and two window sample boreholes to 4.5m were sunk in between January and February 2016.**
   - **0.2m to 4.7m of MADE GROUND over LONDON CLAY FORMATION described as firm brown laminated clay becoming stiff blue grey at 7.7m and 9.5m.**
   - **Groundwater was found at one of the boreholes at 3m and at 4.1m in the window samples.**
4. **From a BGS Borehole at Priory road, NW3 (555m South East of the site), sunk in Jan 1983:**
   - **0m to 1m of MADE GROUND over LONDON CLAY FORMATION described as firm slightly silty brown mottled grey CLAY with extensive close fissuring up to 3m. After which the clay becomes stiffer over the depth approaching 10m.**
- **No groundwater was enountered during investigation and remain dry during monitoring.**

Each of these investigations is within 1km of 9 Woodchurch Road and all are representative of the near-surface geology in the area and can be expected with a high degree certainty at the site.

A site specific soil investigation by GEA was done in April 2023. A single open drive percussive borehole to a depth of 8m and two window sample boreholes to depths of 3m and 3.8m were sunk in the front and rear gardens. The expected ground conditions were encountered in that, beneath a moderate thickness of made ground, the London Clay Formation was found and proved to the maximum depth of investigation. The made ground generally comprised sandy gravelly clay including fragments of brick, concrete and clinker, plus pottery fragments locally and roots and rootlets in the rear garden, and extended to depths of between 0.40 m and 1.00 m. The London Clay initially comprised firm orange-brown mottled light grey silty clay with partings of orange-brown fine sand and fragments of claystone, becoming stiff fissured grey-brown silty clay with blue-grey mottling with depth and was proved to the base of the boreholes and maximum depth of investigation at 8.00 m.

Groundwater was not encountered at the time of the investigation. Standpipes were installed to 5m and groundwater monitoring were carried out 6 weeks after the initial investigation and was found to be dry. In any case, the basement walls including waterproofing and drainage strategy will need to be designed to suit requirements set out in EC7 and BS8102.

GEA's SI report note that the formation level of the proposed basement is expected at 3.4m below existing ground level and should therefore be within the London Clay. Therefore, it should be possible to adopt spread foundations at basement level where they may be designed to apply a net allowable bearing pressure of 150 kN/m^2. Other recommendations within the preliminary report include the provision of heave protection for the basement slab or suitably reinforced to cope with the upward movements and for provision of temporary supports during excavation works to maintain the stability of the pit and surrounding structures at all times. Significant inflows

The EA zone map of flood risk from rivers or the sea

of any perched groundwater are not anticipated within the basement excavation at this stage, although the contractor should have plans in place to deal with possible inflows from within the made ground.

The Environment Agency's Flood Map for planning indicates that 9 Woodchurch Road lies within flood zone 1, an area that is considered Low Risk, having between less than 1 in 1,000 annual probability of river or sea flooding and therefore does not require a full Flood Risk Assessment (FRA) for planning although one has been prepared by The PES dated 8th March 2023 and is submitted as a separate document as part of the planning application of this site.

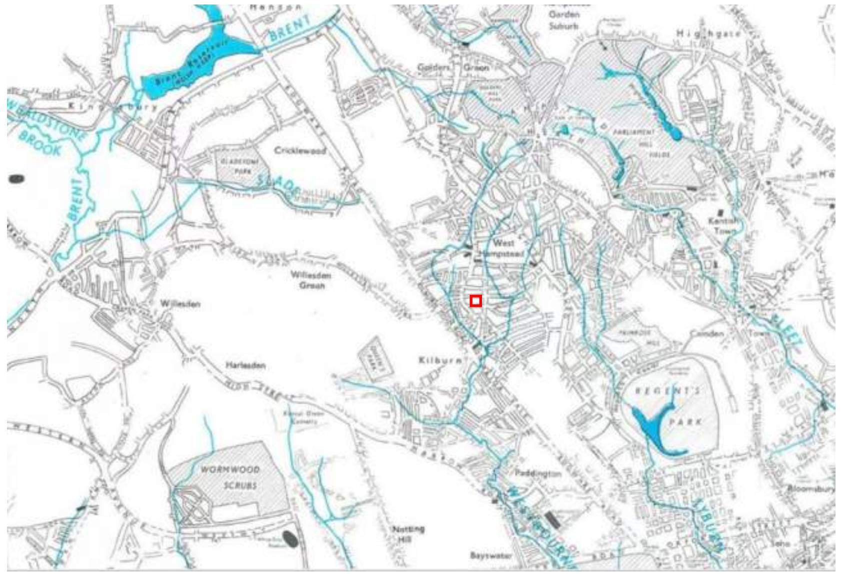
In the FRA document, it is noted that the site is within a zone of Very low Risk having less than 0.1% chance of flooding from rivers or the sea, as shown in the EA's flood map below. There is no historical record of fluvial flooding at the site. The EA flood map site also notes that the site has a very low risk of flooding from surface water flooding although it should be noted that areas adjacent Woodchurch Road has been categorised as having a higher risk to surface flooding ranging from low risk (between 0.1% to 1% annual probability) to medium risk (between 1% and 3.3% annual probability). There is no historical record of surface water flooding at 9 Woodchurch Road

There is no recorded risk of flooding from reservoir on site.

The proposed development does not create an increased area of hard standing so the run-off from site to the public storm water sewer will remain the same in volume and flow rate. However, the new below ground drainage system will be fitted with anti-flood valve and designed to cope with local surface flooding as well as required uplift for climate change.

The EA zone map of flood risk from surface water

The EA zone map of maximum extent of flooding from reservoirs



Lost rivers of London Map

House viewed from Woodchurch Road

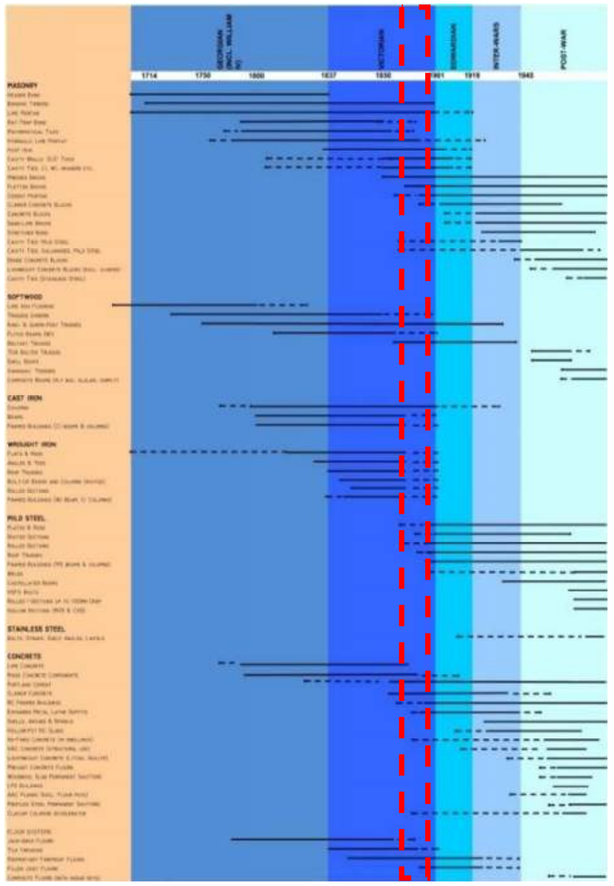
5. THE EXISTING SITE

According to the desk study, the site was originally farmland and part of Kilburn Woods on the Kilburn Priory estate. The woods and farmland were eventually cleared for development as London expanded in the 19th century. Today, there are no remaining traces of the woods or the priory, although some of the local street names, such as Priory Park Road and Woodfield Road serve as reminders of their existence.

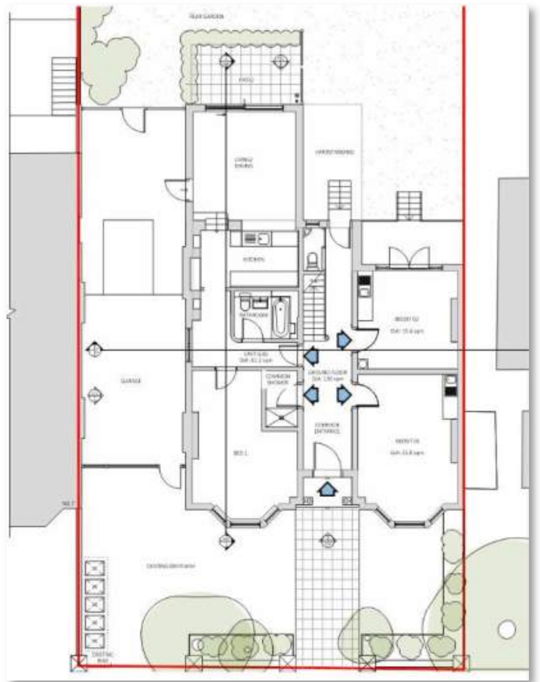
The area remained largely undeveloped until the 1860s and 1870s when several railways including the Hampstead Junction, Midlands and the Metropolitan line were established and train stations at Finchley Road, Kilburn High Road and Loudon Road (now South Hampstead) were opened. 9 Woodchurch Road was built between 1874 and 1886 as part of the Powell-Cotton family estate development. The property is a two-story detached house with rooms in the loft and a cellar under part of the ground floor. Houses in the area are typically set back from the main road with small front gardens and larger rear gardens. The structure is made of London stock bricks, and the front wall has been painted crimson. The symmetrical front elevation includes double-height bay timber sash windows with ornate cornices. The entrance has a portico with Romanesque columns. The structure of the house is of a typical Victorian construction of load bearing masonry walls founded on corbelled masonry footing. The walls support timber floor joists and timber roof. Stability is by means of masonry cross walls. The house has been split into flats and bedsits arranged over the three floor levels. There is a recently built single storey rear extension which houses the living room of the ground floor slat. The new extension is of masonry cavity construction supported on mass concrete strip footings, with a suspended timber ground floor and timber roof. An existing cellar, which is accessed externally through a low door on the east gable wall, is present under part of the ground floor flat. The cellar was most likely used as storage space for coal. The cellar cannot be access from inside the house.

The surrounding buildings are built around the same time as 9 Woodchurch Road and are in a good condition with no evidence of distress or damage to the construction or fabric of the building, such as bulges, cracks, dampness or decay. There is therefore no evidence or suggestion that their construction cannot tolerate the proposed works, both during their execution or when complete.

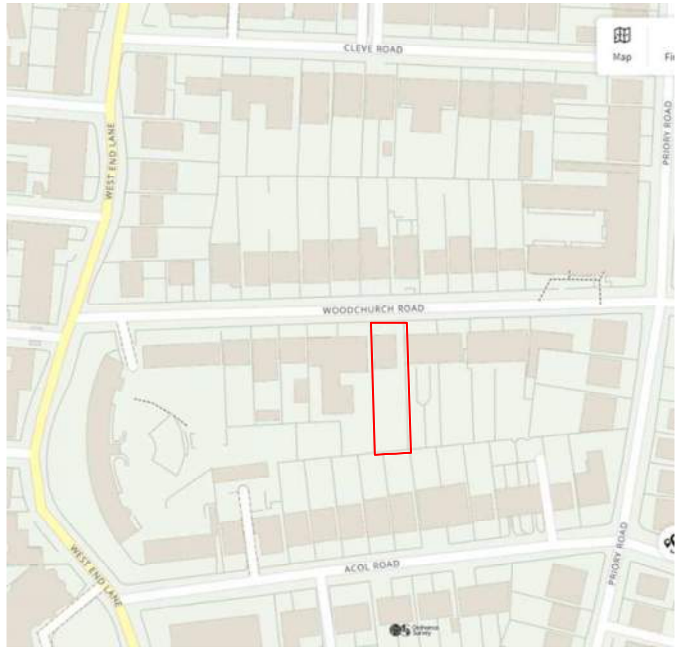
A site-specific ground investigation has been undertaken by GEA in April 2023 which included a single open drive percussive borehole to 8m depth and two window sample boreholes to 3m and 3.8m deep done in the front and rear gardens on the north and south side of the house which revealed the underlying soils to be made ground on London Clay. The ground site investigation and ground movement analysis report by GEA is submitted separately as part of the planning submission of this site.



Construction Timeline

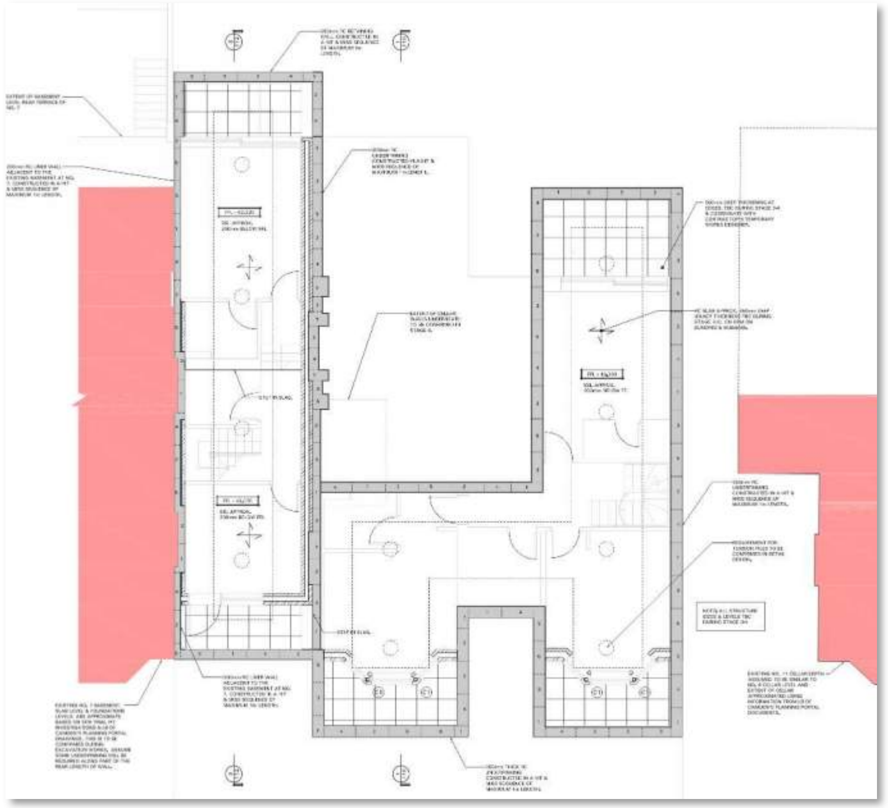


Existing ground floor layout

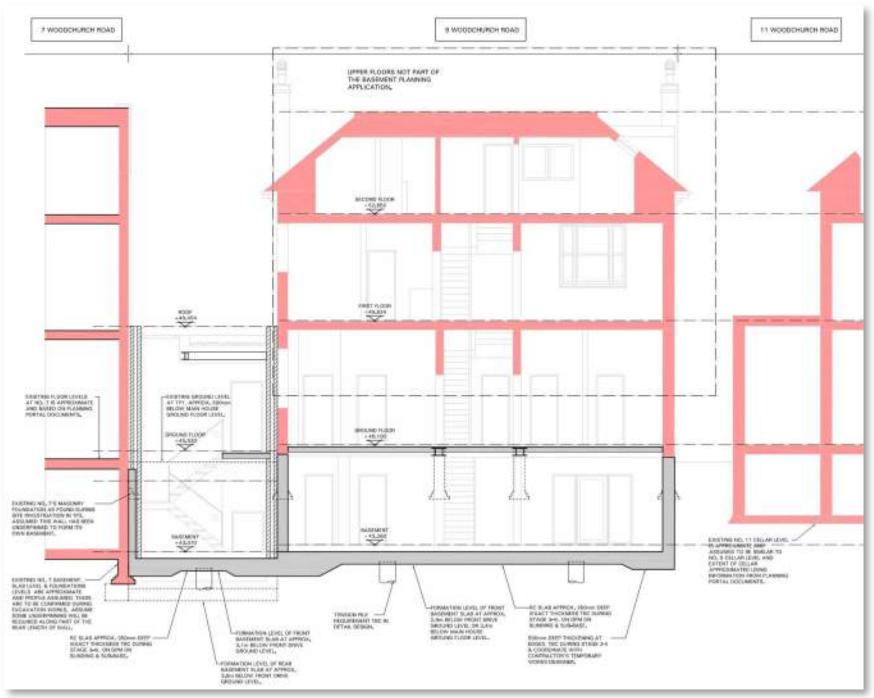


OS Map from Planning Portal

MICHAEL BARCLAY PARTNERSHIP



Proposed Lower ground floor plan



Proposed Cross Section

6. THE PROPOSED DEVELOPMENT

The proposed works involves the excavation of a lower ground floor to form a full height habitable space underneath the full length of the east bay of the house and up to half of the west bay of the house. A new house is also proposed to be built, which will have accommodation on the lower ground and ground floor levels, on the site of the current garage and in between the flank walls of no. 7 and no 9 Woodchurch Road.

