j. Structural Engineer's report by Mint Structures

STRUCTURAL ENGINEER'S REPORT AND SUGGESTED BASEMENT CONSTRUCTION METHODOLOGY

PRODUCED AS PART OF A BASEMENT IMPACT ASSESSMENT IN SUPPORT OF AN APPLICATION FOR PLANNING

PROJECT REF:	M20202/LS
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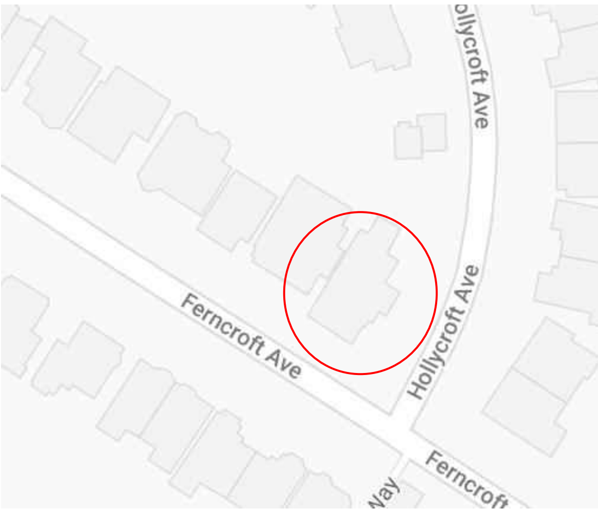
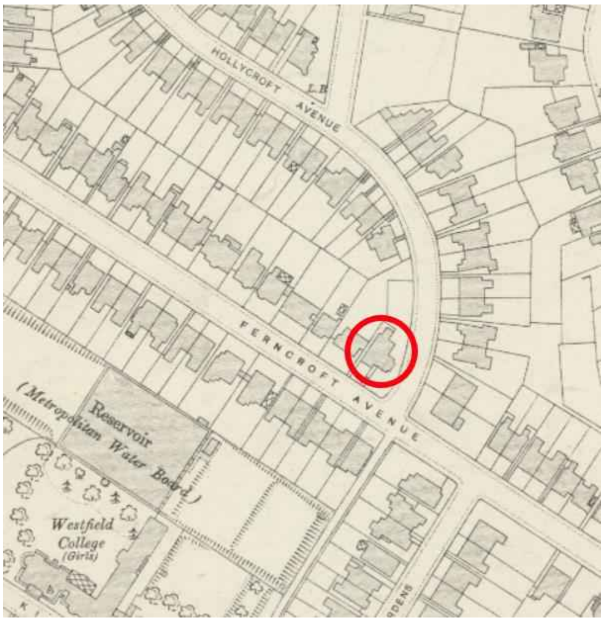
TABLE OF CONTENTS

TABLE OF CONTENTS		2
1.	OVERVIEW	3
2	STRUCTURAL ASSESSMENT - DESK STUDY	6
3.	BASEMENT CONSTRUCTION	14
4	SUGGESTED STRUCTURAL MONITORING PLAN	17
5.	SUGGESTED METHOD STATEMENTS	19
6	CONCLUSION	23
7	REFERENCES	24
8	APPENDICES	25

1. OVERVIEW	
Preliminaries	**ARCHITECT:** 4M GROUP. *Disclaimer:* *The primary details contained within this document are generated from drawings produced by 4M GROUP.* *This document is intended for the exclusive usage of the client /s listed above, the report remains the property of MiNT Structures and must not be reproduced in full or in part or used by any third party without prior written consent. This report is not intended as an engineering design package and should not be relied on solely for any construction processes and should be read in conjunction with relevant Structural Engineering detailed designs and Architectural packages.* Mint Structures have been instructed by the client to provide advice with regard to the impact of the proposed work from a structural perspective. The following document follows guidelines set out within Camden Council's basement planning documentation; Camden Planning Guidance 4, and DP27. Basement and Lightwells. The Basement Impact Assessment has been produced jointly between MINT Structures and Geotechnical & Environmental Associates (GEA) to ensure correctly qualified parties produce the varied constituent parts. The aim in producing this report is to provide a suggested method of construction for the proposed development that will reduce impact on neighbouring structures and local infrastructure as far as is practicably possible.
Scope of document	The content of this planning support document should be read in conjunction with suggested temporary works drawings set B1 – B6 and the reports produced by GEA as part of the Basement Impact Assessment, as well as any other relevant drawings/ details by other parties. The purpose of this report is to assess and comment upon the impact of the proposed development on neighbouring structures, and provide an indicative method statement and suggested construction sequence. These are intended to show a safe and practical way that the elements of temporary works could be installed and permanent works activities could be carried out. It should be noted that all temporary works drawings are indicative only and are not intended as detailed construction drawings, therefore all specific construction details should be provided by the structural engineer or other relevant parties at the detailed design stage. Construction sequences are illustrative only and should not be relied upon to provide specific construction arrangements or any dimensions; these should be specified by others and confirmed on site.
Summary of Proposed Works	Existing structure: The subject property is located on a site of approximately 0.086 Hectares, a substantial late 19th/ early 20th century four-storey (including existing lower ground floor) detached house, situated on a corner plot at the junction between Ferncroft Avenue and Hollycroft Avenue, in the London Borough of Camden. The house is of traditional construction, and over its life has undergone alterations to the internal layouts carried out by previous occupants. However the main building envelope remains largely unchanged from that of the originally constructed building. Proposed works: 1. Lowering of existing lower ground floor level & extension of the lower ground floor into the front and rear of the site. 2. Mass excavation at the rear of the property to form a single storey basement extension, including the installation of a lower ground level swimming pool. 3. Rebuilding of part of the single storey extension to the rear. 4. Support of superstructure over to allow RC basement wall construction and new basement slab installation.

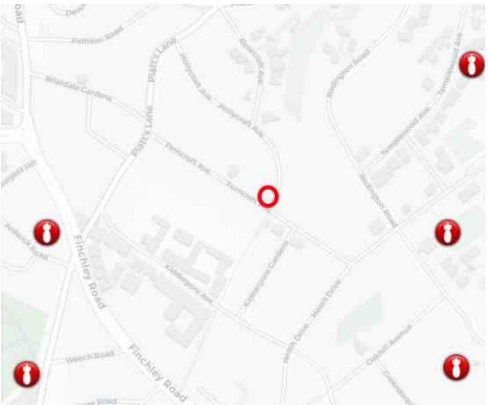
Assumptions made at time of writing	• External walls are solid masonry, supported on either traditional corbelled or mass concrete footings. • Internal walls at ground and lower ground floor level are solid masonry. • Ground & upper floors are of suspended timber construction (barring existing rear extension). • Property currently remains mainly unchanged structurally from its original form with any changes/repairs being in line with a property of its age.
Fundamental Construction Areas / Structural Methodology	• Reinforced concrete (RC) basement walls to be formed in one phase enabling the full required basement depth to be achieved to the majority of the property. • Where a swimming pool is to be installed at lower ground floor level this will require phased underpinning formed in two "hits". • Formation of basement and ground floor slabs to form a basement box capable of permanently resisting earth, water and surcharging pressures. • Rows of horizontal props/shoring will be required to all full height underpins spanning or raking across to suitable propping positions or thrust blocks. Props should remain in place until the basement and any ground floor slabs have been fully constructed and have sufficiently cured providing permanent lateral restraint to the new RC retaining walls. • A monitoring regime should be agreed with all relevant parties to ensure any movements are recorded frequently and managed to keep them with acceptable limits. • All underpins and retained earth, require temporary works and shuttering during excavation and casting until slabs are cast.
Site Location	 *Fig.1* - Site map (Image from Google Maps, Copyright 2021).

General Comments	• Buildings of the age of the property in question have often reached equilibrium with their surroundings. The superstructure slowly deforms with time during its life to accommodate any minor settlements and therefore some work is likely to have been carried out in the past and additional repairs may be necessary as a result of the proposed works. • Any modifications to the existing property should be investigated with local opening up works to assess their potential impact on the proposed scheme. . The contractor is responsible for the design and correct installation of all temporary works required to safely install the proposed basement and any other affiliated works. The contractor is to ensure that all excavations, any new structure and any neighbouring structures are adequately supported for the full duration of the works.

2 STRUCTURAL ASSESSMENT – DESK STUDY	
Site History	With reference to Figure 2 and Figure 3 the site was constructed between 1870 & 1915, with the style of construction putting the construction most likely between 1890-1915. The house was built as part of a larger development of the area generally close to the former site of the large mansion house, Kidderpore Hall. The houses in the immediate area were mainly constructed within a similar time period creating an urbanised area to the west of Hampstead. Prior to construction the surrounding area, as can be seen in the figure below, mainly comprised of agricultural land.  Fig.2 – 1870 OS map excerpt.  Fig.3 – 1915 OS map excerpt.

UXO Risk

Following a web based UXO search, there is no record of WW2 bombing in the immediate vicinity; see Figure 4 for results from a Bombsite.org web search. Further to this a bomb risk map was also consulted, which further indicated a low/moderate risk from UXO. From our research of these results it is considered unlikely that any structural implications associated with direct or indirect bomb detonations will be a risk at the subject property, please also see GEA report for further information on their commissioned preliminary UXO assessment.



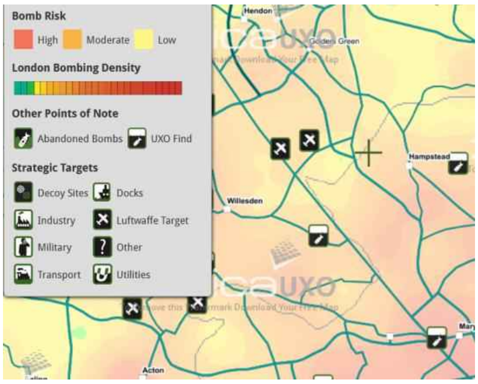


Fig.4 & 5 – UXO map excerpts (bombsite.org, 2021; zeticauxo.com, 2021).

Geological information

The 1:50000 Geological Survey of Great Britain (England and Wales) GeoIndex covering the area indicates the site is underlain by the Claygate Formation with a deep London Clay layer below and a deposit of Bagshot Formation to the Northeast of the site.

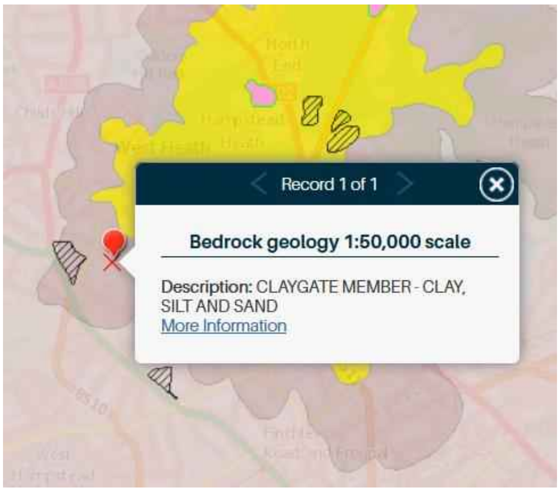


Fig.6 – Taken from BGS.ac.uk (BGS, 2021).

- The location of the site is positioned over Claygate strata and although there is a degree of certainty with soil type, a site investigation was deemed necessary to ascertain the thickness of the Claygate member above the assumed deeper London Clay layer.
- Clayate soil types are generally formed from clays with lenses of sand and silts, having a typical thickness in London of 16m. The location of the site, however, close to the boundary with pure London Clay, suggested the actual thickness would likely be far thinner than this average, as confirmed in the subsequent soil investigation.
- The deeper London Clay formation typically comprises clay, silt and sand with occasional gypsum crystals and claystone at depth. At the site location the London Clay is likely to be approximately 50 to 100+m thick.
- Deeper bedrock stratum is beyond the scope of this report.

Site investigation data

Following the site investigation the site was confirmed to be underlain by a Claygate member over a deeper London Clay (see the appended Borehole logs). The site investigation included eight trial pits and three boreholes (location plan appended to this report) with ground water being encountered at two levels across the site.

The results from the SI showed the site comprised Made Ground extending to a depth between depths of 0.5m -1.3m in the boreholes; below this a Claygate Member was encountered consisting of firm becoming stiff silty sandy clay with lenses of clayey sand to depths between 6.0m-7.0m; Below the Claygate, London Clay was encountered which consisted of stiff becoming very stiff clays to the full depth for Borehole 1. A slow Groundwater inflow was recorded at a depth of 6.0m within the deepest borehole.

The results of the site investigation are summarized typically below. See Appendix 2 for records of boreholes, and GEA report for further detail.

Strata	**Depth to top of strata (mbgl)**	**Depth to base of strata (mbgl)**
Made Ground	0.00	0.5-1.30
Claygate Member	0.5-1.30	6.0-7.0
Clay Member	6.0-7.0	(Continuing)

It should also be noted that isolated pockets of perched groundwater may be present within material of low permeability found at shallow depths (especially within bands of Made Ground). It is recommended that the water levels in standpipe monitoring points are periodically measured immediately prior to, and during construction. This will help to ensure correct measures are taken if water is likely to be encountered during excavation.

For further information on the soil investigation please see the accompanying reports produced by GEA addressing Geology and Hydrogeology in more detail, as well as their interpretative SI report.

Topography

The site slopes Northward from the front to the rear, with the rear garden being 4.0metres above the front of the property. The site is mainly level having been terraced at the front and rear to form a level ground floor/garden area. Also see fig 7 below and GEA reports for greater topographical information.

Fig.7 – Showing location of site inside 0-7° slope areas Figure 16 from Camden Geological, Hydrogeological and Hydrological Study.

Chemical attack on buried concrete

The results of the chemical analysis carried out as part of the site investigation indicated low levels of contaminants, apart from an elevated sample of sulphide in one trial pit. Therefore we have assumed basic precautions based on previous experience.
Within the Claygate layer there is also a good chance that segregations of gypsum will be present which may attack buried concrete. As a result of this, and the discovery of elevated sulphide, we would recommend that buried concrete is designed in accordance with full Class DS-1 conditions.

Hydrology and drainage

Rainfall and run-off

The rainfall in the local area averages around 611mm which is a significantly lower level than the national average which sits at approximately 900mm (See appendix 1 for data extracted from Regional Climate of the British Isles, Mayes (1997)).
Evapotranspiration is typically 332mm/year (see Fig.8 below) resulting in 279mm/year net hydrologically effective rainfall available for infiltration into the ground or to act as surface run-off.

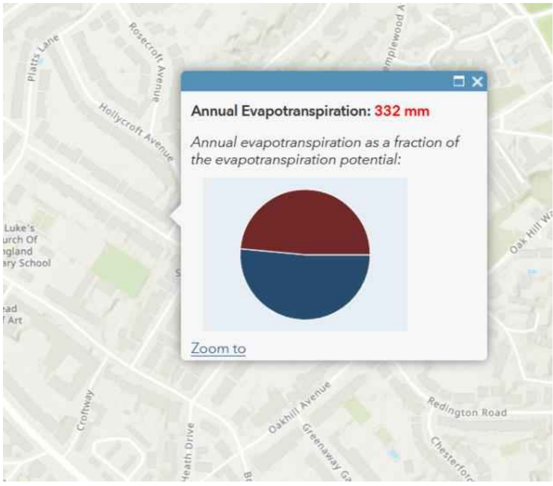


Fig.8 – Average annual evapotranspiration measured by NASA's Moderate Resolution Imaging Spectroradiometer (MODIS) -NTSG, University of Montana.

The area in the immediate vicinity around the site is highly developed and covered with a large proportion of hardstanding. Therefore most of the rainfall in the area will run-off hard surface areas and be collected by the local sewers.

Hydrology

The Lost Rivers of London map indicates that the site is located close to a tributary for the Westbourne. The River Westbourne historically flowed south through Kilburn and on toward Paddington. The River however has been culverted to facilitate local urbanisation which has significantly changed its flow and therefore its tributaries pose little risk to the subject site

Fig.9 – Showing location of site in relation to the Lost Rivers of London (Barton, 1992).

Flood Risk

According to the EA flood map for planning, the site is situated in Flood Zone 1 meaning it is of low risk from flooding, the extracted EA image below shows the proximity of the site to local areas of likely flooding. The specific risks from flooding are then further analysed in the following pages.

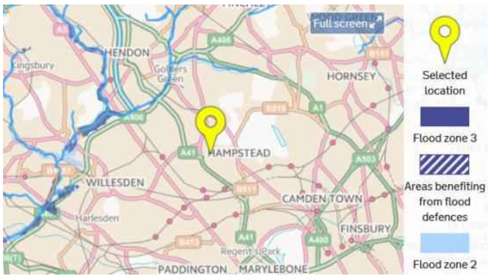


Fig.10 – Showing flood zone according to the Environment Agency (2021).

River or Tidal flooding

The site is located within an area at very low risk of flooding from rivers according to the EA flood warning information service (See below).

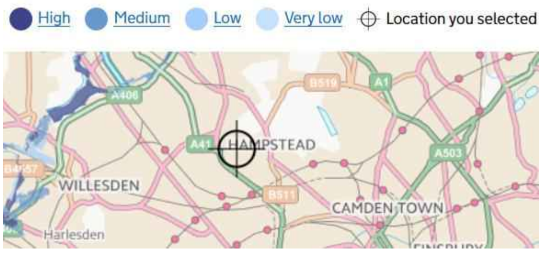


Fig.11 – Showing flood risk from local river flooding according to the EA (2021).

Surface water flooding

The site is again located in an area of very low risk of flooding from surface water (see below).

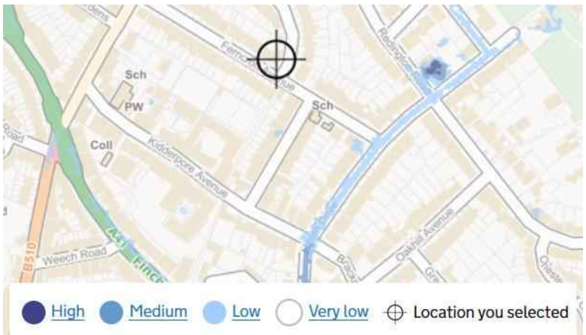


Fig.12 – Showing flood risk from surface water flooding according to the EA (2021).