The proposed structure involves the existing main house walls to be underpinned with reinforced concrete constructed in maximum 1m lengths in a traditional hit and miss sequence. From the desk study and site visual observations and investigation, it is known that 7 Woodchurch Road already has a lower ground floor which extends the full length of their building with lightwells to the front and a lower ground floor level terrace to the rear. One of the trial pits along the flank wall of no. 7 revealed a corbelled footing at around 1.2m below ground level and therefore it is assumed that this wall has been underpinned to form the lower ground floor at the property. Along this wall, a reinforced concrete liner wall is proposed which like the underpins is to be constructed in a hit and miss sequence. The new lower ground floor will be a reinforced concrete raft slab designed to the allowable bearing pressure of 150kN/m^2 and suitably reinforced to deal with the net heave pressure. New lightwells are to be formed at the front and rear of the plot which are to be constructed as reinforced concrete retaining walls.

6.1 BELOW GROUND LEVEL

The new lower ground floor, along with the new ground floor slab it will support, will be constructed in reinforced concrete. The proposed formation level of the new basement at its deepest is at 3.4m bgl. Removing soil to accommodate the basement will relieve some of the pressure on the underlying London Clay: However, there will be the weight of the existing and new construction imposed around the perimeter and at new basement slab level and we estimate that this relief will not be significant, will not lead to noticeable swelling of the clay and so will not impact significantly on the surrounding buildings and foundations, which has been our experience empirically and theoretically in similar developments in this area of London. There is currently no survey of the existing services and therefore a survey before works commence will be required to identify, establish and protect if necessary, during the construction process.

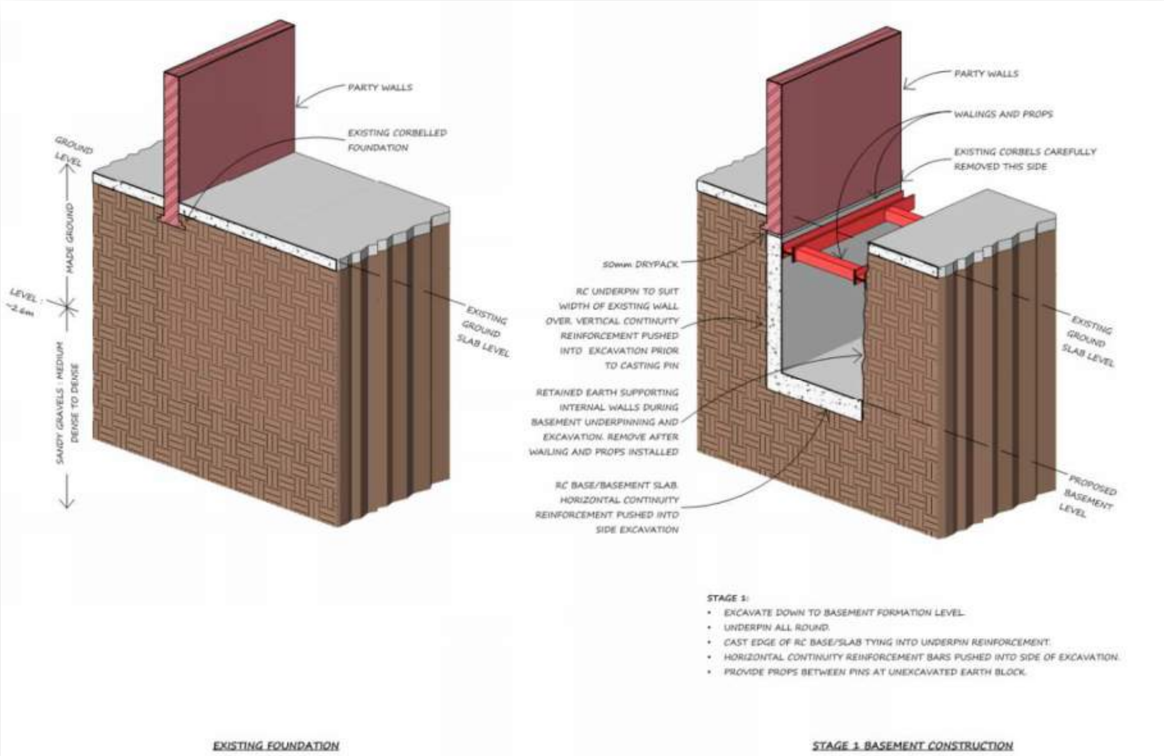
At the time of the site investigation the three boreholes remain dry and as yet we have no information of the results of the subsequent monitoring work. If ground water is found, then appropriate control measures will likely be required and suitable contingencies put in place. Advice from a specialist basement waterproofing contractor should be sought regardless in regard to installing the appropriate waterproofing system within the basement and this is expected to be a combination of either *Type A (barrier)*, *Type B (structurally integrated)* or *Type C (Drained)* protection against ingress of water, as defined by BS 8102:2009 to be constructed and detailed to achieve a Grade 3 Level of Performance, as defined by BS 8102:2009.

Table 2 **Grades of waterproofing protection**

Grade	Example of use of structure A)	Performance level
1	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp areas tolerable, dependent on the intended use B) Local drainage might be necessary to deal with seepage
2	Plant rooms and workshops requiring a drier environment (than Grade 1); storage areas	No water penetration acceptable Damp areas tolerable; ventilation might be required
3	Ventilated residential and commercial areas, including offices, restaurants etc.; leisure centres	No water penetration acceptable Ventilation, dehumidification or air conditioning necessary, appropriate to the intended use

A) The previous edition of this standard referred to Grade 4 environments. However, this grade has not been retained as its only difference from Grade 3 is the performance level related to ventilation, dehumidification or air conditioning (see BS 5454 for recommendations for the storage and exhibition of archival documents). The structural form for Grade 4 could be the same or similar to Grade 3.

B) Seepage and damp areas for some forms of construction can be quantified by reference to industry standards, such as the ICE's *Specification for piling and embedded retaining walls* [1].



We propose that the new lower ground floor is achieved using traditional underpinning techniques and sequencing to build the walls in stages. A minimum of 48 hours must pass before an adjacent excavation can begin. Although a lengthy process, underpinning by hit-& miss-sequencing is a low-impact technique that permits the maximum space to be achieved and has the least impact on existing constructions, boundaries and the like. Casting the wall in pins controls the extent of soil exposed, avoids extensive temporary works and they can be controlled in size and sequence to reflect and accommodate the condition and capability of the walls they will be built beneath.

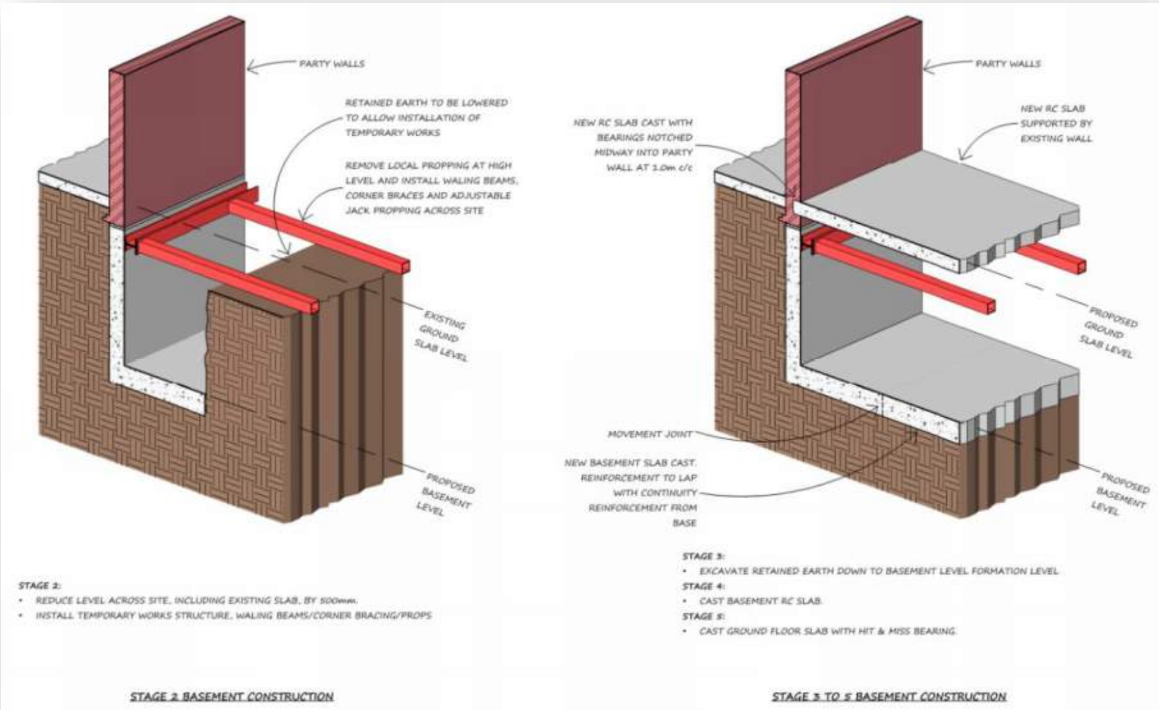
The formation of the basement will require the excavation and removal of between 2.8m to 3.4m deep of made ground and clay soils and will result in an unloading of about 60kN/m^2. Such heave that may occur will mostly, i.e. >50%, happen immediately on excavation and during the works, leaving residual pressure of approximately half the initial unloading that the new construction will need to accommodate. In this case 30kN/m^2 of heave should be allowed for in the design of the basement structure. Hydrostatic uplift pressure has also been considered and the requirement for tension piles will be confirmed in the detail design stage.

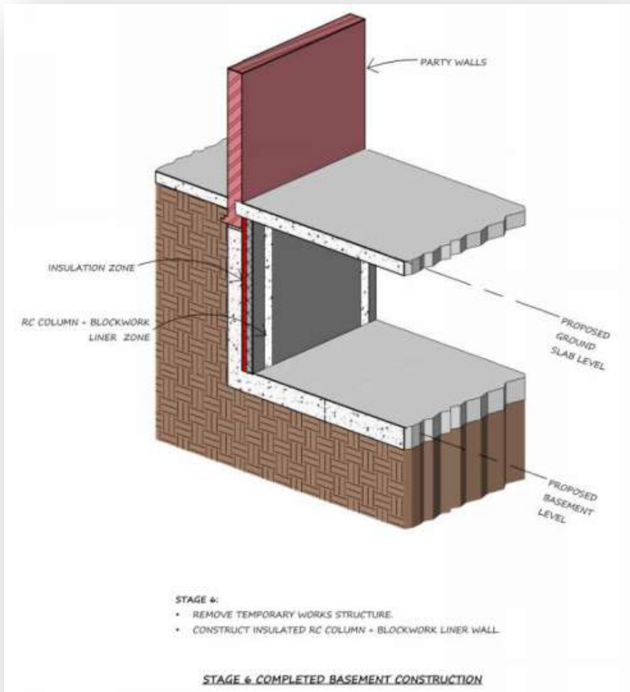
The basement slab will be a thick, reinforced concrete raft cast on a suitable sub-base and will be formed off the underlying London Clay with an allowable bearing resistance of 150kN/m^2, a construction that will allow bearing pressure to be generated evenly across the plot. The slab will be suitably reinforced to deal with the net uplift pressures.

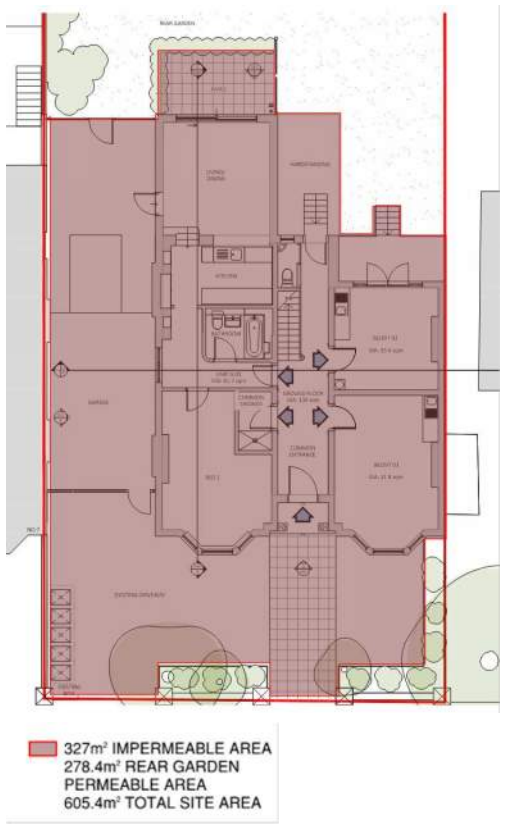
A detailed and considered temporary works strategy by the contractor is required to ensure the underpinning and retaining walls are adequately supported in the temporary case until the new basement and floor slab are constructed.

6.2 ABOVE GROUND LEVEL

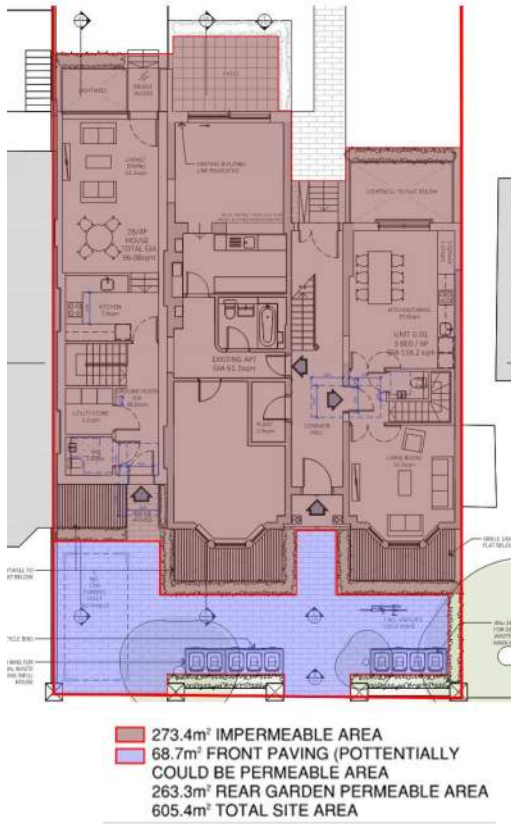
The main house above ground floor level is to be retained and some alterations proposed to the layout of the upper floors which are not covered in this planning application and will not be discussed in detail in this report.