Very low risk means that each year this area has a chance of flooding of less than 0.1%. This takes into account the effect of any flood defences in the area. These defences reduce but do not completely stop the chance of flooding as they can be overtopped, or fail.

Sub-basement groundwater flow.

An initial web based search indicated that the site is located over a 'Secondary A aquifer' and is at medium risk in terms of ground water vulnerability, as shown in the figures below. The significance of this is further discussed alongside the results from the site investigation below.

Fig.13 (Left) – Showing location of site over the Secondary A aquifer from Figure 8 from Camden Geological, Hydrogeological and Hydrological Study.

Fig.14 (Right) – Showing location of site over a medium risk from groundwater from DEFRA MAGIC - Groundwater Vulnerability Map.

Groundwater flooding occurs when water levels rise above ground level surfaces, and generally poses greatest risk when a site is underlain by permeable strata such as sands and gravels over impermeable bedrocks. Groundwater fluctuates annually and is affected seasonally as variances in moisture rise and fall during wetter and drier seasons with water channelling into and flowing through local water systems.

The site is underlain by a Claygate member, which is classed as Secondary Aquifer by the Environment Agency. The soil type encountered on site is defined as having the ability to allow shallow ground water to flow through it, within lenses of granular material. The site investigation identified that a "very minor" inflow of ground water was encountered at 3.5-4.5m from perched water and at 6.0m where a slow ground water flow was encountered, with limited rise in water level.

The majority of the front section of the proposed basement is either already at lower ground floor level, requiring only a minimal additional excavation, or is above local ground water levels as recorded by the investigations carried out on site. However, the rear of the site will be excavated to a greater degree, closer to the level of recorded ground water levels. This therefore means that there is some chance that water ingress may occur during the excavations in this area. Typical sump dewatering may therefore be required if ground water is encountered during this phase of work, however the limited rise in water levels and slow seepage indicates this will likely be eminently manageable. The position of the swimming pool means that dewatering would likely be remote from neighbouring structures, reducing the likely effect this may have in terms of potential temporary subsidence. Ground water is expected to run following the local topography i.e. in a mainly southerly direction; therefore there is a risk, to a small degree, that the new basement may increase damming to the ground water. This has been considered alongside the composition of the site soil conditions and is considered unlikely to have a significant effect on groundwater flows. This is because the soil type is already predominantly cohesive with flows likely channelled in granular lenses which if encountered may need temporary dewatering. Groundwater movement around this type of proposed structure is also cited in the Geological, Hydrogeological and Hydrological Study which states that large excavations for subterranean structures in London have, to date, not been seen to cause serious problems resulting from damming groundwater. Considering the above, the impact the proposed construction will have on groundwater conditions is considered low, and related structural issues will be manageable with adequate shoring of excavations, local dewatering and design of the basement walls with a full head of water as is standard practice in a basement of this type.

<table>
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Effect of development on local sewerage/surface water systems

The current sewerage system serving the property in its existing condition is assumed to have sufficient capacity to manage any proposed foul water demands as little flow or volume change is expected. It should be noted that this is a pre-planning report and therefore specific proposed drainage details are not available at the time of writing. It can be safely assumed, however, that because the basement will be below the level of existing site drainage, the new basement will utilize a pumped drainage arrangement, incorporating fail-safe systems to minimize risks of flooding. To minimize the storage capacity and pumping requirements for basement drainage systems, it is suggested that the upper levels of the property above ground level are served by a gravity drainage system. It would also be best practice to maintain separation between the new surface water and foul drainage systems throughout the site, only converging at the final manhole before discharging into the assumed combined public sewer.

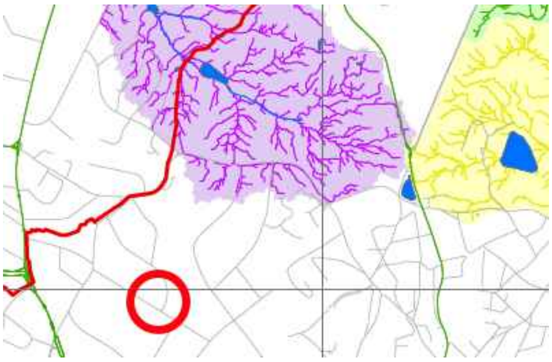

Fig.15 – Showing location of site in relation to the Hampstead Heath Pond Chain from Figure 14 from Camden Geological, Hydrogeological and Hydrological Study.

As shown above, the subject property is not within the catchment of the Hampstead Heath Pond Chain the boundary of which is approximately 300-400m from the site.

The proposed site surface area will remain largely unchanged with proposed areas of hardstanding remaining similar to those in the existing condition, therefore existing surface water discharge rates are not expected to increase.

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<tr><td>Effect of nearby trees</td><td>The soil investigation indicated cohesive Claygate/London Clay members below the site and is therefore likely to have a medium/high susceptibility to shrinkage/swelling. The proposed formation level for the basement however will extend significantly deeper than the level at which tree & vegetative growth would affect soil moisture content. Therefore, the potential effects of nearby trees are not considered in any further detail in this report.</td></tr>
<tr><td>Land Stability</td><td>

The Camden Geological, Hydrogeological and Hydrological Study confirms that an area of significant landslide potential is located at the boundary between the Claygate and London Clay soil types. This boundary however, is remote from the subject property and therefore the proposed works have been judged to pose no risk in terms of landslide potential.
Furthermore, following a site walk over, no obvious evidence could be seen to indicate any issues with previous slope instability, or within the surrounding areas, as far as could be seen at the time of our visit. The proposed excavations will not extend below either of the public highways bounding the property, and are quite far removed from them. Therefore it is very unlikely the construction activities will have any effect on the nearby roads.
With the above taken into account, a full slope stability assessment was not deemed a requirement for this site.

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TfL Infrastructure	The site is not above or in close proximity to the London Underground network of tunnels. The site is remote from the Northern Line, with the closest line on the TFL serving Hampstead station approximately 900m away; the development will therefore not have any effect on any surrounding TfL infrastructure.  Fig.16 – Showing location of site in relation to TFL assets – arcgis.com (2021).
Desk Study Result	• The subject property is not likely to be affected by any of the historic rivers. • The site is located within Flood Zone 1 as confirmed by the Environment Agency Flood Maps; therefore the site is at low risk from flooding. • The site has not been found to be close to any LUL infrastructure. • Preliminary UXO research indicates it is unlikely historic bombing will have had any effect on the development. • There is little change in hardstanding, and drainage through-flow will remain largely the same as existing. • Groundwater is unlikely to be an issue with any inflows being managed with local sump pumping. • The site is remote from the demarcated area of significant landslide potential.

3. BASEMENT CONSTRUCTION

General
The structural proposals for the proposed basement deepening and lightwell construction are included in the Appendices 3 and 4. The deepening and extensions of the existing basement will be carried out in non-sequential underpinning. The majority of the underpinning required to the front will be low-level and formed in reinforced concrete with short stem heights, to the rear full height RC underpinning will be installed, with a short section of wall cast in two phases enabling a swimming pool to be formed. The Basement 'box' will be completed with the casting of RC base and ground floor slabs.
The proposed basement will require a Grade 3 rated basement (BS8102) providing a watertight construction and it is assumed that an internal cavity drained system will be installed to achieve this. The design of all waterproofing is beyond the remit of MiNT Structures and must be confirmed by an appropriate waterproofing specialist.
The design and construction of the proposed scheme must be carried out to comply with current statutory guidelines, British Standards, CDM and H&S requirements.

Basement walls
Basement walls (low-level and full-height) are to be formed in reinforced concrete following the underpinning sequence shown (see fig 17). In the permanent case the concrete walls will support any load applied from the structure over as well as resisting and retaining soil, surcharging & any water present behind them. The proposed construction of the basement walls in this way is supported by Table A3 within the Camden Geological, Hydrogeological and Hydrological study. The study shows that bearing capacity of a footing founded within a cohesive soil is unlikely to be substantially affected by loss of overburden associated excavations near a footing (unless the existing loading case is close to ultimate capacity, which is not the case at the subject property, as shown in the appended estimated existing line loading drawing.)

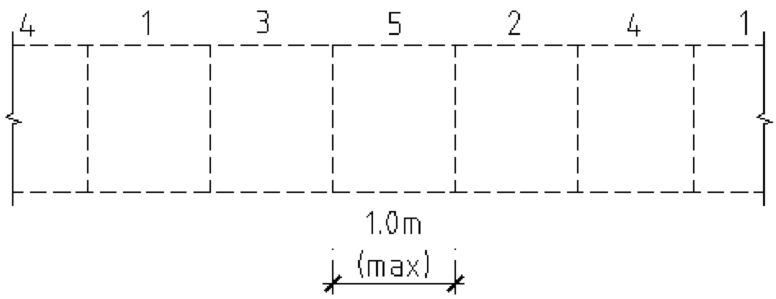

Fig. 17 - Typical underpinning sequence.

Heave protection

The removal of excavated soil to form the basement will significantly reduce the loading on the Claygate and deep London Clay layer present below the property creating the conditions for heave to occur. As stated in GEA's report an uplift of either 40% of maximum unloading pressure (48kN/m^2) should be taken, or implement measures to account for a potential maximum predicted heave movement of 22mm. The suggested layout can accommodate either solution and an example uplift calculation is presented within the appendices 3 of this report, resulting in a requirement of B1131 mesh in the top face of the slab (this assumes the slab spans side-to-side with a centralised RC beam strip to provide stiffness thus splitting the overall span). Alternatively heave protection could be used to negate the effect of heave, taking a high shrinkage soil type and specifying a suitable compressible void former, typically 160-220mm in thickness for basement slabs (slab designs should account for a residual failure uplift load). The selection of the final site specific heave precautions will be driven by the requirements of the final detailed design, taking account of the stability of the basement box as a whole structural system.

Materials	**<u>Concrete used in underpinning/slabs</u>** *(Details below to be confirmed by appointed structural design engineer)* Mix designation: *RC 40* Aggregate size: *20mm* Cube strength: *40N/mm^2* <u>Notes:</u> 1. Unless otherwise instructed a 50mm thick blinding layer should be provided beneath all reinforced concrete to provide a clean level surface and avoid pouring directly on to ground/hard core. 2. High Alumina Cement (HAC) should not be used under any circumstances. <u>Concrete cover</u> All cover should adhere to minimum values specified by the Eurocodes (BS EN 1992-1-1:2004). It is recommended that: Concrete internal cover = *35mm* Concrete external cover = *50mm* Direct contact with ground = *75mm*
Waterproofing	A specialist designed waterproofing system will be required to give the correct level of protection against the ingress of groundwater, the detailed design and specification of measures of this type, however, are beyond the remit of this report. It is advised that a waterproofing specialist is contacted early on in the design process and that, as a minimum, the final waterproofing system complies with BS8102:2009 - *Code of practice for protection of below ground structures against water from the ground*.
Ground Bearing pressure	Allowable GBP @ Basement formation level = 140kN/m^2 (from GEA report) Allowable GBP @ Pool formation level = 200kN/m2 (from GEA report)
Damage Classification	A full ground movement analysis was carried out by GEA to assess the impact of the proposed works on the subject property and the neighbouring buildings. The report produced by GEA provides greater detail with regard to the assessment and analysis carried out to categorise the predicted damage category, and this should be read in conjunction with this report. The excerpt below is taken from the resulting damage category conclusion by GEA. "The analysis has concluded that the predicted damage to the neighbouring properties from the construction of the underpins and excavations would be 'Negligible' to 'Very Slight' and therefore the damage that would occur would fall within the acceptable limits." This statement refers to damage in relation to the Burland scale, the accepted system used to classify damage which is detailed in Table 6.4 of CIRIA report C760, within this Category 0 related to a 'Negligible' damage and Category 1 to 'very slight'.

Differing site/ neighbouring foundation depths.	Mass excavations associated with the construction of basements increase the potential that a development will cause ground movement in the local area if the construction processes are not managed correctly. The proposed basement detailed will significantly increase the difference between the subject and neighbouring propertys' foundation depths. However, these movements can be mitigated with experienced and proper design processes adhering to the relevant Codes of Practice. The design of both the temporary works and permanent works must be carried out by a suitably qualified, and crucially, experienced team of engineers, architects, designers and other specialists working closely to ensure that as far as is practically possible the proposed scheme has any possible weaknesses, where movements may occur, designed out. The Party Wall Etc. Act (1996) applies to the proposed works and the Party Wall legislation must be adhered to throughout the construction period. A ground movement assessment (GMA) has been carried out by GEA to accompany the BIA and this report. The GMA concluded that the proposed development was unlikely to cause significant damage provided good workmanship and logical construction sequences are followed; including but not limited to enabling works, temporary support and correct management of water ingress. **Following the GMA it was predicted that neighbouring structures are predicted to be within the CIRIA C580 Damage Category 1 (Very Slight).** A monitoring plan should be set out at the design stage, however an indicative suggested plan is included in the next section (this is not for any on-site purposes and all monitoring must be carried out by a specialist company to their own method statements).
Resistance to vibration	The resistance of the existing and surrounding buildings is defined in BS 4866:2010 in Annex B and the subject and neighbouring buildings fall into Group 1 in clause B.4.1 (traditionally built) - *"Generally, this group is of heavy unframed construction and has a very high damping coefficient due, for example, to soft lime mortar or plaster".* The foundations for the subject and neighbouring buildings are assumed to fall into Class C in clause B.5.3 (Strip footing/corbelled footings). The soil type from drift maps and boreholes is a Claygate member, therefore according to clause B6 the soil is classified as type e – *"soft cohesive soils (clays)"*. According to table B.1 the subject and neighbouring buildings can be classed as Category 5/6 which indicates a medium to high resistance to vibration. According to Table B.2 the resistance to vibration can be categorised as class 10, which indicates a medium to high level resistance to vibration which requires minor protective measures against vibration. The method of construction detailed with the construction method statement, coupled with the contractor's own method statements for Noise, Dust and Vibration mitigation will protect the neighbouring buildings from the effects of vibration during construction.
Noise	The selected contractor should carry out all site activities trying wherever practically possible to reduce dust, noise, and vibrations. This is especially of concern when working close to neighbouring buildings and public highways, to try to protect neighbours and members of the public. Underpinning should be carried out by hand digging only, the use of excavators should be prohibited. The methods of construction suggested in this report should help to ensure that, construction noise and vibration will be minimised as far as is possible if proper and safe design processes and construction practices are followed.
Party Walls	The proposed works are subject to the Party Wall etc. Act 1996 and it is therefore advised that the client starts the process/instructs a surveyor as early as possible to ensure that the necessary Party Wall awards are in place before work commences.

4 SUGGESTED STRUCTURAL MONITORING PLAN	
Preliminaries	The following suggested monitoring method statement is intended as a purely indicative guidance document to establish a recommended 'base' monitoring level. All details contained within the following pages must be confirmed by a specialist monitoring contractor and are based on what MiNT Structures, as structural engineers, consider to be advisable procedures to minimise damage caused as a result of the proposed construction activities. The monitoring specialist/principal contractor may choose to produce an alternative method of monitoring for specific activities; specifications by the contractor/specialist will supersede/overrule the indicative content of this document. This statement should not be taken as the final monitoring specification and MiNT Structures can accept no liability for any damage caused as a result of deficiencies in monitoring specifications/methods undertaken.
SCOPE OF WORKS	It is proposed to install a retrofit basement at the above address, using non-sequential underpinning to form perimeter basement walls linked at basement founding level with an in-situ cast RC base slab. These works have the potential to cause damage through ground movement or construction related vibrations therefore monitoring is required to attenuate this risk.
REQUIRED LEVEL OF MONITORING	Monitoring should be carried out during construction to aid in ensuring that any movement caused by the proposed construction is not excessive and also to act as a warning indicator to help mitigate damage. It is advised that the final monitoring plan includes the following: • Production of schedules of condition at the neighbouring properties at the beginning (prior to commencement) and at the end of the works, carried out by a relevant Party Wall representative. • Exposure of perimeter existing footings through the digging of trial pits to confirm foundation condition and that any bearing width assumptions made at design stage are appropriate. • Regular visual inspections of walls being underpinned. • Vertical monitoring measurements. • Lateral monitoring measurements.
METHODS OF MONITORING	- GENERAL VISUAL MONITORING OF THE PARTY WALLS - CRACKING TO PARTY WALL MASONRY – *ATTACH DEMEC PINS/TELL-TAILS TO RECORD SIGNIFICANT CRACKING.* - SETTLEMENT MONITORING – *AUTOMATIC LEVELLING EQUIPMENT AND TARGETS.* - LATERAL MONITORING – *MEASURING OF DISTANCES BETWEEN EXTERNAL WALLS VIA TARGETS/LASER MEASURING TO RECORD ANY RELATIVE DIFFERENCES BETWEEN WALL FACES.* *General notes:* *1. The number of and positioning of levelling equipment will likely be required to change during construction, this should be agreed between the contractor and monitoring specialist as work progresses.* *2. It should be ensured that throughout construction all required monitoring can be accomplished with ease.* *3. Levelling equipment and targets should be protected against damage and clearly marked on site.* *4. Any monitoring equipment damaged during site works should be reported to the monitoring specialist and replaced immediately.* *5. All readings should be regularly distributed to the design team and should be presented in a neat and easily comprehensible manner. A summary of readings should be distributed within 2 working days following observations.*