Existing Ground floor Plan



Proposed Ground floor plan

Most Suitable	SuDS technique	Flood Reduction	Pollution Reduction	Landscape & Wildlife Benefit
	Green roofs	✓	✓	✓
	Basins and ponds 1. Constructed wetland 2. Balancing ponds 3. Detention basins 4. Retention ponds	✓	✓	✓
	Filter strips and swales	✓	✓	✓
	Infiltration devices 5. Soakaways 6. Infiltration trenches and basins	✓	✓	✓
	Permeable surfaces and filter drains 7. Gravelled areas 8. Solid paving blocks 9. Porous paviours	✓	✓	
Least Suitable	Tanked systems 10. Oversized pipes/ tanks 11. Box storage systems	✓		

SuDS hierarchy (from the London SDA Plan Table 1 & Box 1)

7. DRAINAGE & SUDS

In the existing condition, rain and surface water infiltration into the ground are restricted to areas of soft landscaping within the rear and front gardens, although the rates are likely limited by the cohesive London Clay ground beneath the site. The surface runoff from the building, paved drive and garage are likely to drain directly into the combined sewers in the road. The new lower ground floor and single unit house will occupy areas where hard standing or existing structure are already present. It is estimated that the existing impermeable areas created by the garage, front driveway and house is 327m^2, while the new development's impermeable areas created by the new lower ground floors, lightwells and new single unit house will be 342m^2 or an estimated increase of 15m^2 or 5%. However, a significant proportion of the site will remain as soft landscaping. In addition, the low permeability of the underlying London Clay would result in a low recharge in any case and consequently there would be little to no effect on groundwater.

The proposed development will create additional habitable space and therefore will generate an increase in discharge to the public sewer than it has currently. Although not yet designed, it is anticipated that the scale and scope of the development will require a new below ground drainage system to be provided by combining gravity flow from the upper floors and roof with a new pumped flow from the lower ground floor level. The final connection between this system and the public sewer, as highlighted in the London Sustainable Drainage Action Plan, will include an anti-flood or non-return valve to protect the property from surcharges in the public sewers. The system will also be designed to cope with local surface flooding as well as the required uplift for climate change.

The London Borough of Camden's Flood Risk Management Strategy indicates that the site is within Group 3-010 Critical Drainage Area, which the council's Surface Water Management Plan defines as "A discrete geographic area (usually a hydrological catchment) where multiple and interlinked sources of flood risk (surface water, groundwater, sewer, main river and/or tidal) cause flooding in one or more Local Flood Risk Zones during severe weather thereby affecting people, property or local infrastructure." The Technical Note - Flood Risk Assessment report done by The PES dated 8th March 2023 assesses the sources of flooding at the site, identifies the relevant national and local planning policies and guidance for the proposed development and associated on/off site flood risks.

7.1 BELOW GROUND DRAINAGE – EXISTING

A CCTV survey has not yet been done on the property. However, as other properties of this type it is assumed that the existing building collects all the foul and surface water above ground and discharges it into the local sewer network under gravity. Three manholes are present in the front driveway which are assumed to be fed directly by rainwater pipes, RWP, and soil vent pipes, SVP, without any attenuation or water storage.

7.1 BELOW GROUND DRAINAGE - PROPOSED

The redeveloped internal layouts above ground level are assumed to use the existing as well as new network of SVP's and RWP's which will collect and discharge the foul and surface water into the sewer under gravity. The remainder and all the foul and surface water at the new basement level will be collected in separate pumping chambers and then pumped up to the ground floor level to be discharged through the last manhole into the sewer network. There may be ability to incorporate rainwater harvesting before this point to add attenuation to the system.

There will be two types of pump chamber included in the scheme:
Type 1: To collect and manage the Foul Water from the installations below basement level and to collect and manage any Surface Water that passes below the ground floor level.
Type 2: To collect and manage any Ground Water that passes into the drained cavity secondary waterproofing system adopted in the basement construction.

Both these chambers have storage capacity, to attenuate the flow into the sewer system, and operate on a "backup" dual pump arrangement. They are also normally attached to the Building Management Systems, BMS, and included in the alarm system for the

Box 1: London Plan Policy 5:13 Sustainable Drainage

Planning decisions

A. Development should utilise sustainable urban drainage systems (Sustainable drainage) unless there are practical reasons for not doing so, and should aim to achieve greenfield run-off rates and ensure that surface water run-off is managed as close to its source as possible in line with the following drainage hierarchy:
1. store rainwater for later use
2. use infiltration techniques, such as porous surfaces in non-clay areas
3. attenuate rainwater in ponds or open water features for gradual release
4. attenuate rainwater by storing in tanks or sealed water features for gradual release
5. discharge rainwater direct to a watercourse
6. discharge rainwater to a surface water sewer/drain
7. discharge rainwater to the combined sewer.

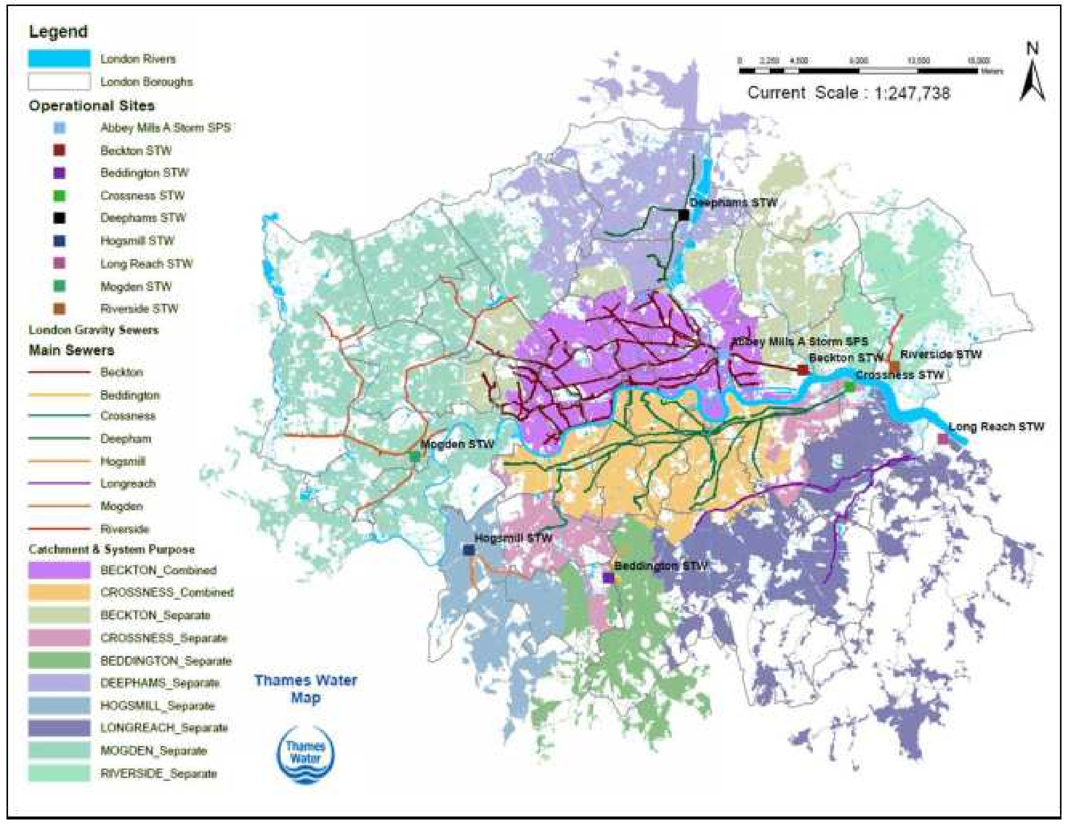
Drainage should be designed and implemented in ways that deliver other policy objectives of this Plan, including water use efficiency and quality, biodiversity, amenity and recreation.

LDF preparation

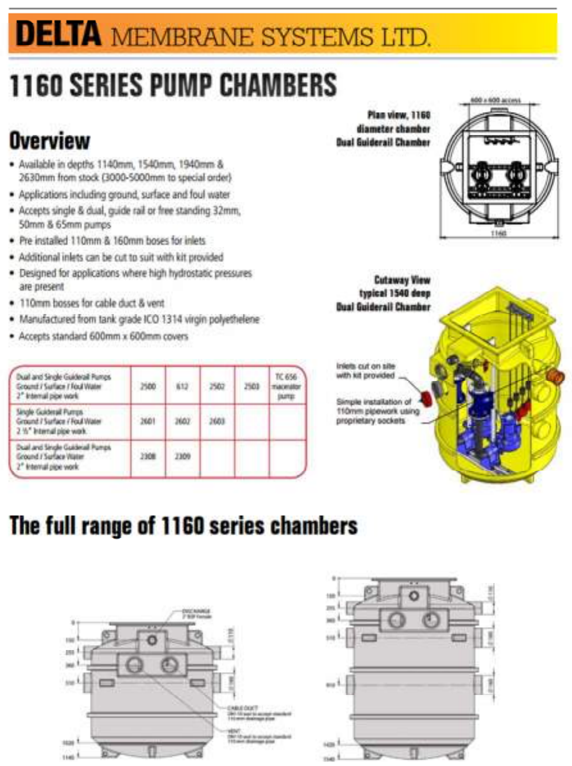
B. Within LDFs boroughs should, in line with the Flood and Water Management Act 2010, utilise Surface Water Management Plans to identify areas where there are particular surface water management issues and develop actions and policy approaches aimed at reducing these risks.

property. Both these systems have their own integrated non-return valve flood protection. The sizes of the chambers vary with Type 1 being the largest, and Type 2 being the smallest as any flow through a piled wall and waterproof reinforced concrete liner wall is unlikely.

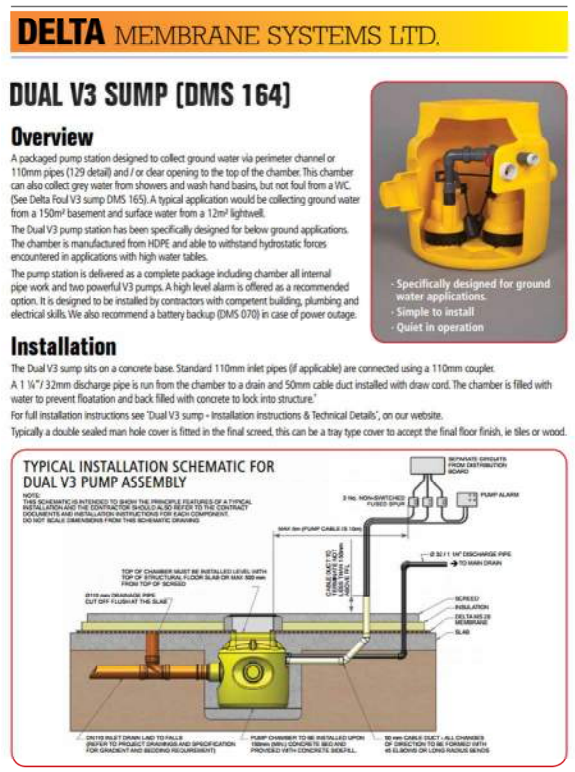
The figures below indicate the Type 1 and Type 2 chambers that provide a positive pumped device as part of the flood management system.



Combined and separated sewer system (Source: Thames Water/London Sustainable Drainage Action Plan)



Type 1 – Typical positive pumping chamber for SVP's and RWP's to be incorporated in the basement.



Type 2 – Typical positive pumping chamber for drained cavity waterproofing to be incorporated in the basement.

MICHAEL BARCLAY PARTNERSHIP

MBP

Damage Category	Description of Typical Damage	Approximate Individual Crack Width
Negligible (0)	Hairline cracks	< 0.1 mm
Very Slight (1)	Very slight damage includes fine cracks which can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external brickwork visible on close inspection.	1 mm
Slight (2)	Slight damage includes cracks which can be easily filled and redecoration would probably be required, several slight fractures may appear showing the inside of the building, cracks which are visible externally and some repointing may be required, and doors and windows may stick.	< 5 mm
Moderate (3)	Moderate damage includes cracks that require some opening up and can be patched by a mason, recurrent cracks that can be masked by suitable linings, repointing of external brickwork and possibly a small amount of brickwork replacement may be required, doors and windows stick, service pipes may fracture, and weather-tightness is often impaired.	5 mm to 15 mm or a number of cracks > 3 mm
Severe (4)	Severe damage includes large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows), distorted windows and door frames, noticeably sloping floors, leaning or bulging walls, some loss of bearing in beams, and disrupted service pipes.	15 mm to 25 mm but also depends on the number of cracks
Very Severe (5)	Very severe damage often requires a major repair job involving partial or complete rebuilding, beams lose bearing, walls lean and require shoring, windows are broken with distortion, and there is danger of structural instability.	> 25 mm

Table 1: Severity of Cracking Damage[4, 5]

8. RISKS TO & IMPACT ON SURROUNDING BUILDINGS

The proposed development is a relatively low-level, low-density construction, and will occupy a smaller plan area constructed directly beneath the property's footprint. The new housing unit is also low-level and low-density construction, to be constructed in between the flank walls of no. 7 and no. 9 Woodchurch Road.

The surrounding buildings, which are mainly of Victorian building stock, fall into Group 1a as defined by BS ISO 4866:2010, i.e. **Ancient, Historical** or **Old**; the foundations fall in to Classes B and the soil as Type e. From Table B.1 of BS ISO 4866 the surrounding buildings therefore fall within Category 6 and can be considered to have a medium resistance to vibration. From Table B.2 of BS ISO 4866 these buildings fall into Classes, which is deemed to have a medium level of resistance to vibration and, conversely, to require medium protection against vibration for the types of works intended.

- Although the construction will be further below ground level than the existing site level it will not be significantly deeper than the lowest level of the surrounding buildings. In any case, a ground movement analysis has been completed to predict the likely movements as a result of the excavation and is presented in Part 3 of GEA's Desk Study, Ground Investigation, Basement Impact Assessment and Ground Movement Analysis report.
- The site investigation and subsequent monitoring found the site to be dry and so a continuous shallow ground water table is not anticipated beneath the site. In addition, the formation level of the new lower ground floor slab will be on London Clay, which is not capable of supporting groundwater. It will also not be deeper than the existing lower ground floor level at no. 7 Woodchurch Road and the size and the scope of the excavation is relatively small. On this basis, the basement is not expected to interfere with the natural flow of the groundwater.
- The building will be formed off of the stiff underlying London Clay, which has a significant bearing capacity, and the foundations will be designed to reflect the recommended permissible pressures and ensure that settlement remain within tolerable limits.
- Removal of the underlying soils will generate some heave in the underlying London Clay. However, the basement structure and the superstructure load including allowance for tension piles should mitigate the effects of these upward forces.
- The external and internal walls of the detached property can be retained safely and easily following industry-standard practices and, by following a pre-determined sequence which will allow the basement walls to be constructed without detriment to the existing, surrounding construction.
- Excavations for the pins that form the new basement walls can be undertaken using a small excavator, which will be low-impact technique and known not to generate excessive vibration.

A Ground Movement Analysis has been done by GEA, which addressed the potential horizontal and vertical movements which may occur as a result of underpinning and retaining wall installation, basement excavation and loading by new foundations. Nos. 7 and 11 Woodchurch Road to the east and west of the site are the closest structures to the proposed excavation and the analysis assessed the magnitude of ground movement beneath these nearby properties. Part 3 of GEA's report notes that their analysis indicates that damage to no. 7 and 11 Woodchurch Road fall within categories 0 (negligible) or 1 (very slight) where damage can be treated by normal redecoration. They note that the predicted maximum tensile strain along walls E & G is close to the boundary to category 2 damage and for this reason, they recommend that movements are strictly limited to a maximum of 5mm.