PRINCIPAL CONTRACTOR RESPONSIBILITIES	1. The contractor must take responsibility for ensuring that all site working practices are planned to minimise settlement as far as practically possible, this should also involve ongoing reviews of working methods to mitigate progressive damage if settlement is recorded. 2. The contractor must also take responsibility for the execution of immediate reparation works if required following settlement readings over specified trigger levels (see trigger values below). 3. The contractor should review all monitoring readings with the monitoring specialist prior to distribution to the design team and check all readings are accurate.
TRIGGER VALUES	A 'traffic light' system should be adopted with the use of Green, Amber and Red trigger levels as follows (values based on GEA predicted movements); **GREEN** *(0-6mm)* – Activities OK to proceed. **AMBER** *(6-12mm)* – Increase the monitoring frequency (minimum twice weekly), review of structural scheme and start implementing contingency measures if trends indicate the Red trigger may shortly be reached. [Showing recorded values are close to maximum projected settlement (say max. 80% of predicted settlement)] **RED** *(>12mm)* – Implement measures to secure site, cease movements and stop all construction works. [Showing recorded values are at, or above tolerable levels, exceeding serviceability limit states.] Where maximum movements are recorded exceeding Amber/Red trigger values these should immediately be reported to the design team along with a description of all recent on- site activities. A review of the results should be undertaken and readings re-checked to confirm their accuracy, the design team should not assess the movement focussing solely on the affected areas but also review the site as a whole, checking for non-proximate contributory factors. Appropriate repair specifications and reviews of working practices should be specified and implemented to minimise risk of progressive settlement. NOTE: The trigger levels suggested within this document are indicative only. Final movement levels must adhere to Local Authority guidelines and specialist guidance, with these being obtained before confirmation of final trigger values.
TARGET LOCATIONS	Precise locations for levelling targets should be prescribed by the monitoring specialist, however, the following guideline is suggested; As stated in the BRE digest 386 – *Monitoring building and ground movement by precise levelling* – a minimum of 8-12 target locations should be installed around the whole site to provide Northing, Eastings and Level measurements to an accuracy of ±0.3mm (It is recommended that targets are installed at each storey height). Consideration should also be given to the provision of monitoring locations on neighbouring structures (provisions of this type TBC by monitoring specialist and relevant Party Wall representative).
MONITORING FREQUENCY	• PRIOR TO COMMENCEMENT OF CONSTRUCTION, MONITORING READINGS SHOULD BE CARRIED OUT ONCE TO ESTABLISH A SET OF CONTROL VALUES. • AS SOON AS BASEMENT CONSTRUCTION COMMENCES MONITORING SHOULD BE CARRIED OUT, READINGS SHOULD BE TAKEN FOLLOWING THE CURING OF EACH OF THE FIRST FIVE PINS POURED. IF LEVELS OF OBSERVED SETTLEMENT ARE WITHIN ACCEPTABLE LIMITS FOLLOWING THE FIRST FIVE PINS, MONITORING FREQUENCY CAN BE REDUCED TO TAKING READINGS FOLLOWING CASTING OF EVERY OTHER PIN. • POST-CONSTRUCTION- FOLLOWING COMPLETION OF ALL WORKS MONITORING SHOULD BE TAKEN TWICE MORE AS A MINIMUM. Note: Final monitoring intervals and levels of pre- & post-construction readings must be confirmed by the specialist monitoring contractor.

5. SUGGESTED METHOD STATEMENTS

SUGGESTED OVERALL SEQUENCE OF WORKS

The site position is advantageous for the proposed works due to access being available from both the front and side of the property. Deliveries can be taken easily to the rear of the site where there will be little construction activity being carried out.

The brief sequence of works laid out below is suggested only and should be confirmed and superseded by the appointed contractor following delivery of the full construction design package.

Set-Up: Hoard the full open perimeter of the site. Services running through the site should be identified accurately and safely isolated and any measures to protect trees or vegetation should be implemented as agreed with relevant parties.

Preparatory Works: An agreed monitoring regime should be set up to keep a watching brief on any movement caused by excavation to ensure they are within acceptable limits. Non-structural strip out can be carried out to remove any internal elements that will enable easier access for underpinning including any suspended timber floors or ground bearing slabs. Confirm the working methods for transfer and removal of spoil from the site; it is assumed, at this stage, soil will be removed using a typical conveyor to deposit soil into skips to be removed by skip or grab lorries.

Superstructure temporary support: Where required for access, temporary beams and needles should be installed to allow works to progress, installed as specified in the Temporary Works package.

Non-consecutive underpinning: Dig initial underpins in a few locations across the site ready for inspection of quality of the soil condition, to confirm suitability of proposed underpinning method. Once confirmation of soil condition has been carried out the underpinning of the main house can continue in a contractor agreed sequence, installing the RC sections in widths as specified in the construction drawings / method statements, installing specified heavy duty horizontal/raking propping to resist lateral movement.

Mass Excavation: Following completion of the permanent underpinning, the central retained earth bund can be reduced, installing horizontal propping across the site in accordance with the Temporary Works specifications. These must remain in place until the full basement construction process is complete and cured to provide full support.

Forming the RC box: Following mass excavation the basement slab can be cast with lateral propping remaining in place. Permanent support to the superstructure should be installed next, allowing the casting of the ground floor slab, thus completing the basement box (barring the formation of the swimming pool).

Swimming Pool Excavation: To the rear of the basement, a swimming pool is to be installed, resulting in the retaining wall needing to be cast in two sections. The first phase of underpinning will initially be taken down to the main basement level allowing the formation of a temporary boot/RC lateral support beam. Following mass excavation and full propping, the swimming pool formation level can excavated in an underpinning sequence forming the pool at sub-basement level, with an RC box homogenously linked to the basement slab (see suggested sequencing on drawings B1 – B5). Following curing of the swimming pool box and basement area as a whole, all shoring and temporary propping can be removed and other construction activities can continue above and within the basement.

1. TYPICAL BASEMENT UNDERPINNING SEQUENCE

1.1. Remove existing timber floor (if required for access).

1.2. Hand excavate pins in sections not exceeding 1.0m following numbered sequence provided in Temporary Works drawing package. (Typical number sequence shall be 1, 3, 5, 2, 4) under no circumstances are adjacent pins to be opened during construction.

1.3. During excavations ensure vertical faces are shored at all times using 18mm ply, timber wailing pieces and horizontal strutting. The exposed face of the excavation should be lined with 'Hardie Backer 500' cement board trench sheeting or similar permanent sacrificial shuttering with de-bonding membrane installed to the inside face of trench sheets prior to concreting.

1.4. Reinforcement should be placed in position in preparation for casting the underpinning base, starter bars should be provided to enable a connection between the base and the vertical stem to be formed.

1.5. Local authority building control officer or appointed inspector to inspect and pass reinforcement prior to concreting base section.

1.6. Pour concrete base and kicker sections to structural engineer's details. Use vibrating pokers to ensure full compaction of concrete and removal of trapped air pockets within forms.

1.7. Once base has sufficiently cured (min 24 hours) place reinforcement to vertical stem including horizontal dowel link bars to neighbouring pins (horizontal dowels to structural engineer's specification).

1.8. Formwork to be secured with heavy timbers and "Leada Acrow" or similar trench props supported off of the central earth mass to retain the concrete during pouring. Leave 75mm clearance between top of concrete pour and underside of existing foundation.

1.9. Pour concrete stem section to structural engineer's details, use vibrating pokers to ensure full compaction of concrete within forms.

1.10. Allow 48 hours curing time between concrete pour and installation of dry pack. Clean underside of existing foundation using wire brush or similar in preparation for installation of dry pack.

1.11. Use 1:3 dry pack well rammed into position between head of pin and underside of existing foundation (Dry pack to be installed after each individual pin has been cured see point 2.1 regarding corbel removal.)

1.12. Strike formwork following lapsing of sufficient curing period (normally approximately 7 days)

1.13. Underpinning is to continue according to sequence specified in Temporary Works drawing package following previously described method.

1.14. Central earth mass is to be retained to enable local shoring of pins and trenches as underpinning progresses.

1.15. Following completion of all underpinning the central soil mass can be excavated in stages to allow installation of high level lateral "Mabey Mass 50" or similar engineer approved props in accordance with propping plan (Drwg TW1).

1.16. The remaining central soil mass can now be removed and a second row of lateral props can be installed to restrain the lower 3rd of the pins.

1.17. Excavate for reinforced concrete basement slab ensuring lateral propping remains in place at all times.

1.18. Compact base of slab excavation and place reinforcing bars to structural engineer's specification.

1.19. Cast basement slab to structural engineer's details using vibrating pokers to ensure full coverage of concrete and removal of trapped air pockets.

1.20. Once basement slab has sufficiently cured (min 14 days) the remaining propping can be removed.

2. REMOVAL OF MC OVERHANG

Upon completion of underpinning and sufficient curing of dry packing has been allowed, the existing MC foundation projection can be removed using hand tools to leave the wall over, flush with the face of the RC underpinning. Care should be taken when removing the corbel to avoid causing undue damage. Where brickwork is in poor condition it should be carefully made good in small increments.

3. INSTALLATION OF STEEL BEAMS/FRAMES

The method described below is a typical generic steel beam/frame installation; full requirements for shoring of superstructure should be assessed on site at the start of the project through opening up and inspection of existing structure

Prior to any underpinning or steel work installation the contractor may also carry out the following works:

- Carry out a verticality survey to check walls are plumb.
- Provide bracing to openings including doors and windows with timber constructed frames.

Where frames are to be installed and supported at basement level, pin sections supporting columns/beams should be excavated and cast first prior to any steel installation being carried out.

Connection details, splices and base plates to be installed in accordance with structural engineer's specification.

Installation Method

6.1. First install securely diagonally braced "Leada Acrow" propping placed either side of the wall requiring support, props should be sited on paving slabs bearing on well consolidated ground throughout.

6.2. Install UC needle beams at high level spanning between the Acrow dead shoring to provide support to the brickwork over and enable removal of masonry panel below.

6.3. Once needling/propping is positioned, tightened brickwork below can be carefully removed by hand.

6.4. Where permanent steel framework is specified members needed to transfer loads on to RC pins should be installed in accordance with structural engineer's details to provide a bearing for the high level beam.

6.5. Where bearings are specified, cut slots into walls to accept padstone or bearing plates as specified by structural engineer (allowing 48 hours to cure where padstones are cast).

6.6. Insert permanent steel beams either fixed to columns or seated 100mm into walls at each end on bearings.

6.7. Dry packing should be placed between the top flange and the underside of the wall over, allowing 48 hours to cure. (Where beam is seated on bearings dry pack should also be placed 75mm above and below the beam well rammed into position and any defective brickwork around beam ends should be removed and made good using class B engineering bricks and 1:3 mortar once dry pack has cured.)

6.8. Following the provision of full support to the wall above, (and bracing has been securely fitted if frame installation is being carried out) any temporary works in relation to its support can be removed.

6.9. Any voids in the brickwork where needles had been positioned should now be repaired by bricking up.

6.10. Once adequate support has been provided by the permanent works structure underpinning can proceed as specified in fig.3.

4. DEWATERING DURING CONSTRUCTION

7.1. If during any excavation work significant ground water ingress is found, a local $1m^3$ sump should be provided formed at a level below the base of the excavation being worked on.

7.2. The vertical faces of the sump chamber should be supported with a pre-made shutter positioned in the area excavated for the sump. The sump shutter should be constructed from 18mm thick plywood sheets with drilled vertical faces to provide a porous surface allowing ground water to flow through.

7.3. Ground water will now flow into the excavated sump to be extracted using a suitable Semi Trash dewatering pump and appropriate diameter discharge hose.

7.4. Discharge from the sump should be directed to the nearest manhole and a drain filter should be fitted to avoid any large debris being deposited into the sewer.

7.5. After completion of the excavation and preparation for the concrete pour has been carried out ensure the sump area is fully dewatered before removing pump and pouring concrete.

7.6. The process above should then be repeated for each excavation where ground water is found.

6 CONCLUSION	
CONCLUSION	The result of this preliminary pre-planning report indicates that the proposed basement can be completed successfully without causing undue impact on neighbouring buildings, or its local surroundings, provided the works are undertaken by suitably qualified and experienced contractor/s. The suggested use of standard and well proven construction methods/materials mean the inherent risks often associated with largescale mass excavations will be largely mitigated in this case, provided careful design and onsite practices are adhered to. In making this conclusion, it is assumed the suggested recommendations and sequence of works above will be largely similar to those used in the final design. The works must be constructed in adherence with all relevant statutory guidelines, designed by a suitably experienced and qualified design team. Detailed and well-designed method statements and calculations for all enabling and temporary works must be prepared well in advance of the commencement of site activities. These must also be distributed for comment from all relevant parties. It is imperative that professional monitoring is carried out to record the movement of the subject and neighbouring properties over the full course of works, to be agreed and included as a part of Awards produced under the Party Wall Etc. Act 1996. Relevant parties such as Party Wall surveyors, Design engineers and site management members will be required to ensure that adequate supervision and monitoring are carried out, paying particular attention during critical stages such as excavation and demolition. The proposed development is unlikely to significantly increase flood risk at the site and its surrounding area. It should be noted that the above conclusion is based on the information available at the time of writing and should also be read alongside recommendations and conclusions made in the site specific geotechnical analysis.

7 REFERENCES

1) Codes / Regulations

I) Eurocode : Basis of structural design (BS EN 1990:2002)

II) UK National Annex for Eurocode : Basis of structural design (NS BS EN 1990:2002)

III) Eurocode 1 : Actions on structures (BS EN 1991:2005)

IV) UK National Annex for Eurocode 1 : Actions on structures (NA BS EN 1991:2005)

V) Eurocode 2 : Design of concrete structures (BS EN 1992-1-1:2004)

VI) UK National Annex for Eurocode 2 : Design of concrete structures (NA BS EN 1991-1-1:2004)

VII) Eurocode 3 : Design of steel structures (BS EN 1993-1-1:2005)

VIII) UK National Annex for Eurocode 3 : Design of steel structures (NA BS EN 1993-1-1:2005)

IX) The Building Regulations 2000 : part A - Structure

X) Camden geological, hydrogeological and hydrological study

2) Books / Manuals

I) Concrete Basements: Guidance on the design and construction of in-situ concrete basement structures – R. S. Narayanan & C. H. Goodchild.

II) How to Design Concrete Structures using Eurocode 2 – A. J. Bond et al.

III) Manual for the design of steelwork building structures to Eurocode 3 (October 2010) – IStructE.

IV) Reynolds's Reinforced Concrete Designer's Handbook 11th Edition - C. E. Reynolds et al.

V) Standard Method of Detailing Structural Concrete 3rd Edition (June 2006) – IStructE.

8 APPENDICES

Appendix 1 – Males. J. 1997 - Regional Climates of the British Isles

Appendix 2 –Borehole logs / location plan.

Appendix 3 – Preliminary structural calculations.

Appendix 4 –Preliminary structural drawings. B1 & B5

Appendix 5 - Architectural Ground floor & Basement Plans

Appendix 1 – Males. J. 1997 – Regional Climates of the British Isles, p74 Average Precipitation for the period 1961–1990 (SJP-610mm).

more a function of exposure and relief. For example, the driest parts of the south coast are found to the lee of the Isle of Wight, where Southsea averages 685 mm, and the low-lying Dungeness Point has a similar total.

The significance of exposure to the south-west and relief were confirmed by Stone (1983), whose studies revealed that the mean daily rainfall receipt from cyclonic SW airstreams increased from 2 mm around the Thames estuary to over 7 mm around the New Forest. Cyclonic W and S airstreams also gave twice as much rain over the Weald of East Sussex as around the Thames estuary. By contrast, in cyclonic northerly and north-easterly situations the driest areas were south Hampshire and around Dungeness.

The greater the prevalence of south-west winds, the more conspicuous the wetness of the southern coastal counties. A blocked situation, with abundant northerlies or easterlies, gives a more uniform distribution to rainfall across the region. It follows that the seasonal variation in the frequency of westerly winds helps to determine the seasonal variation in rainfall. Frontal rainfall, for example, is more frequent, heavier and more likely to approach from the south-west in autumn and winter. This seasonal intensification of the westerly circulation leads to a subtle backing of the surface wind towards the south-west, increasing the exposure of the south coast. As Table 3.4 shows, this leads to a peak in monthly rainfall in autumn and winter for southern coastal locations. Summer rainfall, more dependent on the contribution from convectional sources, however, shows a slightly different pattern, and tends to decrease southwards despite an increase in annual totals.

A feature of the 1961–90 averages is the large area of southern England that has a minimum in July. Further north, in the Thames valley, the driest month is still February, by a margin of over 10 mm, whilst Goudhurst, on the high ground of the Weald of Kent, is as dry as London in July, despite being 32 per cent wetter over the year as a whole. Bexhill is 58 per cent wetter than London in November but nearly 10 per cent drier in July. Summer rainfall (June to August) contributes over 25 per cent of the annual fall in London and Oxfordshire northwards but as little as 18 per cent on the Isle of Wight, the latter being the lowest proportion in the British Isles. Conversely, the 61 per cent contribution of the winter half-year (October to March) on the Isle of Wight is the largest such value, and exposed coastal sites such as Ryde (Table 3.4) can record annual totals more commonly associated with higher ground in the region. The proximity of high pressure over or to the south of the area in many summers in the 1970s and 1980s has had a marked effect in decreasing summer rainfall (Mayes 1991). Consequently, the melancholy scene of parched landscapes noted by Southern (1976) has reappeared in many subsequent summers, several of which have accumulated significant water deficits (Woodley 1991).

Notable Rainfall Events

The relative importance of different types of rainfall is markedly different from that in north-west

Table 3.4 Average monthly and annual precipitation (mm) for the period 1961–90.

Location	Alt. (m ASL)	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
St James's Park	8	53	36	48	47	51	50	48	54	53	57	57	57	611
East Malling	33	62	41	49	46	47	50	45	48	60	60	67	65	640
Goudhurst	85	86	57	65	55	54	55	48	58	70	87	88	86	809
Bexhill	4	77	50	56	47	45	48	44	49	64	82	90	77	729
Southsea	2	74	50	57	45	47	42	37	52	60	71	75	75	685
Ryde (IoW)	4	86	57	62	48	50	45	40	55	66	82	86	84	760

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Site Plan

Site	30 Ferncroft Avenue, London NW3 7PH	**Job Number** J20268
Client	Mr Dukagjic Lipa	**Sheet** 1 / 1
Engineer	Mint Structures	

Approximate scale in metres

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Geotechnical & Environmental Associates

Project				BOREHOLE No
30 Ferncroft Avenue, London NW3 7PH				**BH1**
Job No J20268	Date 03-12-20 04-12-20	Ground Level (m OD) 100.05	Co-Ordinates ()	
Client Mr Dukagjin Lipa		Engineer Mint Structures		Sheet 1 of 2

Depth	Type No	Test Result	Water	Reduced Level	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
				99.85	0.20	TOPSOIL (dark brown silty slightly gravelly CLAY)	
0.40	D					Firm medium strength to high strength orange-brown mottled brown and grey silty sandy CLAY, locally clayey sandy SILT	
1.00	D						
1.20-1.65	U						
1.70	D						
1.90	D						
2.00-2.45	SPT	2,2/3,3,3,4 N60 = 14					
2.80	D						
3.00-3.45	U				(6.30)		
3.50	D						
4.00-4.45	SPT	1,2/3,3,3,4 N60 = 14					
5.00-5.45	U						
5.50	D		1			5.50 ... becoming stiff	
6.00-6.45	SPT	3,4/4,5,6,7 N60 = 23					
				93.55	6.50	Stiff high strength dark grey silty CLAY with partings of pale grey fine sand and sandy silt	
6.70	D		1				
7.50-7.95	U						
8.00	D						
9.00-9.45	SPT	3,3/4,4,5,5 N60 = 19			(5.50)		

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth
6.50	03-12-20	00.00	6.00	150	
6.50	04-12-20	08.30	6.00	150	5.6

GENERAL REMARKS

Excavating services insepction pit from GL to 1.20 m for 1 hr.
Groundwater monitoring standpipe installed to 7.00 m.
Groundwater monitoring on 14/01/21 recorded groundwater at a depth of 3.80 m.
Groundwater monitoring on 29/01/21 recorded groundwater at a depth of 3.71 m.
Groundwater monitoring on 11/02/21 recorded groundwater at a depth of 3.67 m.

All dimensions in metres Scale 1:62.5	Method/ Plant Used Cable percussion rig	Logged By ML

GEA

Geotechnical & Environmental Associates

Project				BOREHOLE No
30 Ferncroft Avenue, London NW3 7PH				**BH1**
Job No: J20268	Date: 03-12-20 04-12-20	Ground Level (m OD): 100.05	Co-Ordinates ()	
Client: Mr Dukagjin Lipa		Engineer: Mint Structures		Sheet 2 of 2

SAMPLES & TESTS: Depth	Type No	Test Result	Water	STRATA: Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
							Stiff high strength dark grey silty CLAY with partings of pale grey fine sand and sandy silt(continued)	
10.50-10.95	U							
11.00	D		2					
			3	88.05		12.00		
12.00-12.45	SPT	5,5/6,6,7,7 N60 = 28					Stiff becoming very stiff high strength to very high strength fissured dark grey locally greyish brown and brownish grey silty CLAY with mica	
13.50-13.95	U							
14.00	D							
15.00-15.45	U							
15.50	D							
						(8.00)		
17.00-17.45	U							
17.50	D							
18.00-18.45	U						18.00 ... occasional shells	
18.50	D							
18.60-19.05	SPT	5,6/7,7,9,10 N60 = 35						
				80.05		20.00		

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth
20.00	04-12-20	16.00	8.00	150	14

GENERAL REMARKS

Excavating services insepction pit from GL to 1.20 m for 1 hr.
Groundwater monitoring standpipe installed to 7.00 m.
Groundwater monitoring on 14/01/21 recorded groundwater at a depth of 3.80 m.
Groundwater monitoring on 29/01/21 recorded groundwater at a depth of 3.71 m.
Groundwater monitoring on 11/02/21 recorded groundwater at a depth of 3.67 m.

All dimensions in metres Scale 1:62.5	Method/ Plant Used: Cable percussion rig	Logged By: ML

GEA

Geotechnical & Environmental Associates

Project				BOREHOLE No
30 Ferncroft Avenue, London NW3 7PH				BH2
Job No: J20268	Date: 04-12-20 04-12-20	Ground Level (m OD): 97.61	Co-Ordinates ()	
Client: Mr Dukagjin Lipa		Engineer: Mint Structures		Sheet: 1 of 1

SAMPLES & TESTS				STRATA				
Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
0.20	ES			97.31		(0.30) 0.30	TOPSOIL (dark brown silty slightly gravelly CLAY)	
				96.61		(0.70) 1.00	Firm orange-brown silty sandy CLAY	
				94.61		(2.00) 3.00	Orange-brown clayey sandy SILT to sandy silty CLAY	

Boring Progress and Water Observations					
Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater not encountered.
Groundwater monitoring standpipe installed to 3.00 m.
Groundwater monitoring on 14/01/21 recorded the standpipe to be dry.
Groundwater monitoring on 29/01/21 recorded groundwater at a depth of 2.58 m.
Groundwater monitoring on 11/02/21 recorded groundwater at a depth of 2.33 m.

All dimensions in metres Scale 1:31.25	Method/ Plant Used: Window sampler	Logged By: ML

Project				BOREHOLE No
30 Ferncroft Avenue, London NW3 7PH				**BH3**
Job No J20268	Date 04-12-20 04-12-20	Ground Level (m OD) 98.57	Co-Ordinates ()	
Client Mr Dukagjin Lipa		Engineer Mint Structures		Sheet 1 of 1

SAMPLES & TESTS			Water	STRATA				Instrument / Backfill
Depth	Type No	Test Result		Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	
0.20	ES			98.27		(0.30) 0.30	MADE GROUND (greyish brown clayey sandy silt with brick and slate fragments)	
				95.57		(2.70) 3.00	Firm brownish grey mottled pale grey and orange-brown laminated silty sandy CLAY	
				94.57		(1.00) 4.00	Orange-brown clayey sandy SILT to sandy silty CLAY	

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater not encountered.
Groundwater monitoring standpipe installed to 4.00 m.
Groundwater monitoring on 14/01/21 recorded groundwater at a depth of 3.20 m.
Groundwater monitoring on 29/01/21 recorded groundwater at a depth of 3.18 m.
Groundwater monitoring on 11/02/21 recorded groundwater at a depth of 3.15 m.

All dimensions in metres Scale 1:31.25	Method/ Plant Used Window sampler	Logged By ML

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Trial Pit No 1

Job Number J20268

Sheet 1/2

Dates 04/12/2020

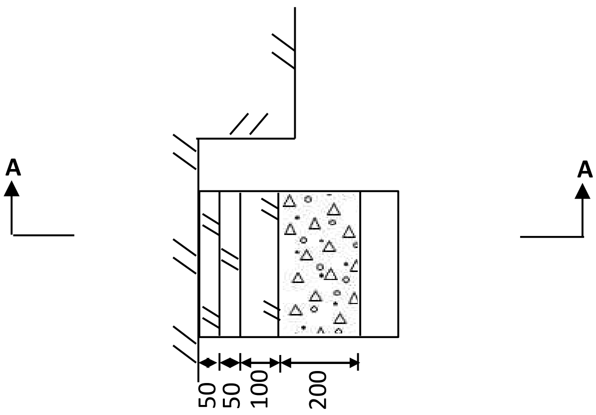
Site 30 Ferncroft Avenue, London NW3 7PH

Client Mr Dukagjin Lipa

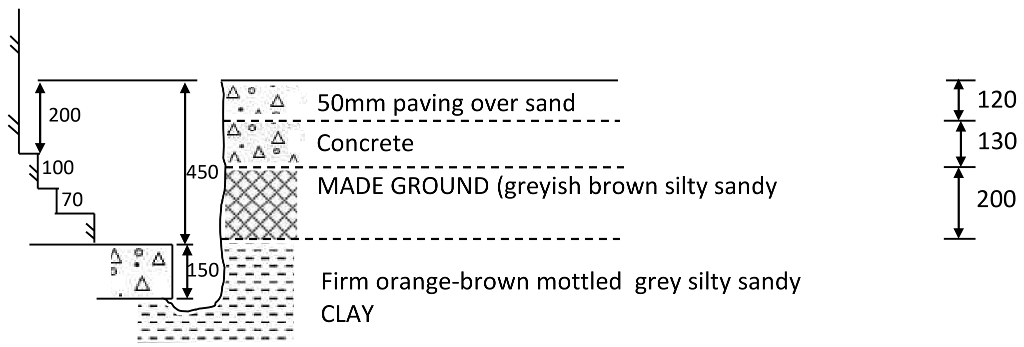
Engineer Mint Structures

Excavation Method	Dimensions	Ground Level (mOD)	Location
Manual	500 x 400 x 600	98.59	Northwestern elevation of main house

PLAN:



SECTION A-A:



Remarks:

All dimensions in millimetres

Sides of trial pit remained stable during excavation

Groundwater not encountered

Scale: 1:20

Logged by: ML

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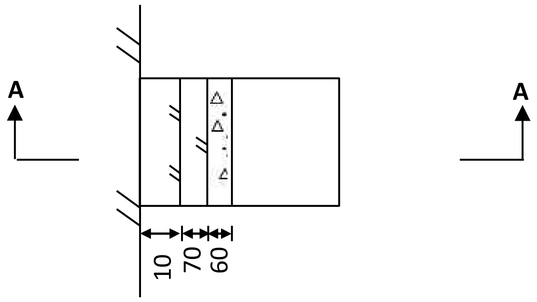
Trial Pit No 2

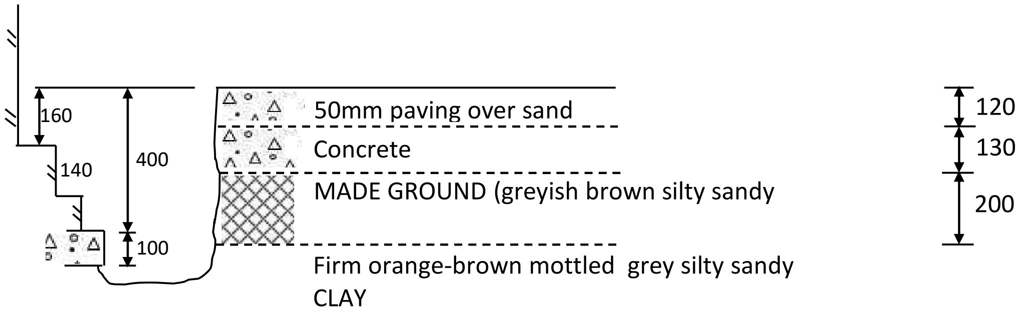
Site 30 Ferncroft Avenue, London NW3 7PH	**Job Number** J20268
Client Mr Dukagjin Lipa	**Sheet** 1/2
Engineer Mint Structures	**Dates** 04/12/2020

Excavation Method	Dimensions	Ground Level (mOD)	Location
Manual	500 x 350 x 500	98.59	Neighbouring house elevation

PLAN:



SECTION A-A:



Remarks:	Scale:
All dimensions in millimetres	1:20
Sides of trial pit remained stable during excavation	Logged by:
Groundwater not encountered	ML

GEA

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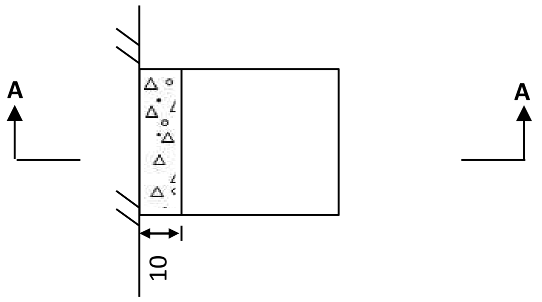
Herts | 01727 824666   Notts | 01509 674888

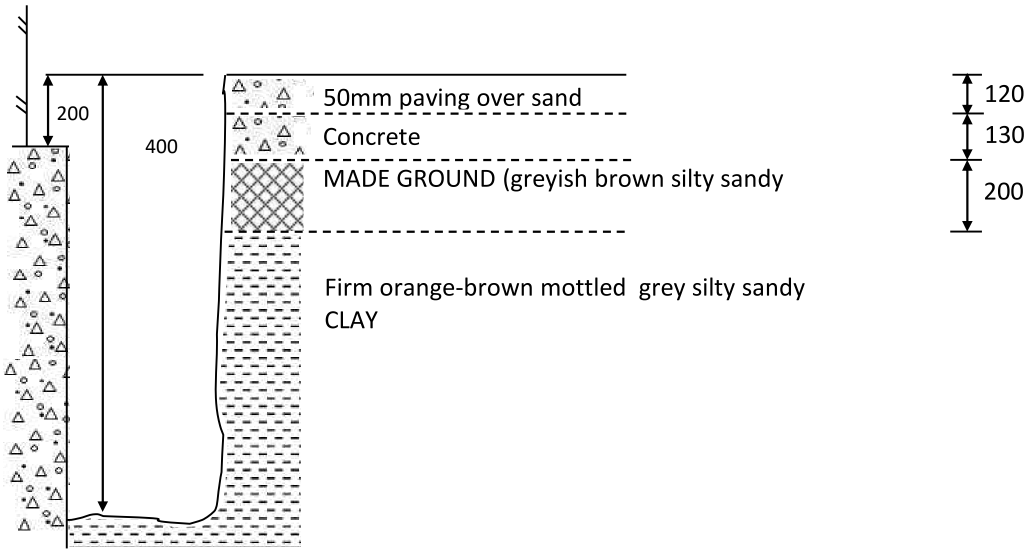
Site 30 Ferncroft Avenue, London NW3 7PH	**Trial Pit No** 3
	Job Number J20268
Client Mr Dukagjin Lipa	**Sheet** 1/2
Engineer Mint Structures	**Dates** 04/12/2020

Excavation Method	Dimensions	Ground Level (mOD)	Location
Manual	500 x 400 x 1300	99.93	Neighbouring single storey extension elevation

PLAN:



SECTION A-A:



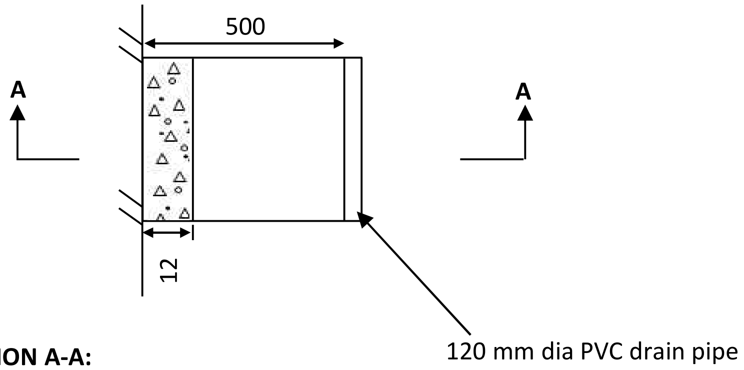
Remarks:	Scale: 1:20
All dimensions in millimetres — Base of footing not established	
Sides of trial pit remained stable during excavation	Logged by: ML
Groundwater not encountered	

GEA	www.gea-ltd.co.uk Herts | 01727 824666 Notts | 01509 674888	**Trial Pit No** 4

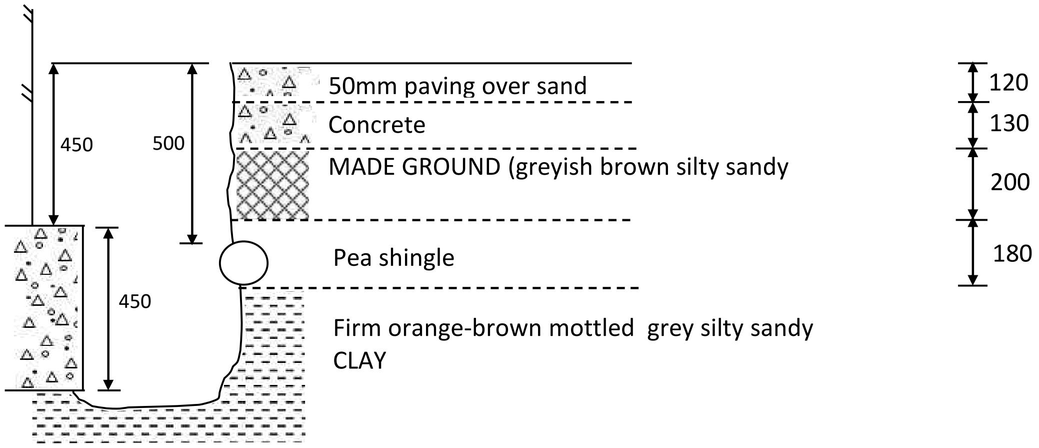
Site 30 Ferncroft Avenue, London NW3 7PH	**Job Number** J20268
Client Mr Dukagjin Lipa	**Sheet** 1/2
Engineer Mint Structures	**Dates** 04/12/2020

Excavation Method	**Dimensions**	**Ground Level (mOD)**	**Location**
Manual	550 x 450 x 900	99.96	Northeastern elevation of extension

PLAN:



SECTION A-A:



Remarks:	Scale:
All dimensions in millimetres	1:20
Sides of trial pit remained stable during excavation	Logged by:
Groundwater not encountered	ML