To ensure that any damage is limited to category 0 and 1 on The Burland Scale, a controlled and sequenced work process needs to be adopted and a robust temporary support system employed during the works to ensure that lateral movements of the retaining structure are minimised. A monitoring regime, forming the party wall agreement, may be used to keep track and limit movements of the structure and adjacent properties during key stages of the construction. In the permanent case, the retaining wall is to be designed to have lateral restraint provided by the ground floor and new lower ground floor.

Conveyor to Remove Excavation Arisings

Shored excavation for an underpin using timber

Small wagon

9. CONSTRUCTION METHODS & SEQUENCE

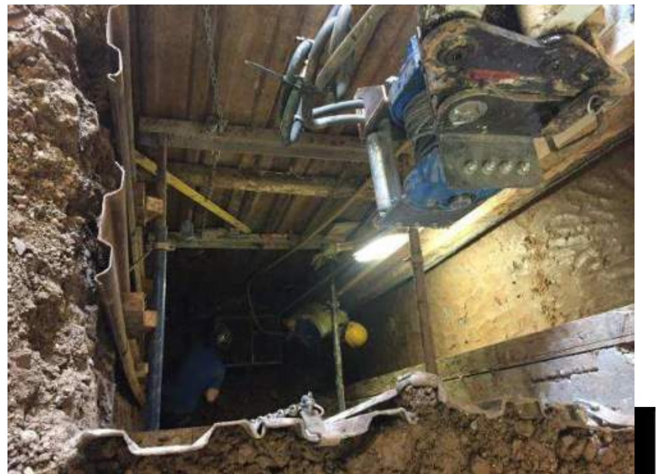
The excavation for, and construction of the lower ground floor level will need to be completed without involving or disturbing the surrounding buildings. The sequence of the works for the construction phase of this project will, ultimately, be prepared by the contractor who will undertake the works, but we expect, and will guide them towards a sequence similar to the following:

- Construction of the reinforced concrete underpinning with toe beneath the front, internal and gable walls of the main house and rear extension starting from the middle of each wall at four or locations following a traditional 1 3 5 2 4 hit and miss sequence,
- Construction of temporary 1m length underpinning under the two internal corridor walls,
- Construction of the reinforced concrete liner wall adjacent to the existing basement wall of no. 7 in a hit and miss sequence,
- Backfill each pin when complete,
- Install new ground floor steel beams,
- Install new ground floor level metal deck and cast concrete,
- Installation of lateral props between the house walls just above existing ground level where required,
- Excavation down to slab formation level,
- Installation of lateral props between the house walls just above proposed basement level,
- Installation of new below ground drainage,
- Formation of reinforced concrete basement slab, including thickened slab at edges,
- Remove basement level props,
- Install needling and vertical props to the two internal corridor walls,
- Install new supporting steel beams under the wall and dry pack,
- Remove needling and vertical props and repair masonry at needle locations,
- Demolish the temporary mass concrete underpins,
- Cast any remaining ground floor slab areas,
- Remove ground floor level props.

Underpinning is done following a hit-&-miss sequence; local props and sheeting will be required to support the excavations. With the conclusion of the perimeter underpins and commencement of excavation works, bracing props will be installed between the walls, and maintained in place until the new lower ground floor slab and liner wall are constructed. Continuity reinforcement between the pins will allow lateral props to be provided at 2-3m c/c rather than to each pin.

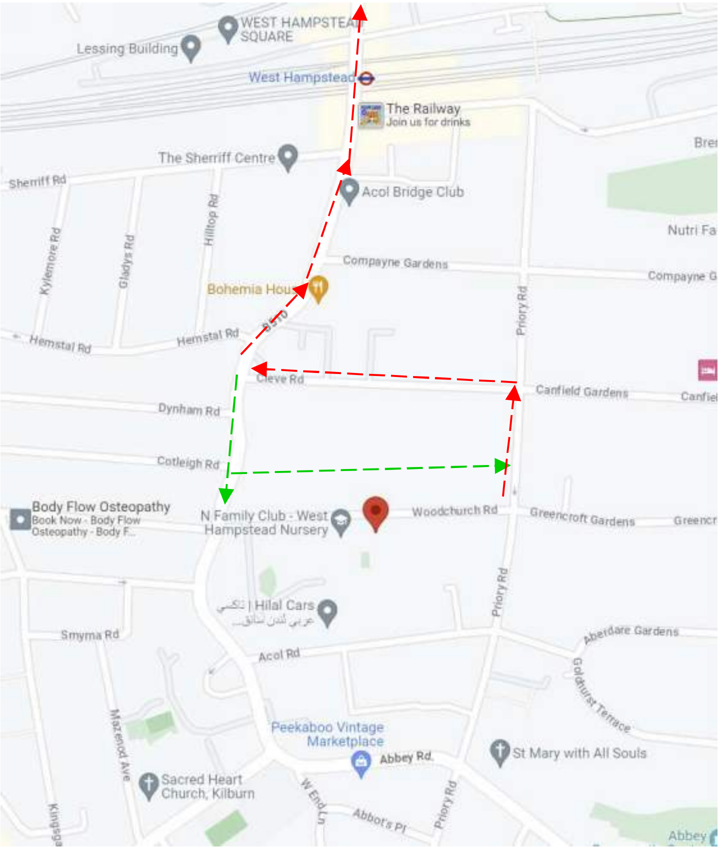
Woodchurch Road is a two-way residential road with parking on both sides but will nevertheless accommodate construction traffic. The site has some but limited space within its boundary for temporary material storage and a Contractor's traffic management plan will be necessary to manage construction traffic and deliveries and storage of construction materials on site. The programme of works will be confirmed once the contractor is appointed.

Temporary storage of arisings



Shored Excavation for an underpin

[redacted]



CONSTRUCTION TRAFFIC ROUTE OUT

CONSTRUCTION TRAFFIC ROUTE IN

10. NOISE & NUISANCE

Construction works generally are a source of noise and nuisance which can affect operatives within the site as well as neighbours and passing members of the public. Demolition and excavation works are particular sources of this potential harm so it will be necessary during these works, at 9 Woodchurch Road, for the contractor to mitigate the extent and impact of noise, dust, traffic and vibration.

Noise: Generated by the mechanical equipment used to excavate for the new basement;
Mitigated by using electrical equipment where possible and mufflers or attenuators on diesel engines or generators, by working only within agreed and designated hours;

Dust: Generated by excavation works and the transfer of arisings from the works area to the disposal skip or wagon;
Mitigated by damping conveyors when in operation, by installing a weatherproof cover over the site, by washing-down vehicle wheels before leaving site;

Traffic: Generated by delivery and removal vehicles travelling to and from site;
Mitigated by establishing a traffic management plan, by identifying and using routes appropriate to the vehicles, by scheduling vehicle movements to avoid peak traffic periods, by ensuring vehicles are low-emission standard;

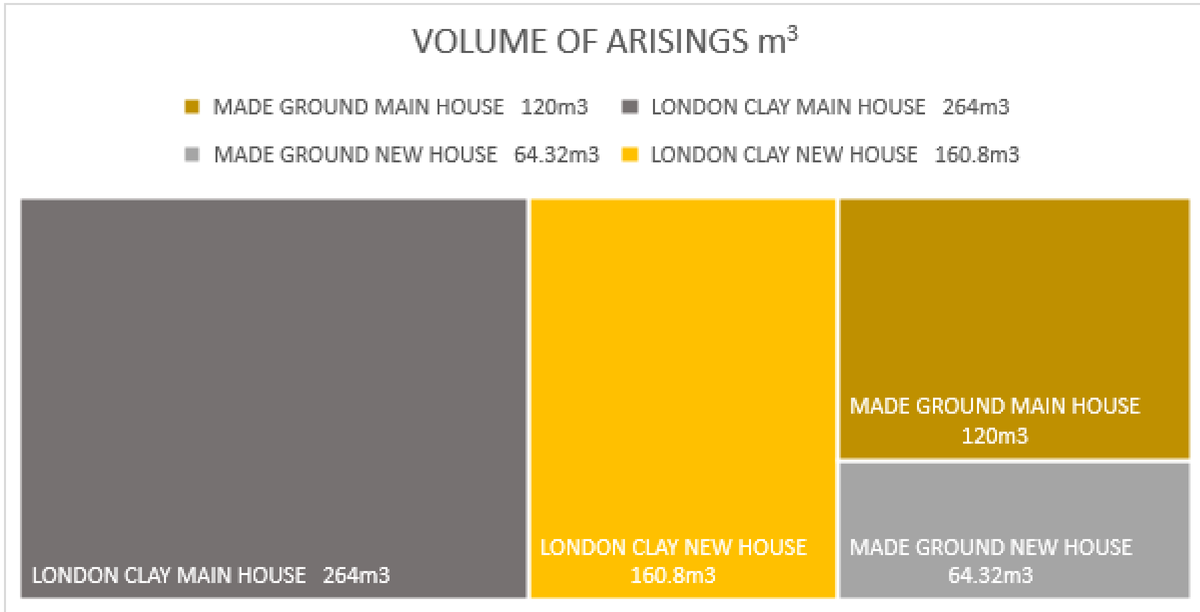
Vibration: Generated by use of heavy breakers for sustained periods and by heavy vehicles or plant;
Mitigated by using light, hand-held and electrical breakers, by avoiding excessively heavy plant.

Protection: Robust hoarding will be erected around the site, front rear and sides, to secure the site from intrusion as well as provide protection to neighbours and passing public from noise, dust and material arisings.

The works will cover around $200m^2$ and excavate to 2.8m to 3.4m over the area, which will generate approximately $609m^3$ of spoil as follows:

Covered Site

Small Excavator used near Boundaries



Amount and type of spoil removed

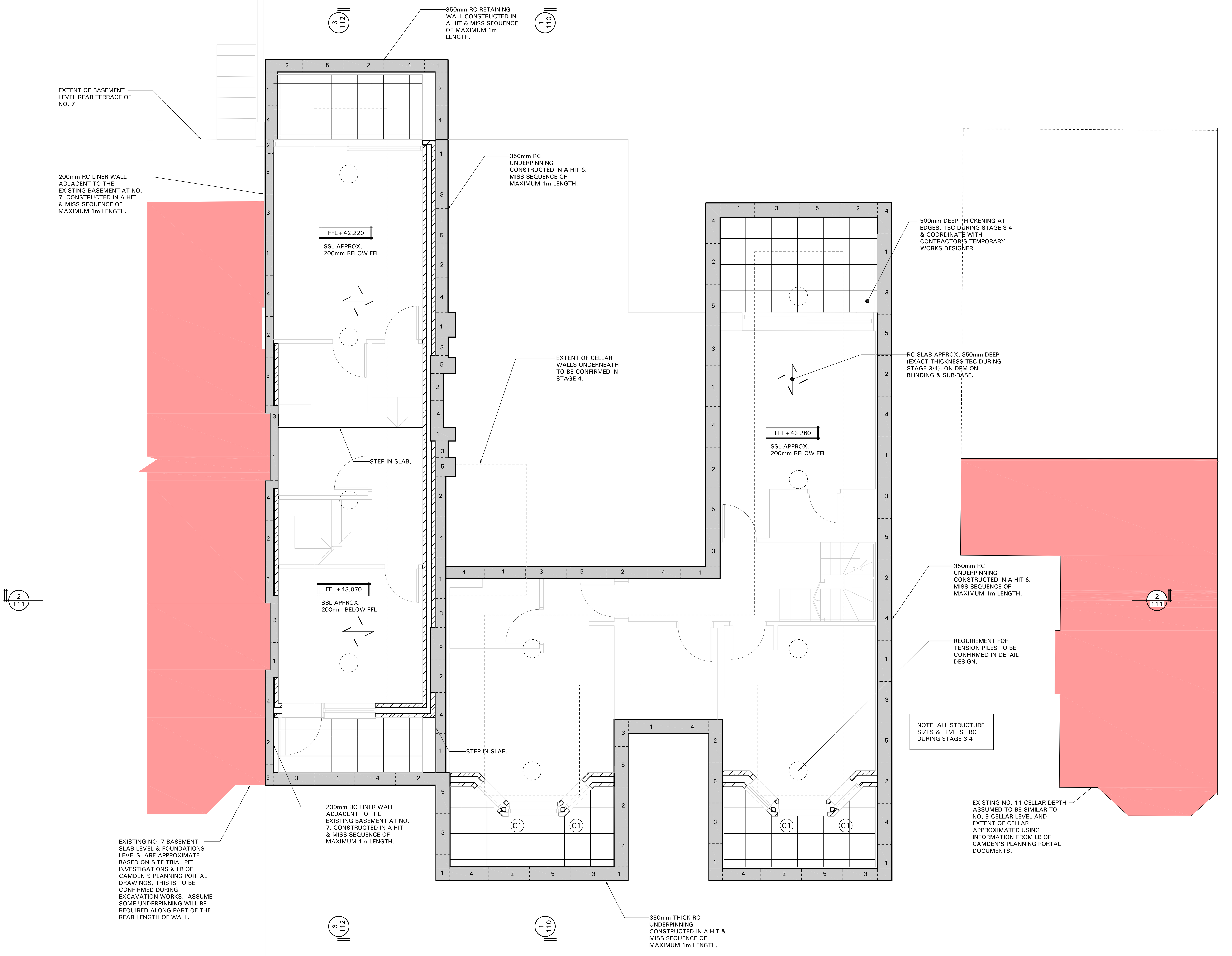
11. CONCLUSIONS

The proposed development at 9 Woodchurch Road can be achieved using standard construction techniques and materials. The lower ground floor level can be constructed using relatively light techniques, in controlled and pre-determined sequences and without the need for a large open excavation before construction can start and consequent extensive temporary works. Where mechanical means are necessary to construct permanent works, these can be of a type that generates low vibrations to which the surrounding buildings have a form and construction that is robust and resistant to.

- As outlined in previous sections, the construction of the lower ground floor level will not affect the integrity of the surrounding building stock, will not disturb underlying hydrogeology or overload the near-surface geology.
- The site is on fairly level ground in any case but, notwithstanding this, the construction techniques and sequences proposed minimises the risk of instability, ground slip and movement.
- There are no critical utilities or infrastructure beneath the site that cannot be relocated easily to accommodate the construction and, the proposed works while it will provide additional habitable space, the increase will not generate great amount of extra load on the local infrastructure.
- The excavation for, and construction of the basement will need to be completed without involving or disturbing the surrounding buildings. Underpinning will commence from the middle of the walls and will be cast in 1m-sections of mass or reinforced concrete.
- The basement works will not impact any known nearby trees.
- By adopting an underpinning technique and following a hit-&-miss sequence, as described above it will be possible to construct the basement in a carefully sequenced and managed process without extensive temporary works.
- Any temporary works needed will be designed by the Contractor to current British Standards or Eurocodes.
- The surrounding roads are wide enough and without tight bends or corners that will hinder or prevent site traffic and will not cause site traffic to hinder or delay local and residential traffic.