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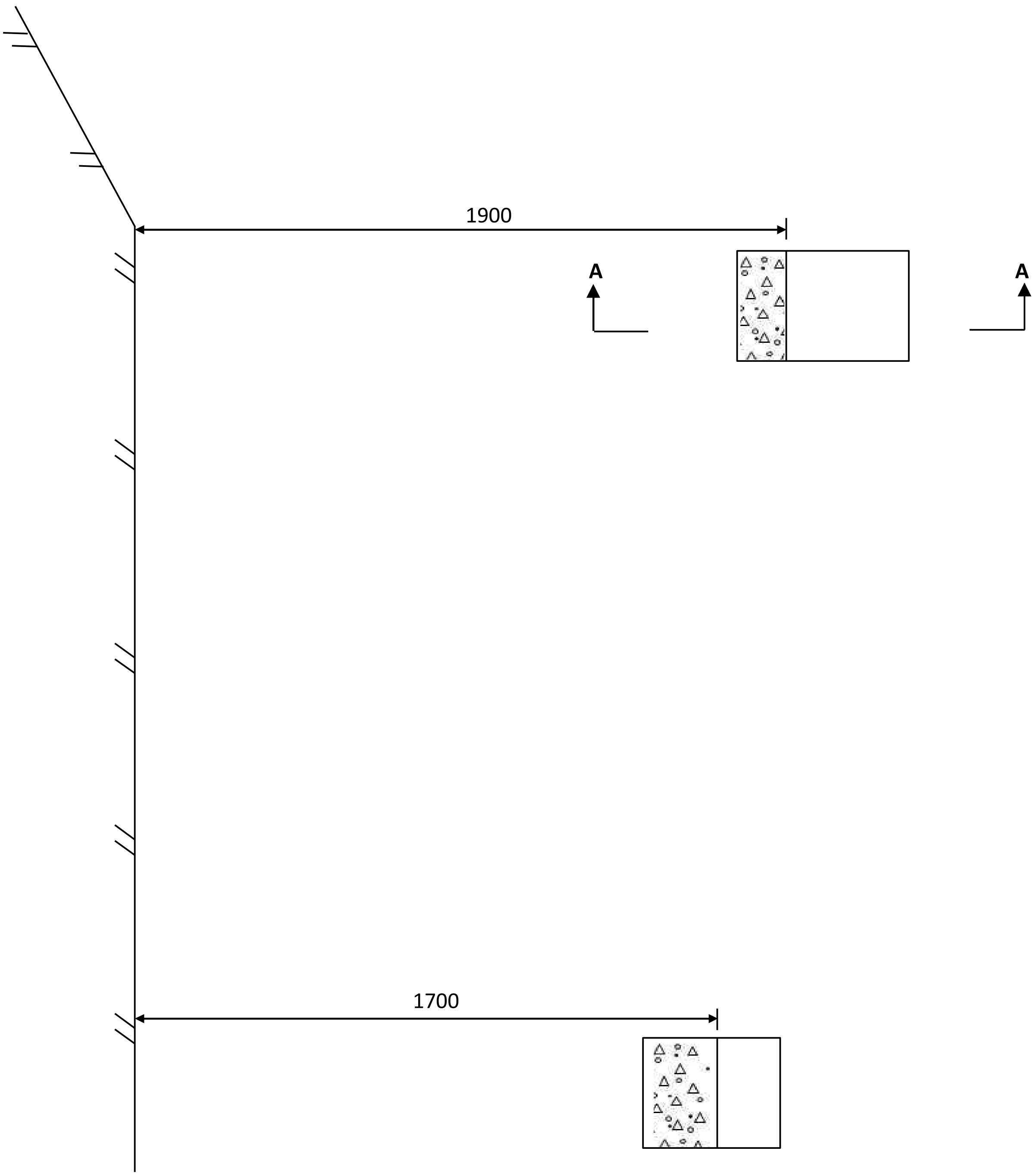
Herts | 01727 824666 Notts | 01509 674888

Site 30 Ferncroft Avenue, London NW3 7PH				**Trial Pit No** 5 & 5A
Client Mr Dukagjin Lipa				**Job Number** J20268
Engineer Mint Structures				**Sheet** 1/3
				Dates 04/12/2020
Excavation Method Manual	**Dimensions** 500 x 300 x 1100	**Ground Level (mOD)** 100.04	**Location** Retaining wall along northwestern boundary	

PLAN:



Remarks:	Scale: 1:20
All dimensions in millimetres	
Sides of trial pit remained stable during excavation	Logged by: ML
Groundwater not encountered	

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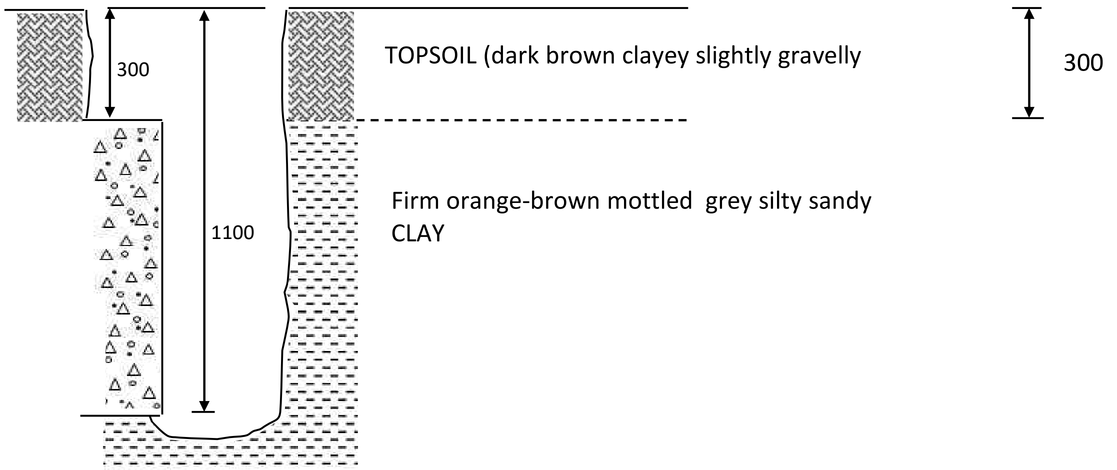
Herts | 01727 824666 Notts | 01509 674888

Trial Pit No 5 & 5A

Site 30 Ferncroft Avenue, London NW3 7PH	**Job Number** J20265
Client Mr Dukagjin Lipa	**Sheet** 2/3
Engineer Mint Structures	**Dates** 04/12/2020

Excavation Method	Dimensions	Ground Level (mOD)	Location
Manual	500 x 300 x 1100	100.04	Retaining wall along northwestern boundary

SECTION A-A:

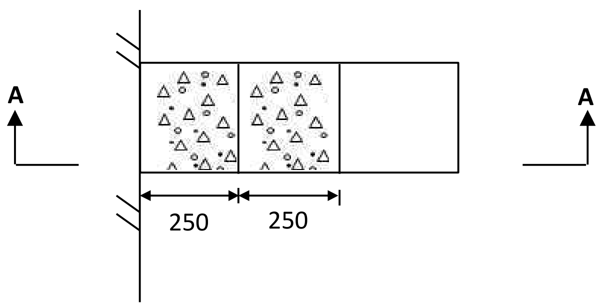


Remarks:		Scale:
All dimensions in millimetres	Rainwater ingress into the trial pit	1:20
Sides of trial pit remained stable during excavation		Logged by:
Groundwater not encountered		ML

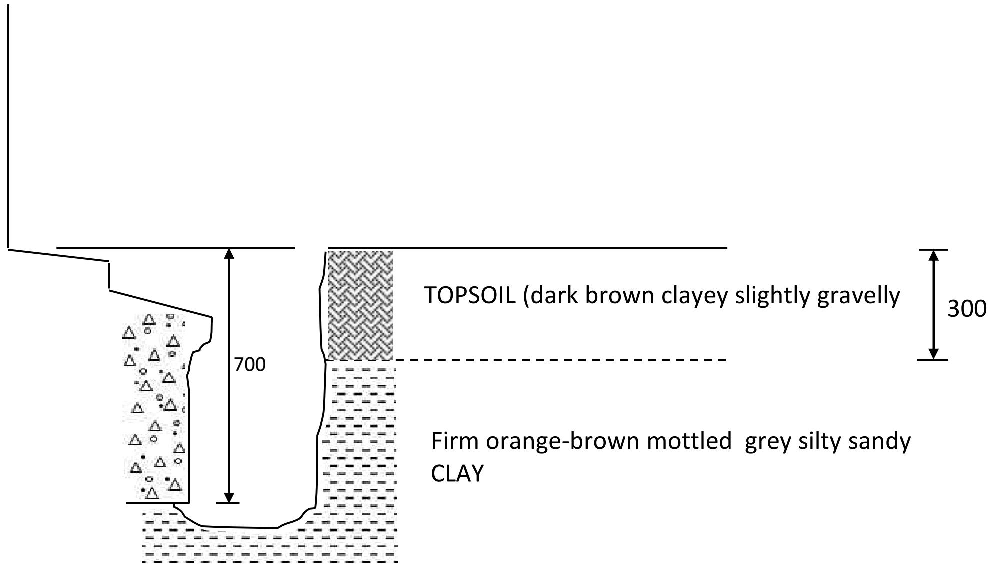
GEA www.gea-ltd.co.uk Herts | 01727 824666 Notts | 01509 674888	**Trial Pit No** 6
Site 30 Ferncroft Avenue, London NW3 7PH	**Job Number** J20268
Client Mr Dukagjin Lipa	**Sheet** 1/2
Engineer Mint Structures	**Dates** 07/12/2020

Excavation Method	**Dimensions**	**Ground Level (mOD)**	**Location**
Manual	800 x 300 x 700	97.72	Southeastern eleavtion

PLAN:



SECTION A-A:



Remarks:	Scale: 1:20
All dimensions in millimetres Sides of trial pit remained stable during excavation Groundwater not encountered	Logged by: ML

GEA	www.gea-ltd.co.uk Herts | 01727 824666 Notts | 01509 674888	**Trial Pit No** 6

Site 30 Ferncroft Avenue, London NW3 7PH

Client Mr Dukagjin Lipa

Engineer Mint Structures

Job Number J20268

Sheet 2/2

Dates 07/12/2020

Excavation Method	**Dimensions**	**Ground Level (mOD)**	**Location**
Manual	800 x 300 x 700	97.72	Southeastern eleavtion

Remarks:

All dimensions in millimetres

Sides of trial pit remained stable during excavation

Groundwater not encountered

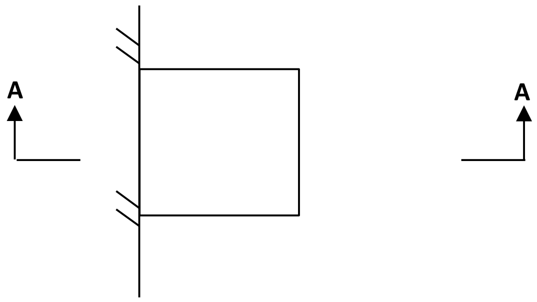
Scale: 1:20

Logged by: ML

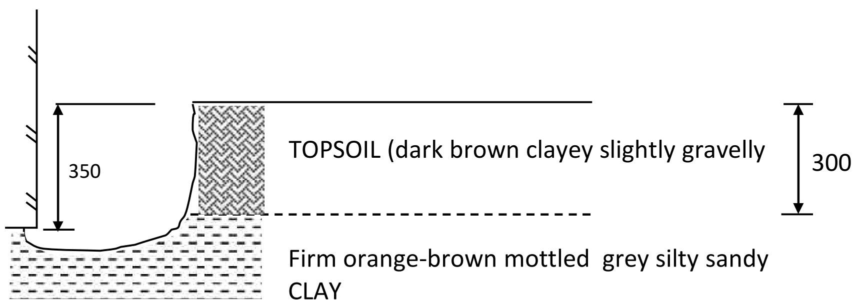
GEA	www.gea-ltd.co.uk Herts | 01727 824666 Notts | 01509 674888	**Trial Pit No** 7
Site 30 Ferncroft Avenue, London NW3 7PH		**Job Number** J20268
Client Mr Dukagjin Lipa		**Sheet** 1/1
Engineer Mint Structures		**Dates** 04/12/2020

Excavation Method	**Dimensions**	**Ground Level (mOD)**	**Location**
Manual	400 x 400 x 350	97.72	Southeastern elevation of terrace structure

PLAN:



SECTION A-A:



Remarks: All dimensions in millimetres Sides of trial pit remained stable during excavation Groundwater not encountered	Scale: 1:20 Logged by: ML

LOAD TAKEDOWN 1 - preliminary area loading

Length of Walls - $L_w := 22\ m$; *Span across site -* $L_{pw} := 11.5\ m$

(*i*) Brickwork - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 2.5\ m$; *Wall width -* $t := 225\ mm$

Wall loading - $w = 4.63\ kN \cdot m^{-2}$

$i_p := w \cdot h = 11.58\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p$

$G_k = 11.58\ kN \cdot m^{-1}$

LOAD TAKEDOWN 2 - preliminary area loading

Length of Walls - $L_w := 16\ m$; *Span front to rear -* $L_{pw} := 9.0\ m$

Vertical Loads:

(*i*) Roof -

Permanent Load

$i_p := \left(1.21\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 5.45\ kN \cdot m^{-1}$

Imposed Load

$i_I := \left(0.75\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 3.38\ kN \cdot m^{-1}$

(*ii*) Floor - No. floors - $n := 3$

Permanent Load

$ii_p := n \cdot \left(0.52\ kN \cdot m^{-2}\ L_{pw}\right) \cdot 0.5 = 7.02\ kN \cdot m^{-1}$

Imposed Load

$ii_I := n \cdot \left(1.5\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 20.25\ kN \cdot m^{-1}$

(*iii*) Partitions - No. storeys - $n := 3$

Permanent Load

Ave. Wall height - $h := 3.0\ m$

$iii_p := \left(\left(\left(0.75\ kN \cdot m^{-2} \cdot h\right) \cdot L_{pw} \cdot 0.5\right) \cdot L_w^{-1}\right) \cdot 0.25 = 0.16\ kN \cdot m^{-1}$

NOTE: Within this calculation party wall brickwork is assumed to 'step' at first floor level from 330mm back to 215mm

(*iv*) Brickwork upper - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 5.5\ m$; *Wall width -* $t := 225\ mm$

Wall loading - $w = 4.63\ kN \cdot m^{-2}$

$iv_p := w \cdot h = 25.47\ kN \cdot m^{-1}$

(*v*) Brickwork lower - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 7.0\ m$; *Wall width -* $t := 330\ mm$

Wall loading - $w = 6.69\ kN \cdot m^{-2}$

$v_p := w \cdot h = 46.83\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p + iii_p + iv_p + v_p$

$G_k = 84.92\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := i_I + ii_I = 23.63\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 108.55\ kN \cdot m^{-1}$

LOAD TAKEDOWN 3 - preliminary area loading

Length of Walls - $L_w := 17.5\ m$; *Span side to side -* $L_{pw} := 6.0\ m$

Vertical Loads:

(i) Roof -

Permanent Load

$i_p := \left(1.21\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 3.63\ kN \cdot m^{-1}$

Imposed Load

$i_I := \left(0.75\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 2.25\ kN \cdot m^{-1}$

(ii) Partitions - No. storeys - $n := 1$

Permanent Load

Ave. Wall height - $h := 3.0\ m$

$ii_p := \left(\left(\left(0.75\ kN \cdot m^{-2} \cdot h\right) \cdot L_{pw} \cdot 0.5\right) \cdot L_w^{-1}\right) \cdot 0.25 = 0.1\ kN \cdot m^{-1}$

NOTE: Within this calculation party wall brickwork is assumed to 'step' at first floor level from 330mm back to 215mm

(iii) Brickwork (cavity) - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 2$

Permanent Load

Wall height - $h := 3.0\ m$; *Wall width -* $t := 300\ mm$

Wall loading - $w = 2.97\ kN \cdot m^{-2}$

$iii_p := w \cdot h = 8.91\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p + iii_p$

$G_k = 12.64\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := i_I = 2.25\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 14.89\ kN \cdot m^{-1}$

LOAD TAKEDOWN 4 - preliminary area loading

Length of Walls - $L_w := 22\ m$; *Span across site -* $L_{pw} := 11.5\ m$

(*i*) Brickwork - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 2.5\ m$; *Wall width -* $t := 225\ mm$

Wall loading - $w = 4.63\ kN \cdot m^{-2}$

$i_p := w \cdot h = 11.58\ kN \cdot m^{-1}$

(*ii*) Slab (say 275mm thk O/A) - Span - $Sp := 5.0\ m$

Permanent Load

$ii_p := \left(\left(24\ kN \cdot m^{-3} \cdot 0.275\ m\right) \cdot Sp\right) \cdot 0.5 = 16.5\ kN \cdot m^{-1}$

Imposed Load

$ii_I := \left(3.0\ kN \cdot m^{-2} \cdot Sp\right) \cdot 0.5 = 7.5\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p$

$G_k = 28.08\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := ii_I = 7.5\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 35.58\ kN \cdot m^{-1}$

LOAD TAKEDOWN 5 - preliminary area loading

Length of Walls - $L_w := 16\ m$; *Span front to rear -* $L_{pw} := 9.0\ m$

Vertical Loads:

(i) Roof -

Permanent Load

$i_p := \left(1.21\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 5.45\ kN \cdot m^{-1}$

Imposed Load

$i_I := \left(0.75\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 3.38\ kN \cdot m^{-1}$

(ii) Floor - No. floors - $n := 2$

Permanent Load

$ii_p := n \cdot \left(0.52\ kN \cdot m^{-2}\ L_{pw}\right) \cdot 0.5 = 4.68\ kN \cdot m^{-1}$

Imposed Load

$ii_I := n \cdot \left(1.5\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 13.5\ kN \cdot m^{-1}$

(iii) Partitions - No. storeys - $n := 3$

Permanent Load

Ave. Wall height - $h := 3.0\ m$

$iii_p := \left(\left(\left(0.75\ kN \cdot m^{-2} \cdot h\right) \cdot L_{pw} \cdot 0.5\right) \cdot L_w^{\ -1}\right) \cdot 0.25 = 0.16\ kN \cdot m^{-1}$

NOTE: Within this calculation party wall brickwork is assumed to 'step' at first floor level from 330mm back to 215mm