Report Prepared by:	Report Approved by:
Natalie Martono	Malcolm Brady (Director)
For MBP	Date: 04/09/23
Issue: P02	Issued on: 04/09/23

APPENDIX A MBP DRAWING SET 10014



LEGEND:

- NEW REINFORCED CONCRETE STRUCTURE
- EXISTING STRUCTURE TO BE RETAINED
- NEW BRICKWORK
- NEW LOADBEARING BLOCKWORK
- B1 — NEW STEEL BEAM & REFERENCE. REFER TO SCHEDULES FOR TYPE AND SIZE
- C1 — COLUMN REFERENCE. SIZE TBC IN STAGE 3-4 DESIGN.
- TWO-WAY SPAN OF RC SLAB. REFER TO LAYOUTS FOR DEPTH. FINAL DEPTH TBC IN STAGE 3-4 DESIGN.
- SPAN OF COMPOSITE FLOOR TBC
- STEP IN LEVEL

ABBREVIATIONS:

SSL	-	STRUCTURAL SLAB LEVEL
FFL	-	FINISHED FLOOR LEVEL
TOF	-	TOP OF FLOOR
TOB	-	TOP OF BEAM
TOPC	-	TOP OF PILECAP
TOS	-	TOP OF STEEL
TOW	-	TOP OF WALL
TOJ	-	TOP OF JOIST
CB	-	COLUMN BELOW
WB	-	WALL BELOW

NOTES:

1. THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS AND ENGINEERS DRAWINGS AND SPECIFICATIONS.
2. FOR SETTING OUT REFER TO ARCHITECT'S DRAWINGS.
3. ALL DIMENSIONS ARE IN MILLIMETRES (mm) UNLESS NOTED OTHERWISE.
4. DO NOT SCALE FROM THE DRAWING OR THE COMPUTER DIGITAL DATA. ONLY FIGURED DIMENSIONS TO BE USED.

Rev	Date	Description	By
P04	04/09/23	EDGE THICKENING WIDEN UNDERNEATH COLUMNS AS COORDINATED WITH GEOTECH ENGINEER.	NM
P03	25/07/23	NOTE ON REQUIREMENT FOR TENSION PILE ADDED (PENDING CONFIRMATION IN FINAL SI REPORT).	NM
P02	21/07/23	EXTENT OF BASEMENT & GROUND FLOOR ALTERATIONS UPDATED TO SUIT LATEST ARCHITECT'S LAYOUT.	NM
P01	14/06/23	PRELIMINARY ISSUE	NM
P01-WIP	01/06/23	WORK IN PROGRESS ISSUE	NM

Project Title
9 WOODCHURCH ROAD LONDON NW6 3PL

Drawing Title
PROPOSED LOWER GROUND FLOOR PLAN FOR PLANNING

Scale @ A1	Date	Drawn	Checked
1:50	31/05/23	SF	NM

Drawing Status: S4 - SUITABLE FOR STAGE APPROVAL
Revision: P04

Project Drawing ID
10014-MBP-XX-B1-DR-S-0099
Project Originator Volume Level Type Role Number

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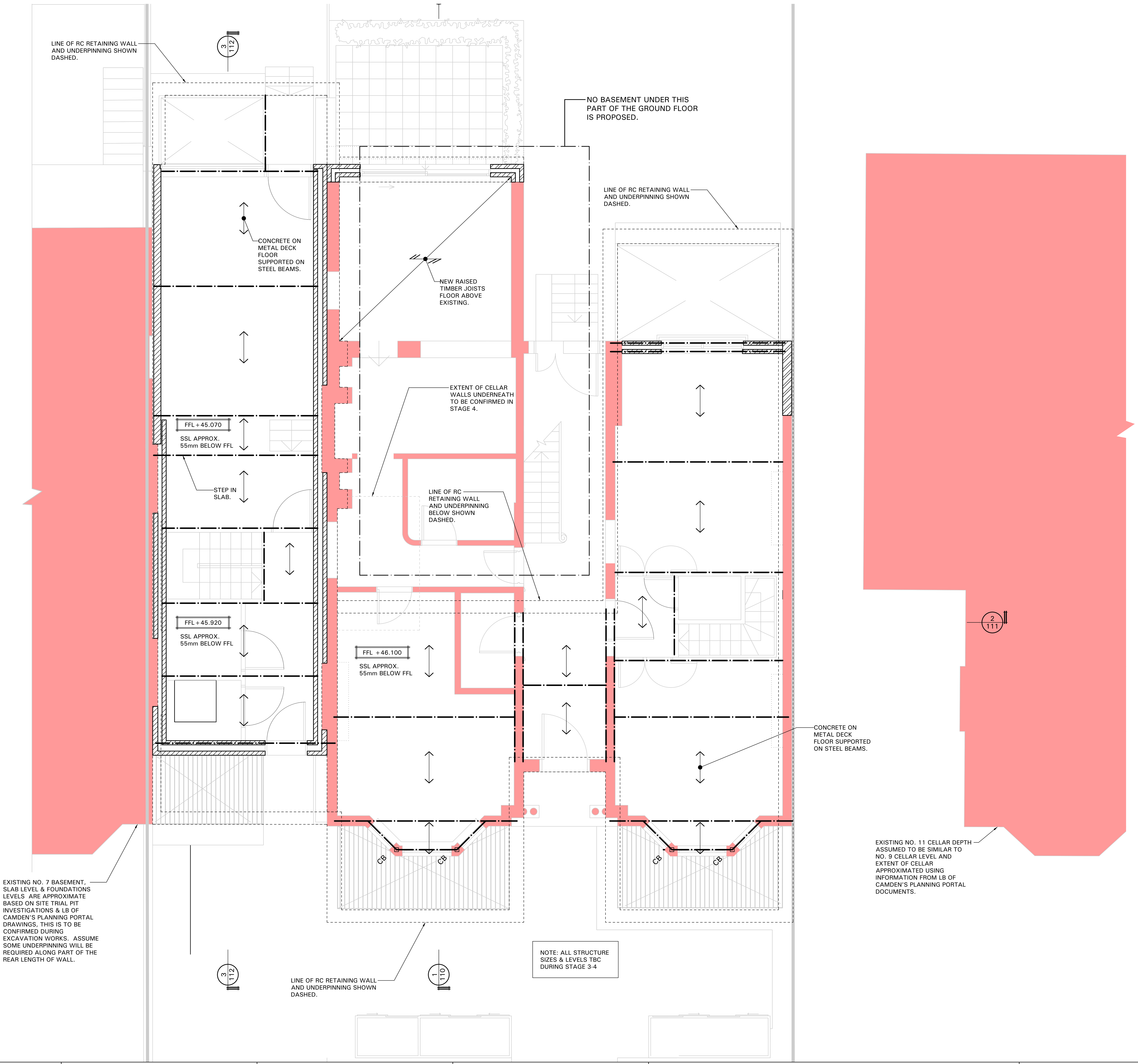

LEGEND:

- NEW REINFORCED CONCRETE STRUCTURE
- EXISTING STRUCTURE TO BE RETAINED
- NEW BRICKWORK
- NEW LOADBEARING BLOCKWORK
- B1 — NEW STEEL BEAM & REFERENCE. REFER TO SCHEDULES FOR TYPE AND SIZE
- C1 — COLUMN REFERENCE. SIZE TBC IN STAGE 3-4 DESIGN.
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- SPAN OF COMPOSITE FLOOR TBC
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WB	-	WALL BELOW

SCALE 1:50 — 0m 1m 2m 3m 4m 5m

NOTES:

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Rev	Date	Description	By
P02	21/07/23	EXTENT OF BASEMENT & GROUND FLOOR ALTERATIONS UPDATED TO SUIT LATEST ARCHITECT'S LAYOUT.	NM
P01	14/06/23	PRELIMINARY ISSUE	NM
P01-WIP	01/06/23	WORK IN PROGRESS ISSUE	NM

Project Title
9 WOODCHURCH ROAD LONDON NW6 3PL

Drawing Title
PROPOSED GROUND FLOOR PLAN FOR PLANNING

Scale @ A1	Date	Drawn	Checked
1:50	31/05/23	SF	NM

Drawing Status: S4 - SUITABLE FOR STAGE APPROVAL
Revision: P02

Project Drawing ID
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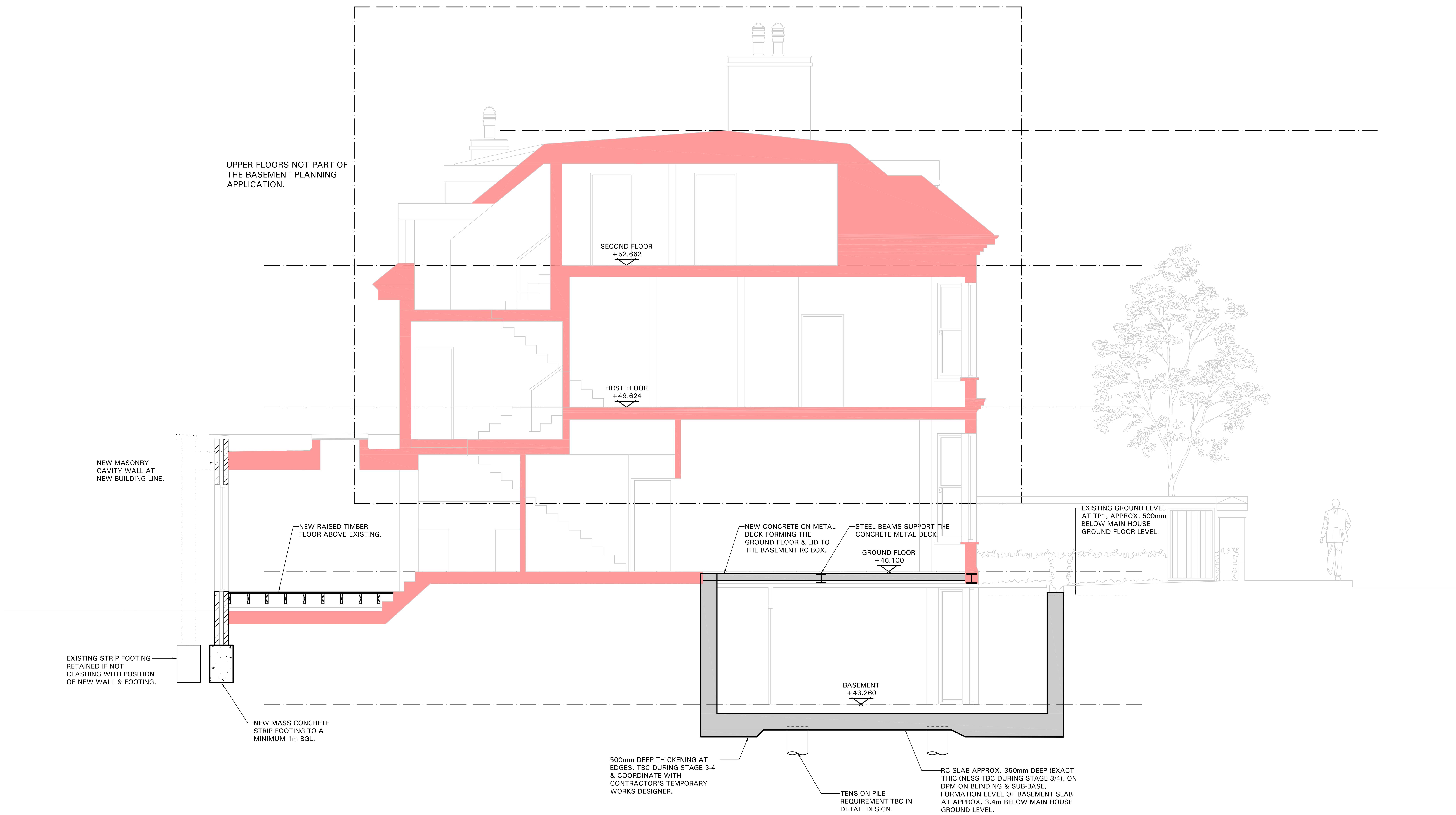

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Rev	Date	Description	By
P04	04/09/23	EDGE THICKENING WIDEN UNDERNEATH COLUMNS AS COORDINATED WITH GEOTECH ENGINEER.	NM
P03	25/07/23	NOTE ON REQUIREMENT FOR TENSION PILE ADDED (PENDING CONFIRMATION IN FINAL SI REPORT).	NM
P02	21/07/23	EXTENT OF BASEMENT & GROUND FLOOR ALTERATIONS UPDATED TO SUIT LATEST ARCHITECT'S LAYOUT.	NM
P01	14/06/23	PRELIMINARY ISSUE	NM
P01-WIP	01/06/23	WORK IN PROGRESS ISSUE	NM

Project Title
9 WOODCHURCH ROAD LONDON NW6 3PL

Drawing Title
PROPOSED CROSS SECTION FOR PLANNING SHEET 1

Scale @ A1	Date	Drawn	Checked
1:50	31/05/23	SF	NM

Drawing Status: S4 - SUITABLE FOR STAGE APPROVAL
Revision: P04

Project Drawing ID
10014-MBP-XX-ZZ-DR-S-0110
Project Originator Volume Level Type Role Number

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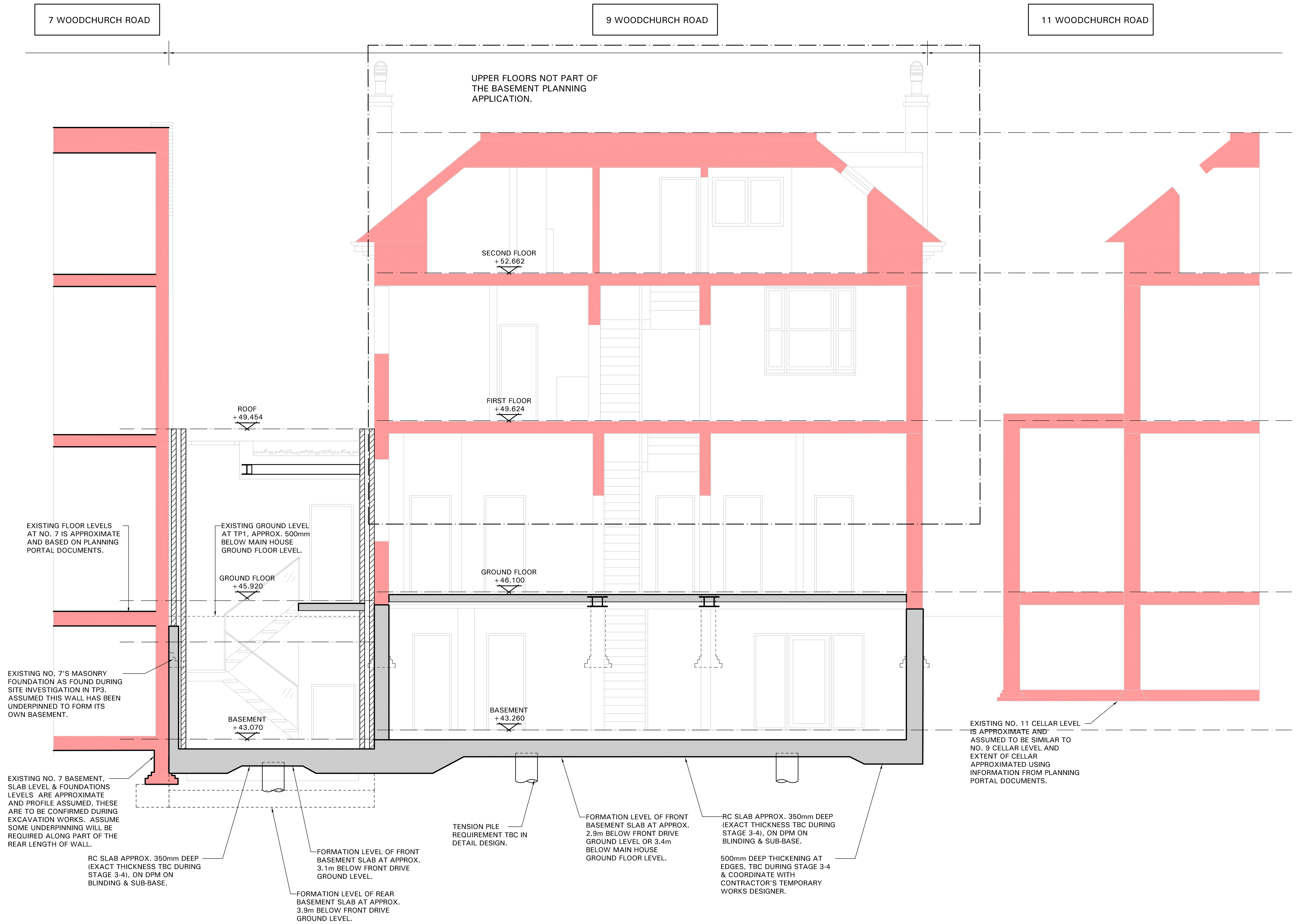

SCALE 1:50 — 0m 1m 2m 3m 4m 5m

NOTES:

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Rev	Date	Description	By
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P03	25/07/23	NOTE ON REQUIREMENT FOR TENSION PILE ADDED (PENDING CONFIRMATION IN FINAL SI REPORT).	NM
P02	21/07/23	EXTENT OF BASEMENT & GROUND FLOOR ALTERATIONS UPDATED TO SUIT LATEST ARCHITECT'S LAYOUT.	NM
P01	14/06/23	PRELIMINARY ISSUE	NM
P01-WIP	01/06/23	WORK IN PROGRESS ISSUE	NM

Project Title

**9 WOODCHURCH ROAD
LONDON
NW6 3PL**

Drawing Title

**PROPOSED CROSS SECTION
FOR PLANNING
SHEET 2**

Scale @ A1	Date	Drawn	Checked
1:50	31/05/23	SF	NM

Drawing Status: **S4 - SUITABLE FOR STAGE APPROVAL**

Revision: **P04**

Project Drawing ID

10014-MBP-XX-ZZ-DR-S-0111

Project | Originator | Volume | Level | Type | Role | Number

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SCALE 1:50 — 0m 1m 2m 3m 4m 5m

NOTES:

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Rev	Date	Description	By
P04	04/09/23	EDGE THICKENING WIDEN UNDERNEATH COLUMNS AS COORDINATED WITH GEOTECH ENGINEER.	NM
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P01	14/06/23	PRELIMINARY ISSUE	NM
P01-WIP	01/06/23	WORK IN PROGRESS ISSUE	NM

Project Title
9 WOODCHURCH ROAD LONDON NW6 3PL

Drawing Title
PROPOSED CROSS SECTION FOR PLANNING SHEET 3

Scale @ A1	Date	Drawn	Checked
1:50	31/05/23	SF	NM

Drawing Status: **S4 - SUITABLE FOR STAGE APPROVAL**
Revision: **P04**

Project Drawing ID
10014-MBP-XX-ZZ-DR-S-0112
Project Originator Volume Level Type Role Number

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APPENDIX B MBP CALCULATION SET 10014

MBP **Michael Barclay Partnership**
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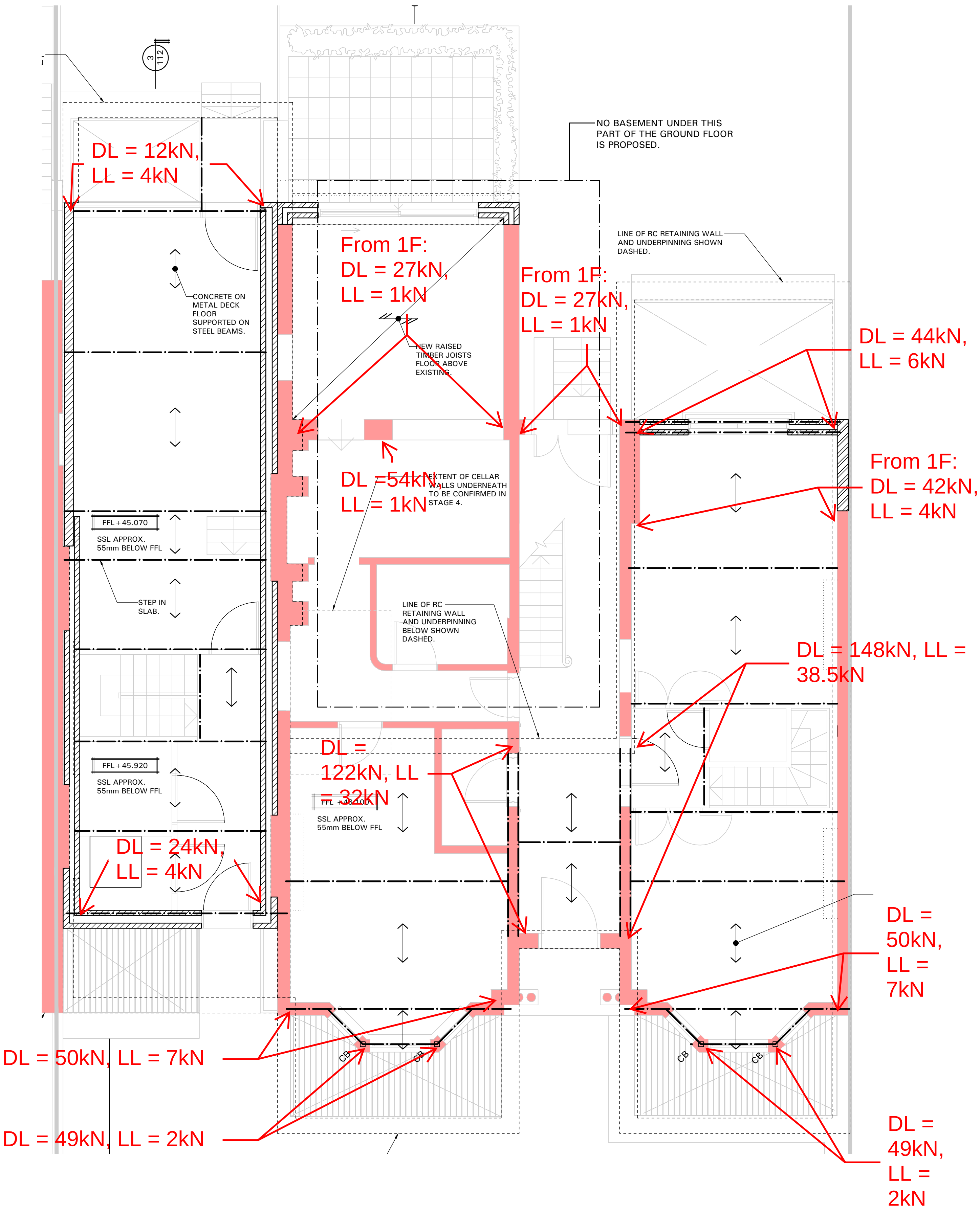
Job Title	Job Number	Sheet Number	Revision
9 WOODCHURCH ROAD LONDON NW6 3PL	10014	1	
Calculation/sketch Title	**Date**	**Author**	**Checked**
REACTION FROM BEAMS ON WALLS	25/07/23	NM	MB



MBP Michael Barclay Partnership consulting engineers	Job Title	Job Number	Sheet Number	Revision
	9 WOODCHURCH ROAD LONDON NW6 3PL	10014	2	P02
1 Lancaster Place, London, WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
T 020 7240 1191	HEAVE ASSESSMENT	25/07/23	NM	MB
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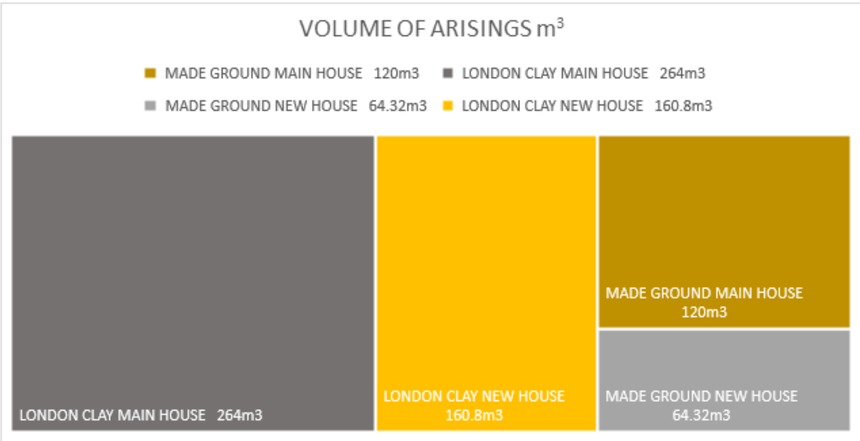
ESTIMATION OF HEAVE

SOIL PROPERTIES

RETAINED SOIL : MADE GROUND = 18 kN/m^3, $\phi = 22^\circ$
SUPERFICIAL CLAY = 20 kN/m^3, $\phi = 22^\circ$
LONDON CLAY = 20 kN/m^3, $\phi = 21^\circ$

Volume of excavation to form the new basement has been estimated as below:



Weight of made ground removed	= 18 kN/m^3 x (120+64.3)m^3	=	3,317 kN
Weight of clay soil removed	= 20 kN/m^3 x (264+160.8)m^3	=	8,496 kN
Total weight of soils removed			= 11,813 kN

Allowing for 50% instantaneous relief (short term heave), the residual long term heave is estimated as:
0.5 x 11,813 kN = 5,907 kN

HYDROSTATIC UPLIFT DUE TO GROUND WATER

The three boreholes were found to be dry during the SI and at subsequent monitoring at 6 weeks after SI. Conservatively, allow for water pressure from water at 1m below ground level.

At the front of the property, the formation depth is at 3.4m, therefore there is 2.4m uplift pressure from water.

At the rear of the property, the formation depth is at 2.8m, therefore there is 1.8m uplift pressure from water.

Uplift due to hydrostatic pressure is estimated at:

10 kN/m^3 x (40 m^2 x 1.8 m)+(160 m^2 x 2.4m) = 4,560kN

CHECK AGAINST FLOTATION

The following calculation notes the dead weight of the building, including the new basement structure, which is then compared with the total uplift force.

Wall Load	DL, kN/m	LL, kN/m	Length, m	DL, kN	LL, kN
(refer to load take down sheet 0400)	25.5	0	8.5	217	0
	~~19~~	~~0~~		0	0
	3	1	5	15	5
	45	5	14	630	70
	37	5	14	518	70
	13	0	2	26	0
	24	0	4.53	109	0
	10	2	2.85	29	6
	10	2	2.65	27	5
	24	0	9.9	238	0
	24	0	10.1	242	0
	95	12	4.77	453	57
	73	12	6.53	477	78
	7	0.1	4	28	0
	41	5	4.3	176	22
	~~33~~	~~5~~		0	0
	~~69~~	~~17~~		0	0
	~~14~~	~~0.2~~		0	0
	48	7	2.85	137	20
	74	13	2.5	185	33
	24	0	1.4	34	0
	26	7	2.85	74	20
	87	16.5	6.53	568	108
	4.5	0.4	9	41	4
	4	1	7	28	7
	3.8	0.3	11	42	3
	8	1	5.5	44	6
	24	0	9.3	223	0
	93	11	11.3	1051	124
				5610	637
Basement Slab Load	DL, kN/m^2	LL, kN/m^2	Area, m^2	DL, kN	LL, kN
350mm slab	10.7	1.5	78	835	117
500mm edge thickening	14.3	1.5	121	1730	182
			TOTAL LOAD	8175	936

The total dead weight of the building > max of heave and hydrostatic pressure
8,175 > 5,907 , but
The total dead weight of the building < heave + hydrostatic pressure
8,175 < 10,467
If a 450mm diameter tension pile is assumed to have a 250kN capacity then 10 number piles will be needed.

Therefore, an allowance for tension piles in planning stage is made. However, this is to be confirmed during the detail design stage.

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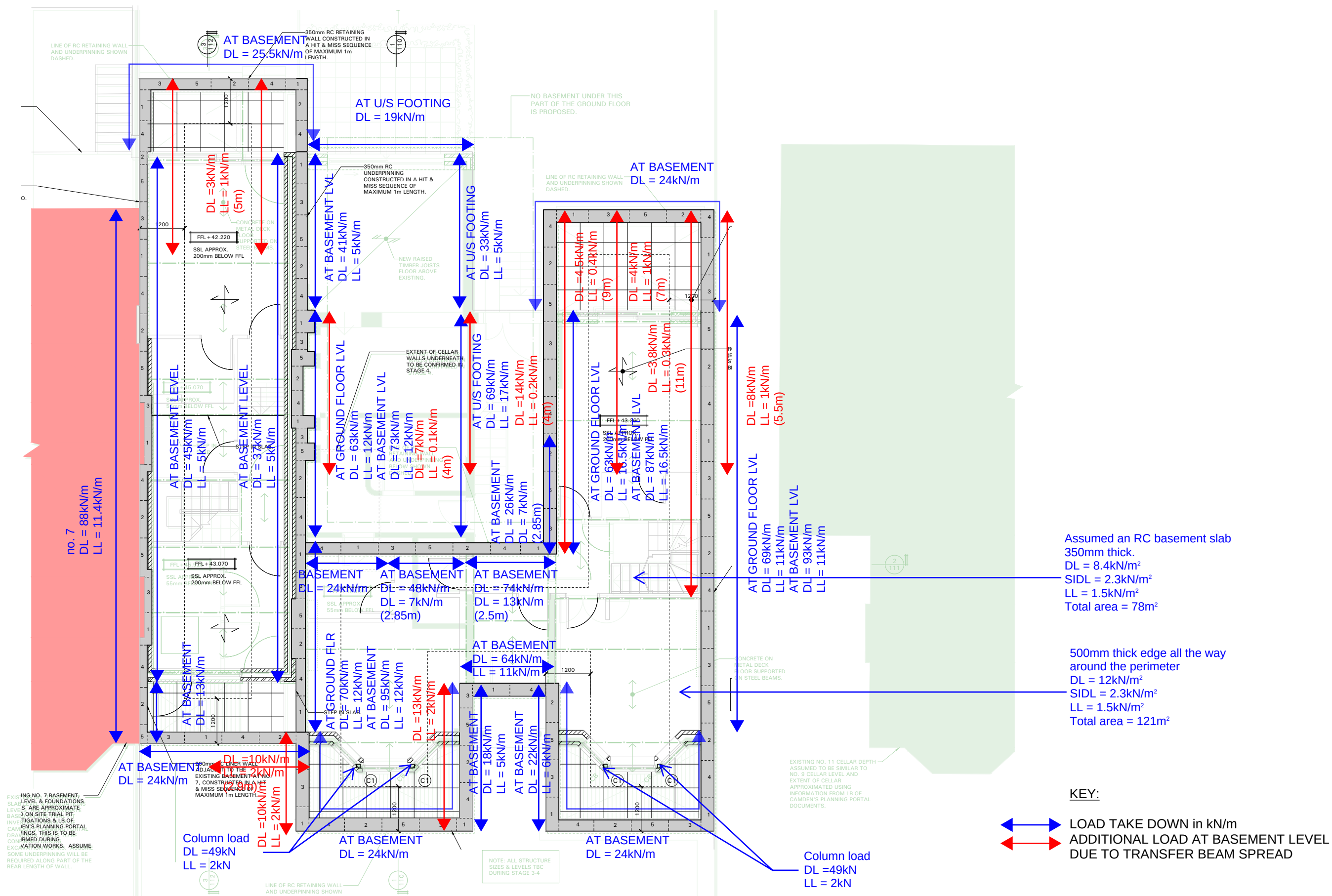
Job Title	Job Number	Sheet Number	Revision
9 WOODCHURCH ROAD LONDON NW6 3PL	10014	4	
Calculation/sketch Title	**Date**	**Author**	**Checked**
HEAVE ASSESSMENT	25/07/23	NM	MB