(iv) Brickwork upper - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 5.5\ m$; *Wall width -* $t := 225\ mm$

Wall loading - $w = 4.63\ kN \cdot m^{-2}$

$iv_p := w \cdot h = 25.47\ kN \cdot m^{-1}$

(v) Brickwork lower - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 1$

Permanent Load

Wall height - $h := 7.0\ m$; *Wall width -* $t := 330\ mm$

Wall loading - $w = 6.69\ kN \cdot m^{-2}$

$v_p := w \cdot h = 46.83\ kN \cdot m^{-1}$

(vi) Slab (say 275mm thk O/A) **-** Span - $Sp := 6.0\ m$

Permanent Load

$vi_p := \left(\left(24\ kN \cdot m^{-3} \cdot 0.275\ m\right) \cdot Sp\right) \cdot 0.5 = 19.8\ kN \cdot m^{-1}$

Imposed Load

$vi_I := \left(1.5\ kN \cdot m^{-2} \cdot Sp\right) \cdot 0.5 = 4.5\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p + iii_p + iv_p + v_p + vi_p$

$G_k = 102.38\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := i_I + ii_I + vi_I = 21.38\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 123.76\ kN \cdot m^{-1}$

LOAD TAKEDOWN 6 - preliminary area loading

Length of Walls - $L_w := 6.0\ m$; *Span side to side -* $L_{pw} := 6.0\ m$

Vertical Loads:

(*i*) Roof -

Permanent Load

$i_p := \left(1.21\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 3.63\ kN \cdot m^{-1}$

Imposed Load

$i_I := \left(0.75\ kN \cdot m^{-2} \cdot L_{pw}\right) \cdot 0.5 = 2.25\ kN \cdot m^{-1}$

(*ii*) Partitions - No. storeys - $n := 1$

Permanent Load

Ave. Wall height - $h := 3.0\ m$

$ii_p := \left(\left(\left(0.75\ kN \cdot m^{-2} \cdot h\right) \cdot L_{pw} \cdot 0.5\right) \cdot L_w^{-1}\right) \cdot 0.25 = 0.28\ kN \cdot m^{-1}$

NOTE: Within this calculation party wall brickwork is assumed to 'step' at first floor level from 330mm back to 215mm

(*iii*) Brickwork (cavity) - Soil Type (1=Solid ; 2=Cavity) - $W_{typ} := 2$

Permanent Load

Wall height - $h := 3.0\ m$; *Wall width -* $t := 300\ mm$

Wall loading - $w = 2.97\ kN \cdot m^{-2}$

$iii_p := w \cdot h = 8.91\ kN \cdot m^{-1}$

(*iv*) Slab (say 275mm thk O/A) **-** Span - $Sp := 6.0\ m$

Permanent Load

$iv_p := \left(\left(24\ kN \cdot m^{-3} \cdot 0.275\ m\right) \cdot Sp\right) \cdot 0.5 = 19.8\ kN \cdot m^{-1}$

Imposed Load

$iv_I := \left(1.5\ kN \cdot m^{-2} \cdot Sp\right) \cdot 0.5 = 4.5\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p + iii_p + iv_p$

$G_k = 32.62\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := i_I + iv_I = 6.75\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 39.37\ kN \cdot m^{-1}$

LOAD TAKEDOWN 7 - preliminary area loading

Length of Walls - $L_w := 6.0\ m$; *Span side to side -* $L_{pw} := 6.0\ m$

Vertical Loads:

(*i*) **Slab** (say 275mm thk O/A) - Span - $Sp := 4.0\ m$

Permanent Load

$i_p := ((24\ kN \cdot m^{-3} \cdot 0.275\ m) \cdot Sp) \cdot 0.5 = 13.2\ kN \cdot m^{-1}$

Imposed Load

$i_I := (3.0\ kN \cdot m^{-2} \cdot Sp) \cdot 0.5 = 6\ kN \cdot m^{-1}$

(*ii*) **Soil** (say 300mm thk O/A) - Span - $Sp := 4.0\ m$

Permanent Load

$ii_p := ((18\ kN \cdot m^{-3} \cdot 0.3\ m) \cdot Sp) \cdot 0.5 = 10.8\ kN \cdot m^{-1}$

Total line load on wall -

Permanent line load on wall - $G_k := i_p + ii_p$

$G_k = 24\ kN \cdot m^{-1}$

Imposed line load on wall - $Q_k := i_I = 6\ kN \cdot m^{-1}$

Total line load on wall - $\Sigma_L := G_k + Q_k = 30\ kN \cdot m^{-1}$

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 2.4.09

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3400** mm
Prop height	h_{prop} = **3400** mm
Stem thickness	t_{stem} = **475** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1500** mm
Heel length	l_{heel} = **250** mm
Base thickness	t_{base} = **450** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3400** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **0** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

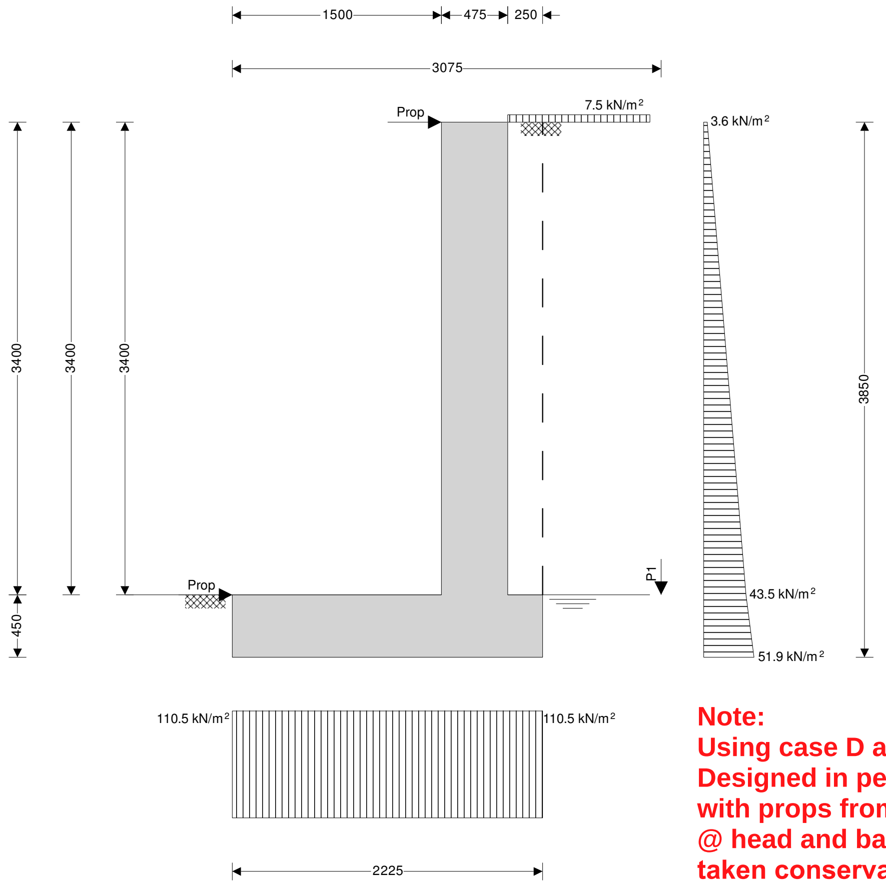
Soil type	Organic clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3

Base soil properties

Soil type	Organic clay
Moist density	γ_{mb} = **18** kN/m^3

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 3075 mm	P_{G1} = **100** kN/m



Note:
Using case D as worst case, Designed in permanent case with props from RC slabs @ head and base. (surcharge taken conservatively as 100kN/m from neighbouring wall)

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2225}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{0}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{3400}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{250}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2100}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3850}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1925}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.615}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1738}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{1.001}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1113}$ mm
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{0.85}$ m^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{2100}$ mm
- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) = \mathbf{1299}$ mm

Partial factors on actions - Table A.3 - Combination 1

Permanent unfavourable action	$\gamma_G = \mathbf{1.35}$
Permanent favourable action	$\gamma_{Gf} = \mathbf{1.00}$
Variable unfavourable action	$\gamma_Q = \mathbf{1.50}$
Variable favourable action	$\gamma_{Qf} = \mathbf{0.00}$

Partial factors for soil parameters – Table A.4 - Combination 1

Angle of shearing resistance — $\gamma_{\phi'}$ = **1.00**

Undrained shear strength — γ_{cu} = **1.00**

Weight density — γ_{γ} = **1.00**

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.325**

Coefficient of friction to front of wall — K_{fb} = **0.325**

Coefficient of friction beneath base — K_{fbb} = **0.325**

Active pressure coefficient — K_A = **0.483**

Passive pressure coefficient — K_P = **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem — $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem}$ = **54.5** kN/m

Wall base — $F_{base} = \gamma_G \times A_{base} \times \gamma_{base}$ = **33.8** kN/m

Surcharge load — $F_{sur\_v} = \gamma_Q \times Surcharge_Q \times l_{heel}$ = **1.9** kN/m

Line loads — $F_{P\_v} = \gamma_G \times P_{G1}$ = **135** kN/m

Moist retained soil — $F_{moist\_v} = \gamma_G \times A_{moist} \times \gamma_{mr}$ = **20.7** kN/m

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **245.8** kN/m

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = K_A \times \gamma_Q \times Surcharge_Q \times h_{eff}$ = **13.9** kN/m

Saturated retained soil — $F_{sat\_h} = \gamma_G \times K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **0.5** kN/m

Water — $F_{water\_h} = \gamma_G \times \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **1.3** kN/m

Moist retained soil — $F_{moist\_h} = \gamma_G \times K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **85.8** kN/m

Total — $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h}$ = **101.6** kN/m

Moments on wall

Wall stem — $M_{stem} = F_{stem} \times x_{stem}$ = **94.7** kNm/m

Wall base — $M_{base} = F_{base} \times x_{base}$ = **37.6** kNm/m

Surcharge load — $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **-22.9** kNm/m

Line loads — $M_P = \gamma_G \times P_{G1} \times p_1$ = **415.1** kNm/m

Saturated retained soil — $M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = **-0.1** kNm/m

Water — $M_{water} = -F_{water\_h} \times x_{water\_h}$ = **-0.2** kNm/m

Moist retained soil — $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h}$ = **-68.1** kNm/m

Total — $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **456.1** kNm/m

Check bearing pressure

Propping force to stem — $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h})$ = **-47.4** kN/m

Propping force to base — $F_{prop\_base} = F_{total\_h} - F_{prop\_stem}$ = **149.1** kN/m

Moment from propping force — $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base})$ = **-182.7** kNm/m

Distance to reaction — $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v}$ = **1113** mm

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

Loaded length of base — $l_{load} = l_{base}$ = **2225** mm

Bearing pressure at toe — $q_{toe} = F_{total\_v} / l_{base}$ = **110.5** kN/m^2

Bearing pressure at heel — $q_{heel} = F_{total\_v} / l_{base}$ = **110.5** kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.267}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Partial factors on actions - Table A.3 - Combination 2

Permanent unfavourable action $\gamma_G = \mathbf{1.00}$

Permanent favourable action $\gamma_{Gf} = \mathbf{1.00}$

Variable unfavourable action $\gamma_Q = \mathbf{1.30}$

Variable favourable action $\gamma_{Qf} = \mathbf{0.00}$

Partial factors for soil parameters – Table A.4 - Combination 2

Angle of shearing resistance $\gamma_{\phi'} = \mathbf{1.25}$

Undrained shear strength $\gamma_{cu} = \mathbf{1.40}$

Weight density $\gamma_\gamma = \mathbf{1.00}$

Soil coefficients

Coefficient of friction to back of wall $K_{fr} = \mathbf{0.325}$

Coefficient of friction to front of wall $K_{fb} = \mathbf{0.325}$

Coefficient of friction beneath base $K_{fbb} = \mathbf{0.325}$

Active pressure coefficient $K_A = \mathbf{0.483}$

Passive pressure coefficient $K_P = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem} = \mathbf{40.4}$ kN/m

Wall base $F_{base} = \gamma_G \times A_{base} \times \gamma_{base} = \mathbf{25}$ kN/m

Surcharge load $F_{sur\_v} = \gamma_Q \times Surcharge_Q \times l_{heel} = \mathbf{1.6}$ kN/m

Line loads $F_{P\_v} = \gamma_G \times P_{G1} = \mathbf{100}$ kN/m

Moist retained soil $F_{moist\_v} = \gamma_G \times A_{moist} \times \gamma_{mr} = \mathbf{15.3}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v} = \mathbf{182.3}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \gamma_Q \times Surcharge_Q \times h_{eff} = \mathbf{12.1}$ kN/m

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{0.4}$ kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{1}$ kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{63.6}$ kN/m

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{77}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{70.2}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{27.8}$ kNm/m

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-19.9}$ kNm/m

Line loads $M_P = \gamma_G \times P_{G1} \times p_1 = \mathbf{307.5}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-0.1}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-0.1}$ kNm/m

Moist retained soil $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{-50.4}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P = \mathbf{335}$ kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h}) = \mathbf{-34.3}$ kN/m

Propping force to base	$F_{prop\_base} = F_{total\_h} - F_{prop\_stem} =$ **111.4** kN/m
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) =$ **-132.2** kNm/m
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} =$ **1113** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **0** mm
Loaded length of base	$l_{load} = l_{base} =$ **2225** mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} =$ **81.9** kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} =$ **81.9** kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.708**

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 2.4.09

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} =$ **30** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} =$ **37** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2
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/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.0** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2
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

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 1696 mm

Depth of section — $h =$ **475** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	$M =$ **16.6** kNm/m
Depth to tension reinforcement	$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 =$ **415** mm
	$K = M / (d^2 \times f_{ck}) =$ **0.003**
	$K' =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d =$ **394** mm
Depth of neutral axis	$x = 2.5 \times (d - z) =$ **52** mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{97}$ 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 — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{625}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sfM.max} = 0.04 \times h = \mathbf{19000}$ mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.622}$

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

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 — $M_{sls} = \mathbf{11.3}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z) = \mathbf{28.6}$ N/mm²

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2) = \mathbf{141042}$ mm²/m

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

Reinforcement ratio — $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = \mathbf{0.007}$

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 — $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} = \mathbf{518}$ mm

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

$w_k = \mathbf{0.044}$ mm

$w_k / w_{max} = \mathbf{0.148}$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section — $h = \mathbf{475}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{36}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{417}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.007}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 – 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{396}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{52}$ mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{209}$ 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 — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{628}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h = \mathbf{19000}$ mm²/m

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

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

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **24.9** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **62.5** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **140958** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **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	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **551** 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 = **0.103** mm
	w_k / w_{max} = **0.344**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **62** 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	$\rho_l = \min(A_{sf.prov} / d, 0.02)$ = **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
	$V / V_{Rd.c}$ = **0.352**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	h = **475** mm

Rectangular section in shear - Section 6.2

Design shear force	V = **18.2** 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	$\rho_l = \min(A_{sf1.prov} / d, 0.02)$ = **0.001**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **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
	$V / V_{Rd.c}$ = **0.103**

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)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **475** mm^2/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** mm^2/m

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

Check base design at toe

Depth of section — h = **450** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **107.2** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **365** mm

$K = M / (d^2 \times f_{ck})$ = **0.027**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **347** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **46** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z)$ = **711** 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 — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **550** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h$ = **18000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.453**

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

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} = **79.3** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **145.7** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **134792** mm²/m

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

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.012**

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 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **547** 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 = **0.239** mm

w_k / w_{max} = **0.796**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **142.9** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.740**

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.440** N/mm²

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

$V_{Rd.c}$ = **178.8** kN/m

$V / V_{Rd.c}$ = **0.799**

PASS - Design shear resistance exceeds design shear force

Rectangular section in shear - Section 6.2

Design shear force V = **1.6** kN/m

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

$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.740**

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

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.440** N/mm²

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

$V_{Rd.c}$ = **160.6** kN/m

$V / V_{Rd.c}$ = **0.010**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

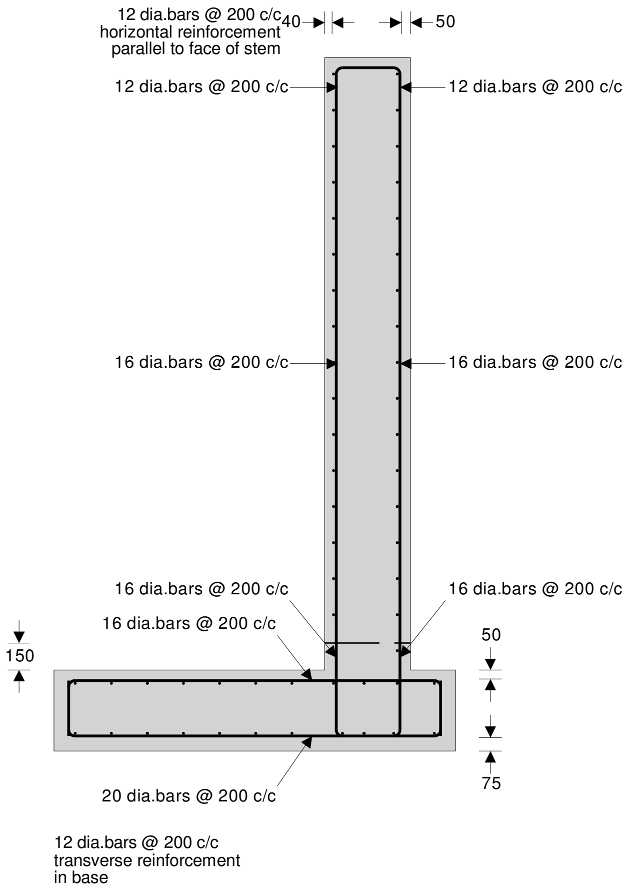
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **314** mm²/m

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

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

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

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



RC BASEMENT SLAB UPLIFT CHECK

NOTE: Slab designed as a simply supported one-way spanning panel, in 1m strips. Because the water level is taken at ground level in all other calculations a partial safely factor of 1.2 is applied to Gound water force (from BS8110-1 table 2.1).
The slab self weight is taken as the most conservative condition i.e. a partial safety factor of 1.0.