ESTIMATED BUILDING DEAD & LIVE LOADS AT BASEMENT LEVEL



RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.17

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **3050** mm
Stem thickness	t_{stem} = **350** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1750** mm
Base thickness	t_{base} = **500** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2750** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2050** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

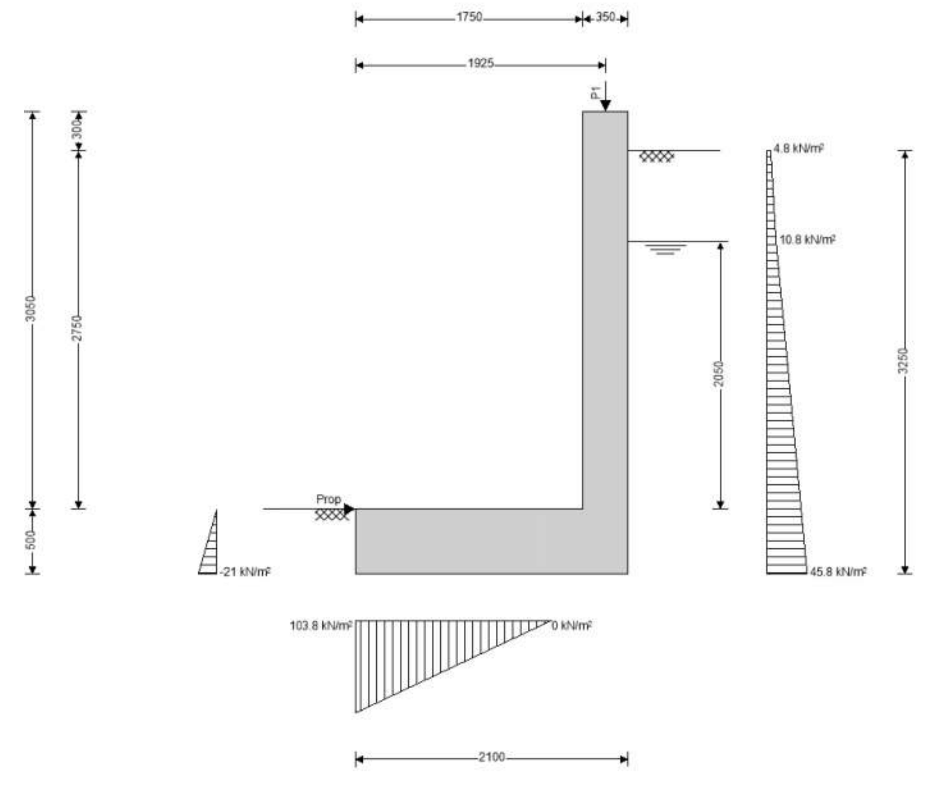
Soil type	Firm clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **150** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1925 mm	P_{G1} = **25.5** kN/m



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{2100}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{2050}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{700}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2100}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3250}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1625}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.068}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1925}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{1.05}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1050}$ mm

Using Coulomb theory

Active pressure coefficient

$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$

Passive pressure coefficient

$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{26.7}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{26.3}$ kN/m

Line loads $F_{P\_v} = P_{G1} =$ **25.5** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} =$ **78.4** kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} =$ **15.5** kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **12.7** kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **31.9** kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **17.4** kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-5.2** kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} =$ **72.3** kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **51.4** kNm/m

Wall base $M_{base} = F_{base} \times x_{base} =$ **27.6** kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-25.2** kNm/m

Line loads $M_P = P_{G1} \times p_1 =$ **49.1** kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} =$ **-10.8** kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} =$ **-27.1** kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-25.4** kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} =$ **39.5** kNm/m

Check bearing pressure

Propping force $F_{prop\_base} = F_{total\_h} =$ **72.3** kN/m

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **504** mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **-546** mm

Loaded length of base $l_{load} = 3 \times \bar{x} =$ **1511** mm

Bearing pressure at toe $q_{toe} = 2 \times F_{total\_v} / l_{load} =$ **103.8** kN/m²

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

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.445**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.17

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

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} =$ **30** N/mm²

Characteristic compressive cube strength $f_{ck,cube} =$ **37** N/mm²

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm² $=$ **38** N/mm²

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm² $\times (f_{ck} / 1$ N/mm²$)^{2/3} =$ **2.9** N/mm²

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.0** N/mm²

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm² $\times (f_{cm} / 10$ N/mm²$)^{0.3} =$ **32837** N/mm²

Partial factor for concrete - Table 2.1N $\gamma_C =$ **1.50**

Compressive strength coefficient - cl.3.1.6(1) $\alpha_{cc} =$ **0.85**

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **17.0** N/mm²

Maximum aggregate size $h_{agg} =$ **20** mm

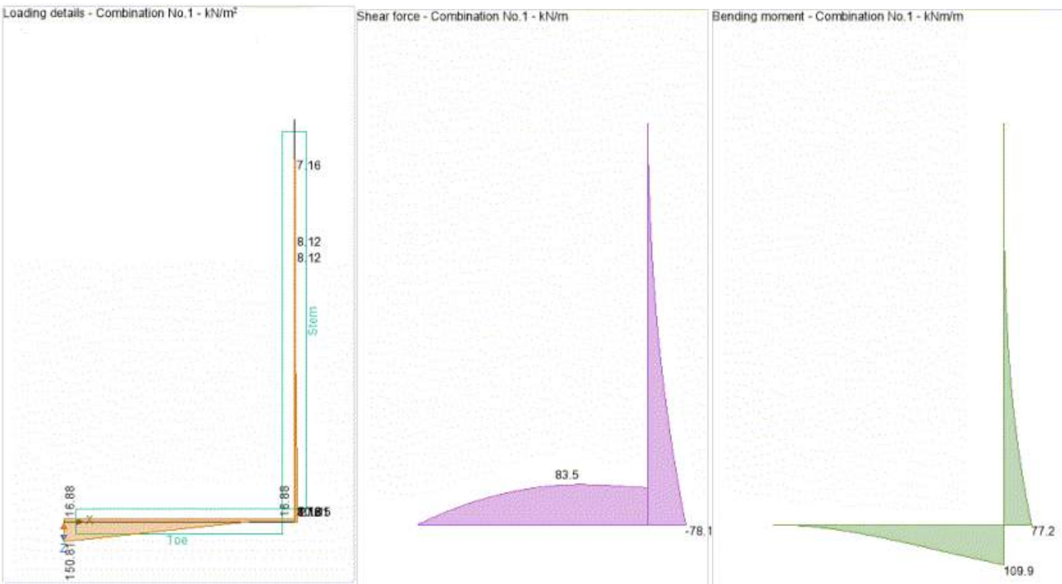
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

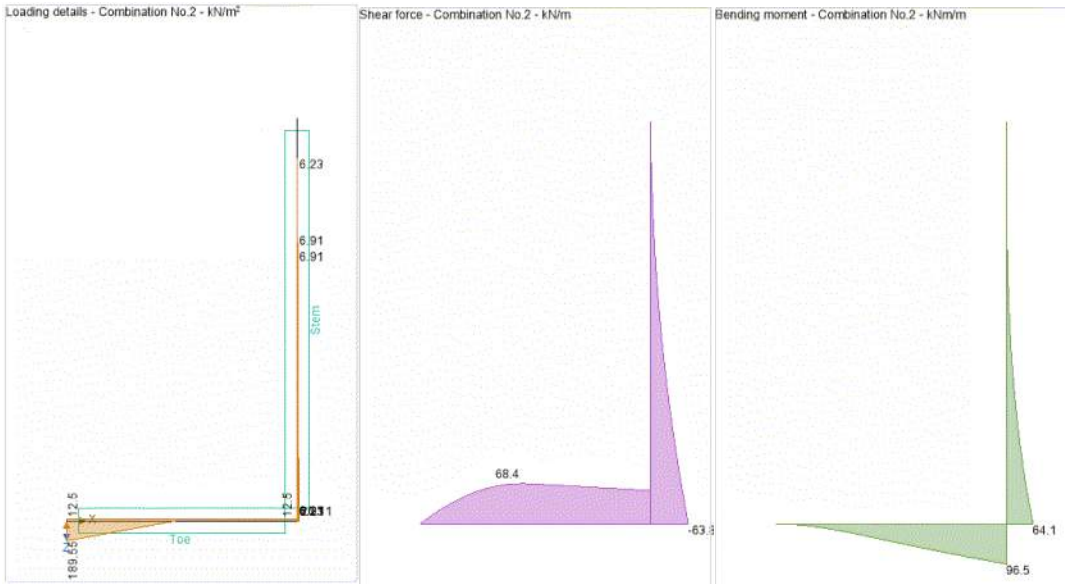
Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **50** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm





Check stem design at base of stem

Depth of section — $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{77.2}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{292}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.030}$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{277}$ mm

Depth of neutral axis — $x = 2.5 \times (d - z) = \mathbf{37}$ mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{640}$ mm²/m

Tension reinforcement provided — 16 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{1005}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{440}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h = \mathbf{14000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.637}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio — $\rho = A_{sr.req} / d = \mathbf{0.002}$

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{0.4}$

Reinforcement factor - exp.7.17 — $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$

Actual span to depth ratio — $h_{stem} / d = \mathbf{10.4}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.6**

Serviceability bending moment M_{sls} = **48** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **172** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **104500** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.010**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **6.091**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **453** mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.234** mm

w_k / w_{max} = **0.779**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **78.1** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.828**

Longitudinal reinforcement ratio $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.003**

$v_{min} = 0.035 \text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.474** N/mm²

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **139.5** kN/m

$V / V_{Rd.c}$ = **0.560**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) s_{sx_max} = **400** mm

Transverse reinforcement provided 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **565** mm²/m

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

Check base design at toe

Depth of section h = **500** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **109.9** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **415** mm

$K = M / (d^2 \times f_{ck})$ = **0.021**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **394** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **52** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **641** mm²/m

Tension reinforcement provided 20 dia.bars @ 200 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1571** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **625** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **20000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.408**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.6**

Serviceability bending moment M_{sls} = **78.6** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **126.9** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **149375** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.011**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **6.091**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **578** mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.22** mm

w_k / w_{max} = **0.734**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **83.5** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.694**

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.423** N/mm²

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **189.6** kN/m

$V / V_{Rd.c}$ = **0.440**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

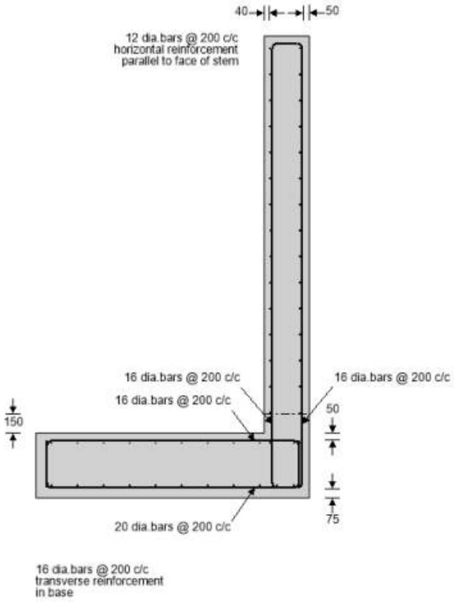
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **314** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) s_{bx_max} = **450** mm

Transverse reinforcement provided 16 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **1005** mm²/m

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



Reinforcement details

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.17

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3000** mm
Prop height	h_{prop} = **3000** mm
Stem thickness	t_{stem} = **350** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **850** mm
Base thickness	t_{base} = **500** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2540** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2000** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

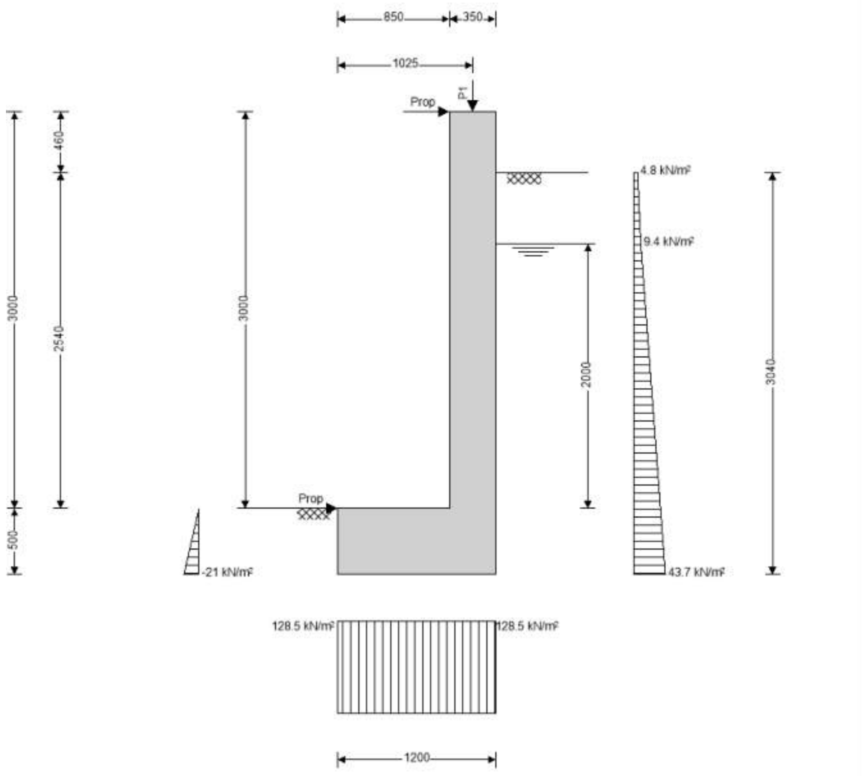
Soil type	Firm clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **150** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1025 mm	P_{G1} = **101** kN/m
	P_{Q1} = **12** kN/m



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{1200}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{2000}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{540}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{1200}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3040}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1520}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.05}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1025}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.6}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{600}$ mm

Using Coulomb theory

Active pressure coefficient

$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$

Passive pressure coefficient

$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{26.3}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{15}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} =$ **113** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} =$ **154.3** kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} =$ **14.5** kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **12.2** kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **30.7** kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **12.8** kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-5.2** kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} =$ **65** kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **26.9** kNm/m

Wall base $M_{base} = F_{base} \times x_{base} =$ **9** kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-22** kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 =$ **115.8** kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} =$ **-10.2** kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} =$ **-25.5** kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-17.8** kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} =$ **76.1** kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) =$ **4.7** kN/m

Propping force to base $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} =$ **60.3** kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) =$ **16.4** kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} =$ **600** mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **0** mm

Loaded length of base $l_{load} = l_{base} =$ **1200** mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **128.5** kN/m²

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **128.5** kN/m²

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.167**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.17

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

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} =$ **30** N/mm²

Characteristic compressive cube strength $f_{ck,cube} =$ **37** N/mm²

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8\ N/mm^2 =$ **38** N/mm²

Mean value of axial tensile strength $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck} / 1\ N/mm^2)^{2/3} =$ **2.9** N/mm²

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.0** N/mm²

Secant modulus of elasticity of concrete $E_{cm} = 22\ kN/mm^2 \times (f_{cm} / 10\ N/mm^2)^{0.3} =$ **32837** N/mm²

Partial factor for concrete - Table 2.1N $\gamma_C =$ **1.50**

Compressive strength coefficient - cl.3.1.6(1) $\alpha_{cc} =$ **0.85**

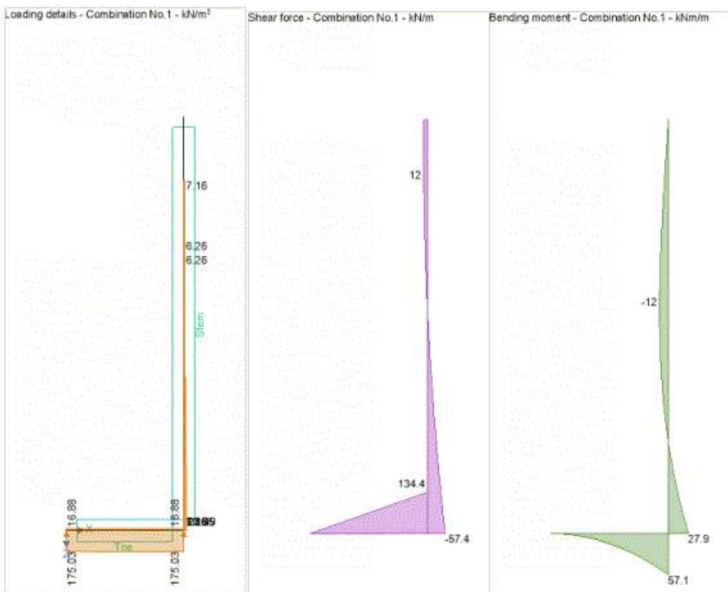
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **17.0** N/mm^2
Maximum aggregate size	$h_{agg} =$ **20** mm
Ultimate strain - Table 3.1	$\varepsilon_{cu2} =$ **0.0035**
Shortening strain - Table 3.1	$\varepsilon_{cu3} =$ **0.0035**
Effective compression zone height factor	$\lambda =$ **0.80**
Effective strength factor	$\eta =$ **1.00**
Bending coefficient k_1	$K_1 =$ **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) =$ **1.00**
Bending coefficient k_3	$K_3 =$**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) =$**1.00**

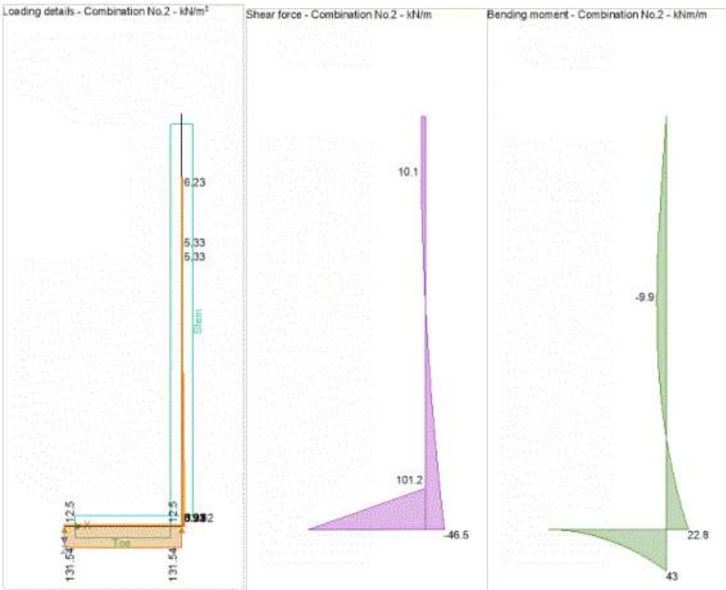
Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} =$ **500** N/mm^2
Modulus of elasticity of reinforcement	$E_s =$ **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S =$ **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S =$ **435** N/mm^2

Cover to reinforcement

Front face of stem	$c_{sf} =$ **40** mm
Rear face of stem	$c_{sr} =$ **50** mm
Top face of base	$c_{bt} =$ **50** mm
Bottom face of base	$c_{bb} =$ **75** mm





Check stem design at 1368 mm

Depth of section — $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{12}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{290}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.005}$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{276}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{36}$ mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{100}$ mm²/m

Tension reinforcement provided — 16 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{1005}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{437}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sfM.max} = 0.04 \times h = \mathbf{14000}$ mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.434}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{}(f_{ck} / 1\ N/mm^2) / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio — $\rho = A_{sfM.req} / d = \mathbf{0.000}$

Required compression reinforcement ratio — $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{40}$

Actual span to depth ratio — $h_{prop} / d = \mathbf{10.3}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **7.5** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **27.2** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **104583** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.010**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **419** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.034** mm
	w_k / w_{max} = **0.114**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **27.9** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **292** mm
	$K = M / (d^2 \times f_{ck})$ = **0.011**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **277** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **37** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **231** mm^2/m
Tension reinforcement provided	16 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1005** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **440** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.437**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d$ = **0.001**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{40}$
Actual span to depth ratio	$h_{prop} / d = \mathbf{10.3}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment	$M_{sls} = \mathbf{18}$ kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{64.7}$ N/mm^2
Load duration	Long term
Load duration factor	$k_t = \mathbf{0.4}$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{104500}$ mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.010}$
Modular ratio	$\alpha_e = E_s / E_{cm} = \mathbf{6.091}$
Bond property coefficient	$k_1 = \mathbf{0.8}$
Strain distribution coefficient	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{453}$ mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.088}$ mm
	$w_k / w_{max} = \mathbf{0.293}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{57.4}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{}(200 \text{ mm} / d), 2) = \mathbf{1.828}$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.003}$
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.474}$ N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} = \mathbf{139.5}$ kN/m
	$V / V_{Rd.c} = \mathbf{0.412}$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	$h = \mathbf{350}$ mm

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{12}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{}(200 \text{ mm} / d), 2) = \mathbf{1.828}$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02) = \mathbf{0.003}$
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.474}$ N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} = \mathbf{139.5}$ kN/m

$V / V_{Rd.c}$ = **0.086**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) s_{sx_max} = **400** mm

Transverse reinforcement provided 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **565** mm²/m

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

Check base design at toe

Depth of section h = **500** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **57.1** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **417** mm

$K = M / (d^2 \times f_{ck})$ = **0.011**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C) \times (1 - \lambda \times (\delta - K_1)/(2 \times K_2)) \times (\lambda \times (\delta - K_1)/(2 \times K_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **396** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **52** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **332** mm²/m

Tension reinforcement provided 16 dia.bars @ 200 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1005** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **628** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **20000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.625**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.6**

Serviceability bending moment M_{sls} = **41.9** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **105.3** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **149292** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.007**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **6.091**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **659** mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.208** mm

w_k / w_{max} = **0.694**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **134.4** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **1.693**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02) =$ **0.002**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.422** N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **176** kN/m

$V / V_{Rd.c}$ = **0.764**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

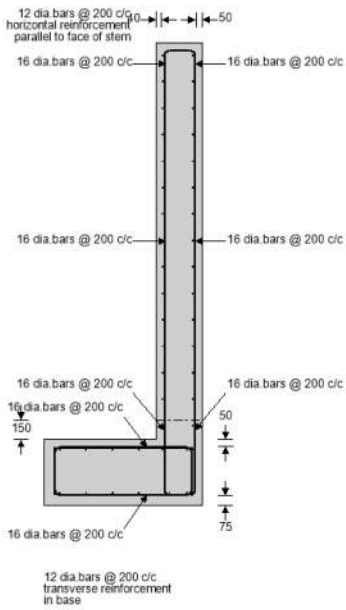
Minimum area of reinforcement – cl.9.3.1.1(2) — $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **201** mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) — s_{bx_max} = **450** mm

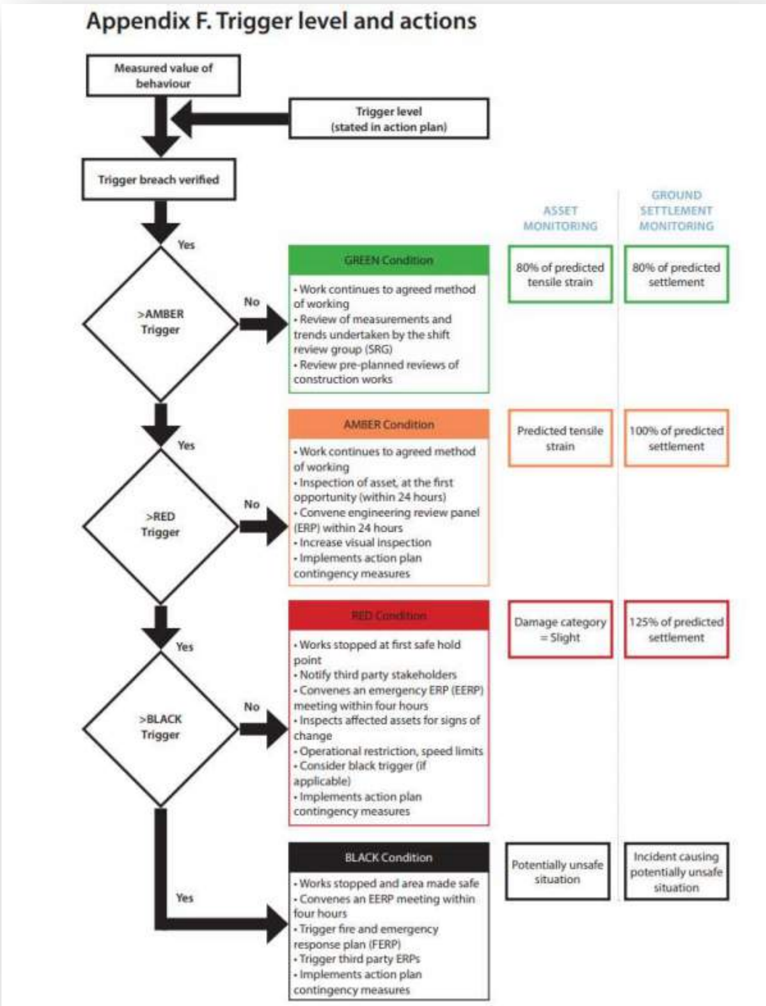
Transverse reinforcement provided — 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **565** mm^2/m

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



Reinforcement details



APPENDIX C PROCEDURE FOR MONITORING ADJACENT BUILDINGS

The contractor will monitor the adjacent structures and party walls for movements throughout the principal demonstration & construction works and, in the event of any movements exceeding the agreed target levels the method of works will be reviewed and altered as necessary.

- The proposed monitoring points will be agreed with the contractor
- The Green/Amber trigger level will be 5mm
- The Amber/Red trigger level will be 10mm

The monitoring regime and frequency proposed is:

Activity	Frequency of monitoring
Site set up	Bi-Weekly
Demolition & Excavation	Weekly
Underpinning & Ground Works	Weekly
Principal Construction Works	Bi-Weekly

Target monitoring will monitor the party walls and front and rear elevations with an accuracy of +/- 2mm. The results of the monitoring are to be recorded and issued by email to the project engineer, CA and engineers for the adjoining properties, on the day that the results are taken. The results are to be presented both in table and graphical form with the graphs for each point plotting the readings taken against time. The following actions will be taken if the trigger levels are exceeded:

Trigger Level	Action
Green/Amber	Immediately notify the engineers. Increase frequency of monitoring to a daily basis.
Amber/Red	Contractor to stop all works and immediately notify the engineers. Contractor and project engineer to put forward proposals, such as additional propping, to limit further movement to an acceptable level.

APPENDIX D PROCEDURES FOR CONTROL OF NOISE, DUST & NUISANCE

To control the disturbance due to noise and vibrations, all works on site will be restricted to the hours of Monday to Friday 8am to 6pm, Saturdays 8am to 1pm. Works that create excessive noise and/or vibration are prohibited, as are any works on Sundays and the bank holidays. The contractor employed to undertake the work will be a member of the Considerate Constructor Scheme. As the site does not appear in the designated neighbourhood areas of London, the basement developers need to consult with the neighbours affected by basement development. Where affected neighbours would like no noisy construction work to take place on Saturday developers should agree to this as part of their construction management plan.

Appropriate measures will be taken to keep dust pollution to a minimum. These measures are compliant with Camden Planning Guidance – Basements dated 2021. Such measures will include the use of water to suppress dust and soil being excavated from basement level, covers for conveyors and skips, and barriers installed around dusty activities that are undertaken externally.

All work will be carried out in accordance with BS 5228-1:2009 and BS 5228-2:2009. All works will employ Best Practicable Means as defined by section 72 of the Control of Pollution Act 1972 to minimise the effects of noise and vibration. All means of managing and reducing noise and vibration which can be practicably applied at reasonable cost will be implemented.

The following measures will be taken:

- Consultation/ communication with neighbours/affected others prior to the start of the works.
- Use only of modern, quiet and well-maintained equipment, all of which will comply with the EC Directives and UK regulations set out in BS 5228-1:2009
- Use of electrically powered hand tools rather than air powered tools and a compressor will be used for to the minimum extent practicable
- Avoidance of unnecessary noise (such as engines idling between operations or excessive engine revving, no radios, no shouting)
- Use of screws and drills rather than nails for fixing hoarding.
- Careful handling of materials, so no dropping off materials from an excessive height (no more than 2m) into skip etc.
- Ensuring that the conveyor is well maintained with rollers in good working order and well oiled.
- Collection /delivery times will be as given in the CTMP

Collection/delivery vehicles will not loiter/wait in the area before the allowed times

- No site run-off of water or mud until the water has been left to settle and is free from particles
- During Demolition:
- Special Care to ensure the site is closed-over
- Dust suppression with water if necessary if needed (recommended)
- Cutting equipment to use water suppressant or local extraction & ventilation

If measures to control dust are unsuccessful works will be stopped and alternative methods proposed and implemented

A detailed CTMP will be prepared by the contractor undertaking the works

APPENDIX E BIA CHECKLIST USING THE LONDON BOROUGH OF CAMDEN PLANNING GUIDANCE – BASEMENTS JAN 2021 AS REFERENCE

Screening – the screening process (land stability, groundwater flow, surface flow and flooding) is to determine whether there is any need for a full BIA	BIA & GMA included as separate document by others
Scoping – the identification of the potential impact of the proposed scheme this is done through the geological, hydrogeological and hydrological study	Included
Site investigation and study – desk study, field investigation, monitoring, reporting and interpretation ground movement potential impact on neighbouring properties, if there is a risk of subsidence this should be described using the Burland Scale	Included
Impact Assessment – evaluating the direct and indirect implications including Flood risk Assessment, Landscaping, watercourses, Historical Ground information through OS Maps, identification of Aquifers,	Included
Building Regulations – the submission of building regulations is required with the full details of works planned, full site investigation and Structural Engineers report on the investigation and development proposals	Next phase – detail design
Detailed site-specific analysis of hydrological and geotechnical local ground conditions	Considered
Analysis of how the excavation of the basement may impact on the water table and any ground water flow, and whether perched water is present	Considered
Details of how flood risk, including risk from groundwater and surface water flooding has been addressed in the design, including details of any proposed mitigation measures	Considered
Details of measures proposed to mitigate any risks in relation to land instability	Considered
A comprehensive non- technical summary document of the assessments	Included
Identify the location of the development in relation to an aquifer or a water course	Included
Impact on flooding and drainage including measures to reduce the risk of flooding to the proposed basement and neighbouring properties	Considered
Appropriate basement construction methods to maintain structural stability of the statutory listed host building and neighbouring statutory listed properties	Considered
Details of noise, disruption and vibrations to neighbouring properties would be minimised during the construction process	Considered
Programme duration	Considered
Construction vehicles' routing and movements, The number and types of construction vehicles, Site access and egress arrangements	Considered

APPENDIX F GEA'S DESK STUDY, GROUND INVESTIGATION, BASEMENT IMOPACT ASSESSMENT & GROUND MOVEMENT ANALYSIS (SEPARATE DOCUMENT)