DESIGN DATA *[taken from BS8002:1994, BS8110, BS648]* **:**

Typical material weights:

Concrete [normal reinforced] - (unit load) : $W_{conc} := 24\ kN \cdot m^{-3}$

Water - (Bulk density) : $W_{water} := 9.81\ kN \cdot m^{-3}$

Concrete and Reinforcement specifications:

Characteristic tensile strength of reinforcement: $f_y := 500\ N \cdot mm^{-2}$

Characteristic compressive cube strength of concrete : $f_{cu} := 35\ N \cdot mm^{-2}$

Cover to reinforcement: $c_{cov} := 50\ mm$

Slab/ Wall dimensions:

Wall height: $H := 3.6\ m$

Existing undercroft depth (assumed): $F_d := 0.0\ m$

Slab span (7.0m span assumed to split by mid-span ground beam in detailed design): $l := 3.50\ m$

Slab strip width: $b := 1.0\ m$

Assumed data:

Basic Span/ Effective depth ratio : $r := 20$

Initial assumed depth modifaction factor: $m.f := 1.4$

Assumed diameter of reinforcing bars (B1131 mesh): $\phi := 12\ mm$

SEE FOLLOWING PAGES FOR ONE WAY SLAB UPLIFT CHECK CACULATIONS.

SLAB UPLIFT CALCULATION

Depth of slab and main steel area

Over all depth of slab , h:

Minimum effective depth -

$d_{min} := \dfrac{l}{r \cdot m.f} = 125\ \mathbf{mm}$

Slab thickness = $S_{thk} := 300\ \mathbf{mm}$

Effective depth , d=

$d := S_{thk} - \dfrac{\phi}{2} - c_{cov} = 244\ \mathbf{mm}$

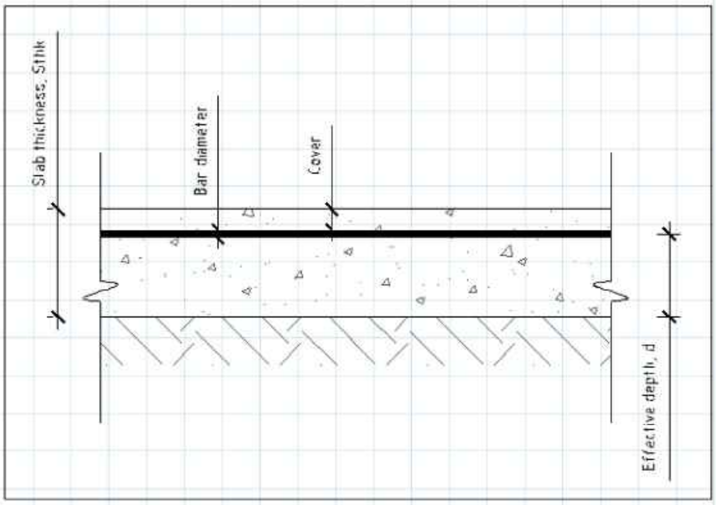


Fig 1 - Showing reinforcement in the top of the slab.

Loading:

Dead Loads (gk):

Self weight of slab: $i := W_{conc} \cdot S_{thk} = 7.2\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Finishes (say): $ii := 2.0\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Total Dead Load: $g_k := i + ii = 9.2\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Uplift:

Height of retained water: $H_w := H - F_d = 3.6\ \mathbf{m}$ Factor of safety: $\gamma_w := 1.2$

Height of water taken as ground level minus assumed existing undercroft depth.

Ground Water: $F_w := H_w \cdot W_{water} \cdot \gamma_w = 42.379\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Heave: $F_h := 48\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Net uplift load: $F_{net} := (F_w + F_h) - g_k = 81.18\ \mathbf{kN} \cdot \mathbf{m}^{-2}$

Ultimate load for 1.0m strip, W:

Total imposed load: $W := F_{net} \cdot b \cdot l = 284.13\ \mathbf{kN}$

Design moment: $M := \dfrac{W \cdot l}{8} = 124.31\ \mathbf{kN} \cdot \mathbf{m}$

Ultimate moment: $M_u := 0.156 \cdot f_{cu} \cdot b \cdot d^2 = 325.07\ \mathbf{kN} \cdot \mathbf{m}$

Check: $M_u > M$ $\mathrm{RESULT} = \text{“SO OK”}$

$\therefore$ **SO NO COMPRESSION REINFORCEMENT NEEDED.**

Slab uplift continued:

Main steel design:

$k := \frac{M}{f_{cu} \cdot b \cdot d^2} = 0.06$; $\mu_z := 0.5 + \left(\sqrt{0.25 - \frac{k}{0.9}}\right)$

Check: $\mu_z < 0.95$ $RESULT = \text{"PASS"}$ $\therefore$ $\mu_z = 0.93$

Lever arm, z : $z := \mu_z \cdot d$ $\therefore$ $z = 226.58\ mm$

So area of steel required is :

$As_{req} := \frac{M}{0.95 \cdot f_y \cdot (0.95 \cdot d)} = 1128.97\ mm^2$ [in per meter i.e. mm^2/m]

Reinforcing bar dia. specified : $\phi_{vert} := 12\ mm$ Spacing : $b_{sv} := 100\ mm$

$As_{actual} = 1131\ mm^2$ [mm^2/m]

Check: $\mathbf{As_{actual}} > As_{req}$ $\mathbf{RESULT} = \text{"SO OK"}$

Check minimum steel area, As:

$As_{min} := 13\% \cdot b \cdot S_{thk} = 39000\ mm^2$ so $\frac{As_{min}}{100} = 390\ mm^2\ /m$

Check: $As_{actual} > As_{min}$ $\mathbf{RESULT} = \text{"SO OK"}$

Transverse reinforcement check :

$As_{req} := 20\% \cdot As_{actual}$ $\therefore$ $As_{req} = 226.195\ mm^2$ [mm^2/m]

Transverse reinforcement provided : $As_{act} := 252\ mm^2$ [mm^2/m]

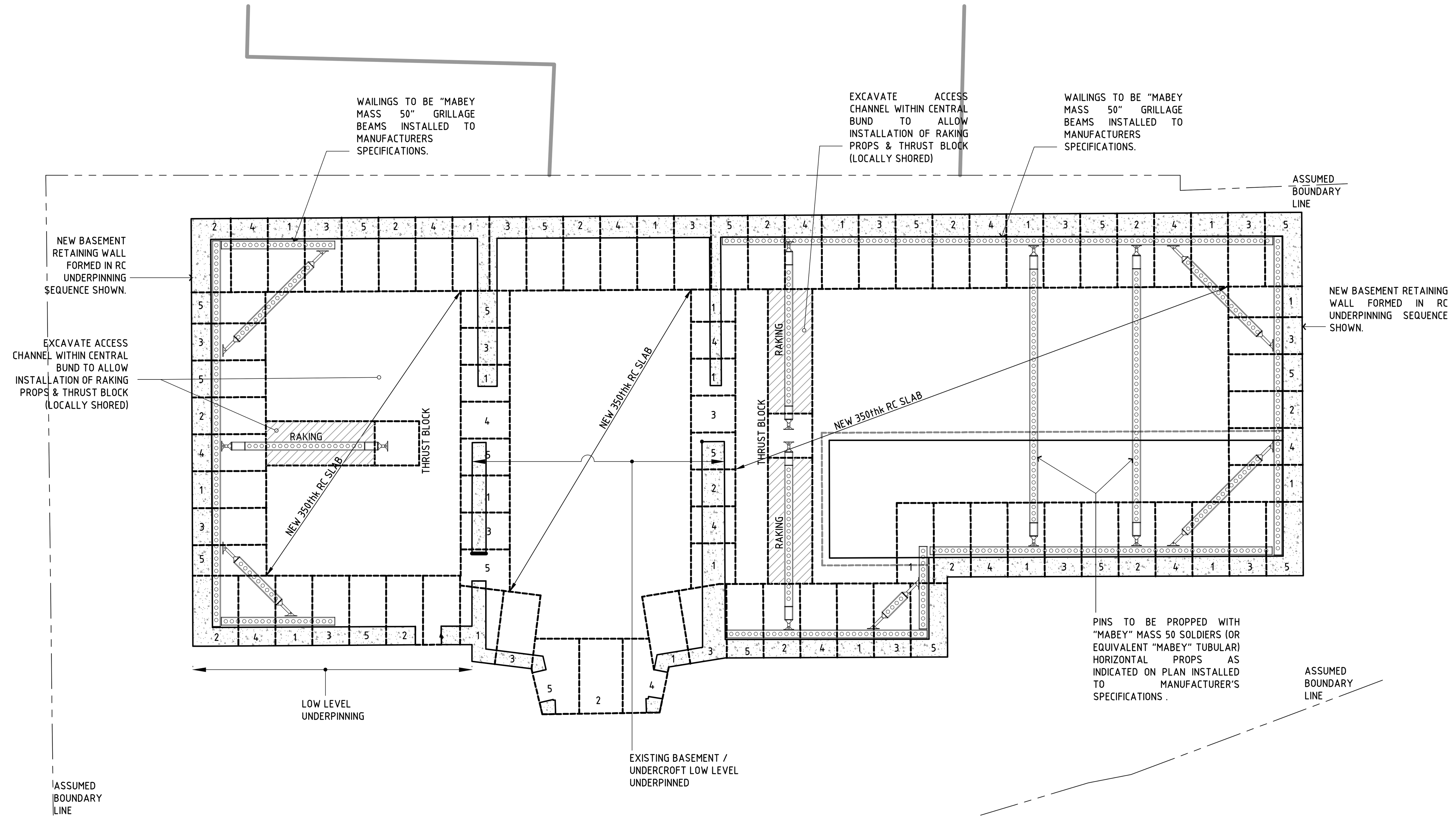
Check : $As_{min} < As_{actual}$ $\mathbf{RESULT} = \text{"SO OK"}$

So provide 350mm thick slab with B1131 Mesh (1131 mm²/m) in the top of the slab.

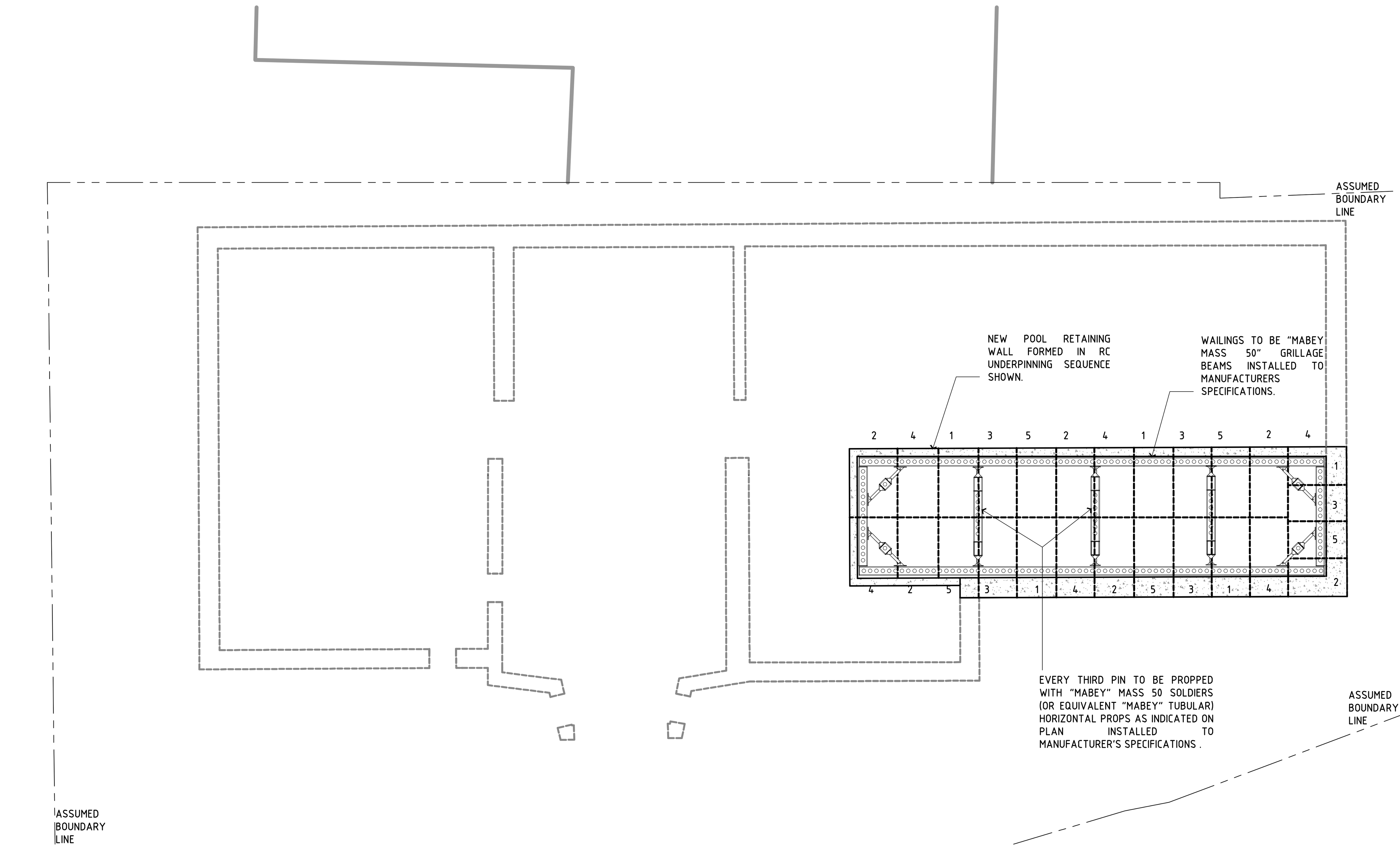
END

SUGGESTED BASEMENT PROPPING PLANS

SCALE 1:100 @ A1



BASEMENT LEVEL PLAN



GROUND LEVEL PLAN

UNDERPINNING GENERAL NOTES

1. THIS DRAWING SHOULD NOT BE SCALED
2. ALL DIMENSIONS AND LEVELS TO BE CONFIRMED BY THE ARCHITECT
3. A SOIL INVESTIGATION IS TO BE CARIED OUT INCLUDING TRIAL PITS AND BOREHOLES PRIOR TO CONFIRMATION OF ENGINEER'S DETAILS
4. NO CONCRETE IS TO BE POURED UNTIL EXCAVATION BOTTOMS HAVE BEEN CHECKED AND APPROVED BY THE LOCAL AUTHORITY
5. THE UNDERSIDE OF EXISTING FOUNDATIONS ARE TO BE THOROUGHLY CLEANED AND ALL LOOSE PARTICLES REMOVED TO RECEIVE DRY PACK.
6. CONCRETE PINS TO BE CONSTRUCTED IN THE ORDER AS SHOWN ON THE DRAWING.
7. ONLY 20% OF WALL IS TO BE OPENED UP AT ANY ONE TIME.
8. DRY PACK MIX (1:3) SULPHATE RESISTING CEMENT: SHARP SAND TO DAMP EARTH CONSISTENCY TO BE THOROUGHLY RAMMED INTO POSITION.
9. ALLOW 48 HOURS FOR DRY PACK TO MATURE BEFORE EXCAVATING ADJOINING AREAS.
10. ALL SOFT MATERIAL IS TO BE REMOVED TO ENSURE A FIRM SOUND FOUNDING MATERIAL TO BE AGREED ON SITE.
11. UNDERPINNING WORKS TO BE IN ACCORDANCE WITH EUROCODE 7, PART 1, BS EN 1997-1
12. THE CONTRACTOR IS RESPONSIBLE FOR THE STABILITY OF ALL EXCAVATION AND THE BUILDING DURING THE COURSE OF THE WORKS AND THEREFORE SHALL HAVE PLANKING AND STRUTTING ON SITE AT ALL TIMES TO COMPLY WITH CURRENT HEALTH AND SAFETY REGULATIONS.
13. DRAINS ENCOUNTERED IN EXCAVATIONS AND ACCESS HOLES ARE TO BE WRAPPED IN MINIMUM 50mm THICKNESS POLYSTYRENE PRIOR TO PLACING CONCRETE IN PIN AND BACK FILLING ACCESS HOLES WITH LEAN MIX CONCRETE WHERE THIS IS REQUIRED
14. FOR GENERAL NOTES SEE DRAWING M20202/GN01 EXCAVATED VERTICAL FACES TO BE SUPPORTED USING NON-DEGRADEABLE SACRIFICIAL TRENCH SHEETING SECURELY PROPPED OFF OPPOSITE FACE.
15. ALL PINS TO BE SUPPORTED AT 3RD HEIGHT FROM BASE OF PIN (FULL HEIGHT TAKEN BETWEEN BTM OF EXCAVATION UP TO GROUND LEVEL). A SECOND ROW OF PROPPING MUST ALSO BE PROVIDED AT THE HEAD OF EACH PIN. ALL PROPS TO EXTEND BACK HORIZONTALLY TO THE CENTRAL EARTH MASS (OR OFF FACE OF OPPOSING PIN/WALL). ALL PROPPING TO BE IN ACCORDANCE WITH CONTRACTORS TEMPORARY WORKS DESIGN & METHOD STATEMENT.
(CORNERS TO BE PROPPED AS ABOVE WITH DIAGONALS ACROSS ADJACENT PINS)
16. CONTRACTOR IS ENSURE THAT ADEQUATE PROPPING AND SACRIFICIAL SHEETING ARE KEPT ON SITE IN CASE LOOSE SOIL IS ENCOUNTERED
17. ALL UNDERPINNING TO BE FORMED WITH GRADE RC35 CONCRETE, WITH SULPHATE RESISTING CEMENT (SRC) TO BS EN 197-1

NOTES:
1. THIS DRAWING IS INTENDED AS AN INDICATIVE SUGGESTED T.W. SPECIFICATION ONLY AND SHOULD BE READ IN CONJUNCTION WITH THE WRITTEN METHOD STATEMENT.
2. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO PROVIDE A FULL T.W. PACKAGE INCLUDING PROPPING PLAN & T.W. DESIGN CALCULATIONS. THIS DRAWING AND ANY SUPPORTING SKETCH CALCULATIONS SHOULD NOT BE RELIED ON FOR THE REQUIRED T.W. PROVISIONS.
3. ALL CONSTRUCTION DETAILS SHOULD ADHERE TO STRUCTURAL ENGINEER'S AND ARCHITECT'S DETAILS.
4. SEE DRAWING TW02 FOR SUGGESTED METHOD OF PROPPING.
5. UNDER NO CIRCUMSTANCES SHOULD DIGGERS BE USED FOR ANY EXCAVATIONS ALL WORKS TO BE HAND DUG.

THIS DRAWING FORMS PART OF A INITIAL DRAFT SCHEME PRODUCED PRIOR TO ANY STRUCTURAL CALCULATIONS BEING CARRIED OUT BY MINT STRUCTURES, IT IS THEREFORE FOR INDICATIVE PURPOSES ONLY FOR USE SOLE IN SUPPORT OF A PLANNING APPLICATION. ALL STRUCTURAL ELEMENTS SHOWN ARE PRELIMINARY AND SUBJECT TO CHANGE AT FINAL CONSTRUCTION ISSUE STAGE. THIS DRAWING SHOULD NOT BE USED FOR ANY CONSTRUCTION PURPOSES.

DRAWING NOTES
1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT THIRD PARTY ARCHITECTS & OTHER SPECIALISTS' DRAWINGS AND SPECIFICATIONS.
2. THIS DRAWING SHOULD NOT BE SCALED IN EITHER PAPER OR DIGITAL FORMAT.
3. ALL DIMENSIONS AND LEVELS TO BE CONFIRMED BY THE ARCHITECT.
4. ANY DISCREPANCIES IN DRAWINGS OR DETAILS TO BE IMMEDIATELY REPORTED TO MINT STRUCTURES.
5. THIS DRAWING REMAINS THE PROPERTY OF MINT STRUCTURES AND MUST NOT BE REPRODUCED WITH OUT PRIOR WRITTEN CONSENT.
6. ALL DETAILS ARE SUBJECT TO BUILDING REGULATIONS APPROVAL.
7. TO ENSURE THIS DRAWING HAS BEEN PRINTED CORRECTLY THE BAR BELOW SHOULD MEASURE 50mm:

0mm 25mm 50mm

8. ALL WORK CARRIED OUT SHOULD ADHERE TO SPECIFIC DRAWING NOTES & GUIDANCE AND COMPLY WITH CURRENT RELEVANT HSE & CDM REGULATIONS TO ENSURE SAFE SITE PRACTICE IS MAINTAINED IN ACCORDANCE WITH DETAILS PRODUCED BY OTHERS.
9. FOR GENERAL NOTES SEE DRAWING: M20202/GN01.
10. ALL DIMENSIONS SHOWN ARE IN mm U.N.O.

DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
JAN 21	A	ACCESS CHANNELS ADDED	LS
FEB 21	B	RAKING PROPPING REVISED	LS
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

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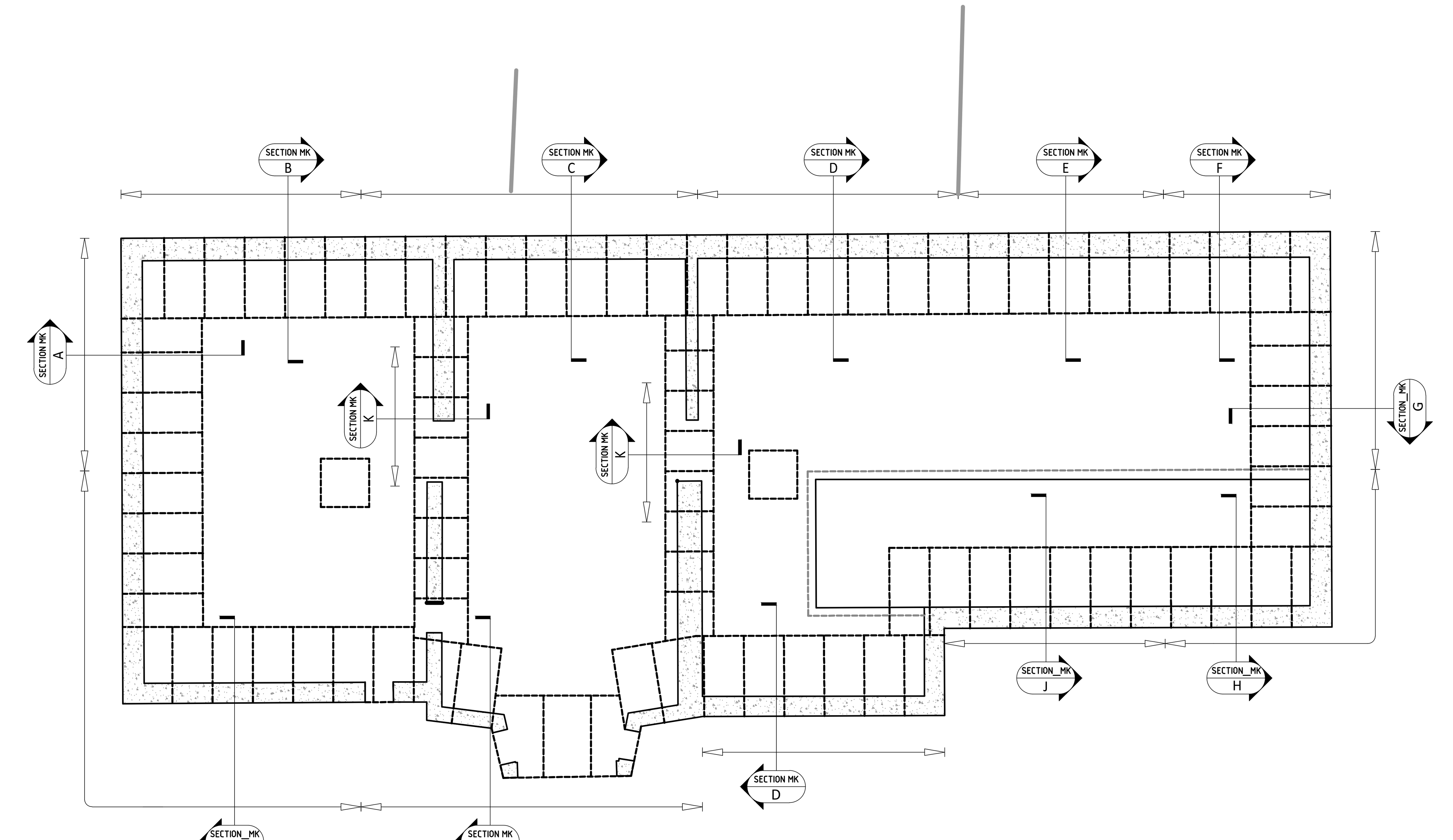
DRAWING TITLE
SUGGESTED BASEMENT PROPPING PLANS

SCALE	DRAWN
1:100 @ A1	LS
	ENGINEER LS
DATE JAN 2021	CHECKED --
DRAWING No. M20202/ B1	REVISION B

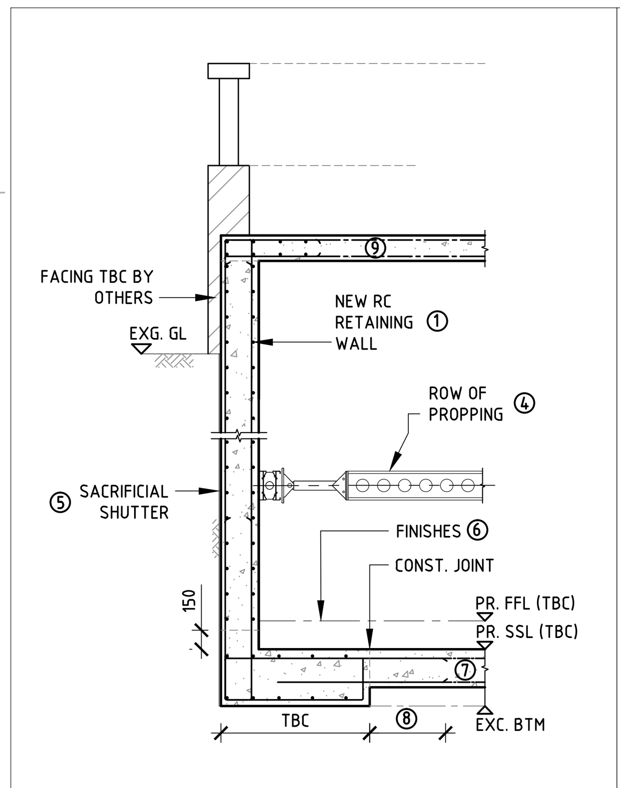


BASEMENT SECTIONAL KEY PLAN
SCALE 1:125

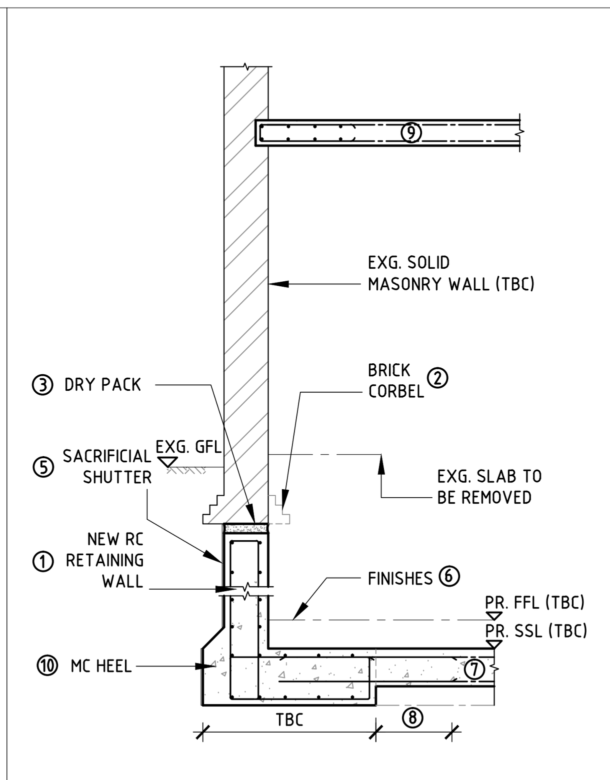
PIN SECTION NOTES

(NUMBERED NOTES BELOW REFERENCE DIRECTLY TO MARKS INDICATED ON PIN SECTIONS A–G)

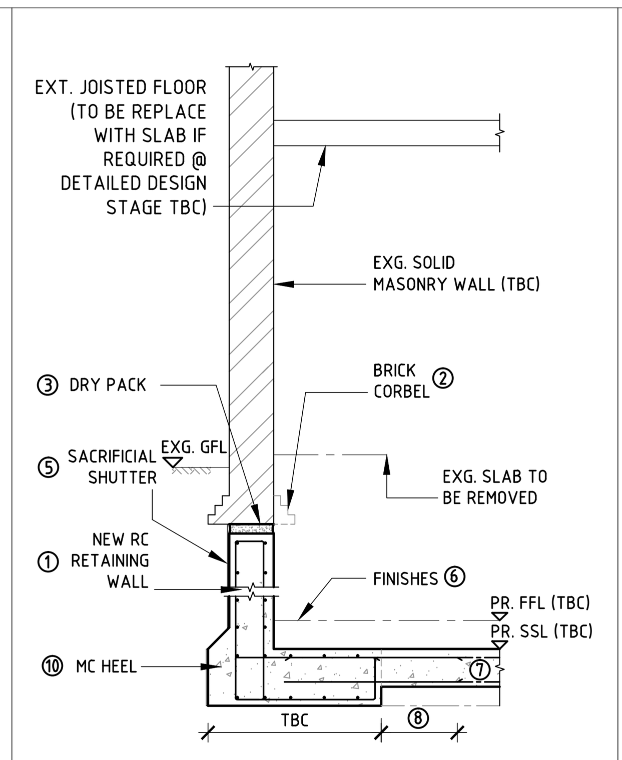
1. NEW R.C UNDERPINNING RETAINING WALL CAST IN MAX 1000mm WIDE SECTIONS IN SEQUENCE SHOWN ON PLAN S001.
2. CAREFULLY REMOVE EXTG PROJECTIONS USING HAND TOOLS ONLY LEAVING EXTG WALL FLUSH WITH FACE OF RC UNDERPINNING UNDER (REMOVAL TO BE CARRIED OUT AFTER COMPLETION OF RC PIN AND DRY PACK HAS SUFFICIENTLY CURED).
3. 75mm 1:3 DRY PACK WELL RAMMED INTO POSITION BETWEEN HEAD OF PIN AND UNDERSIDE OF EXTG FOUNDATION (DRY PACK IS TO BE INSTALLED AFTER EACH INDIVIDUAL PIN HAS CURED SUFFICIENTLY.)
4. NEW HIGH LEVEL RETAINING WALL / UNDERPINNING TO BE HORIZONTALLY PROPPED AS SHOWN.
5. PERMANENT SACRIFICIAL SHUTTER POSITIONED AT BACK FACE OF UNDERPINNING FORMED WITH CEMENTITIOUS 'HARDIE BACKER 500' BOARD TO ENSURE NEW UNDERPINNING DOES NOT ENCROACH BEYOND LINE OF EXTG WALL FACE OVER. SHUTTER RELEASE AGENT ("CEMENTONE" MOULD OIL OR SIMILAR) TO BE APPLIED TO CONTACT FACE OF BOARD TO ENSURE DEBONDING AND EASE OF REMOVAL WHEN REQUIRED.
6. ALL FINISHES, INSULATION, DPM, WATERPROOFING, DRAINAGE ETC. TO ARCHITECT'S DETAILS
7. BASE SLAB REINFORCED WITH BARS AS DESIGN @ DETAILED DESIGN STAGE.
8. STARTER BAR PROJECTION.
9. GROUND FLOOR LEVEL RC SLAB.
10. SHAPED MASS CONCRETE HEEL TO EXTEND UP TO LINE OF CORBELLED FOUNDATION ABOVE.



PIN SECTION A
SCALE 1:50

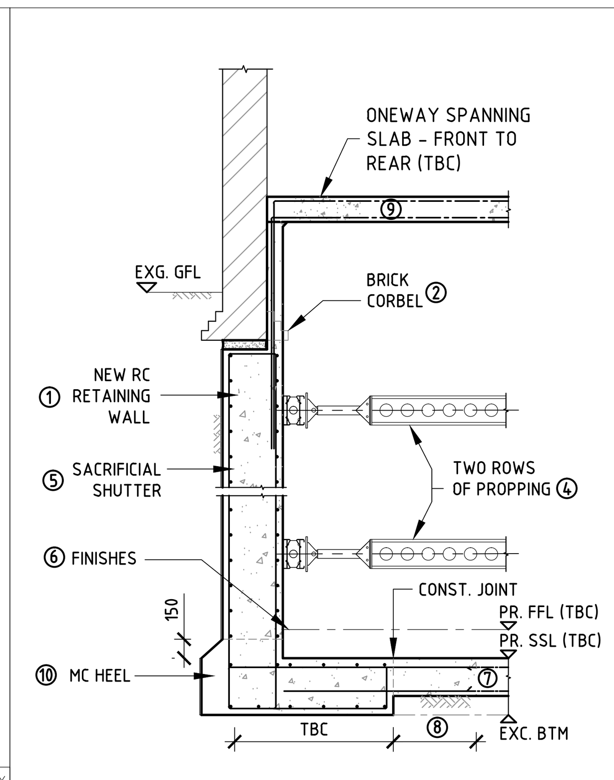


PIN SECTION B
SCALE 1:50

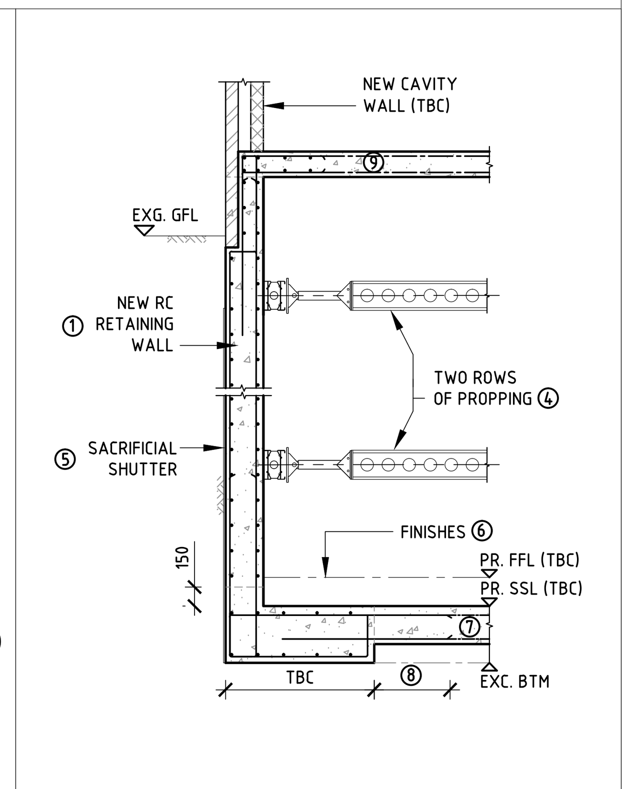


NOTE THAT GROUND LEVELS VARY AS THE SIDEWAY PATH MOVES BACK, PARALLEL TO THE FLANK WALL. THE DIFFERENCE ACROSS THIS CASE HOWEVER IS MINIMAL AND THE SECTION SHOWN IS SUITABLE FOR THE LENGTH OF WALL UNTIL CASE N° 5 IS REACHED.

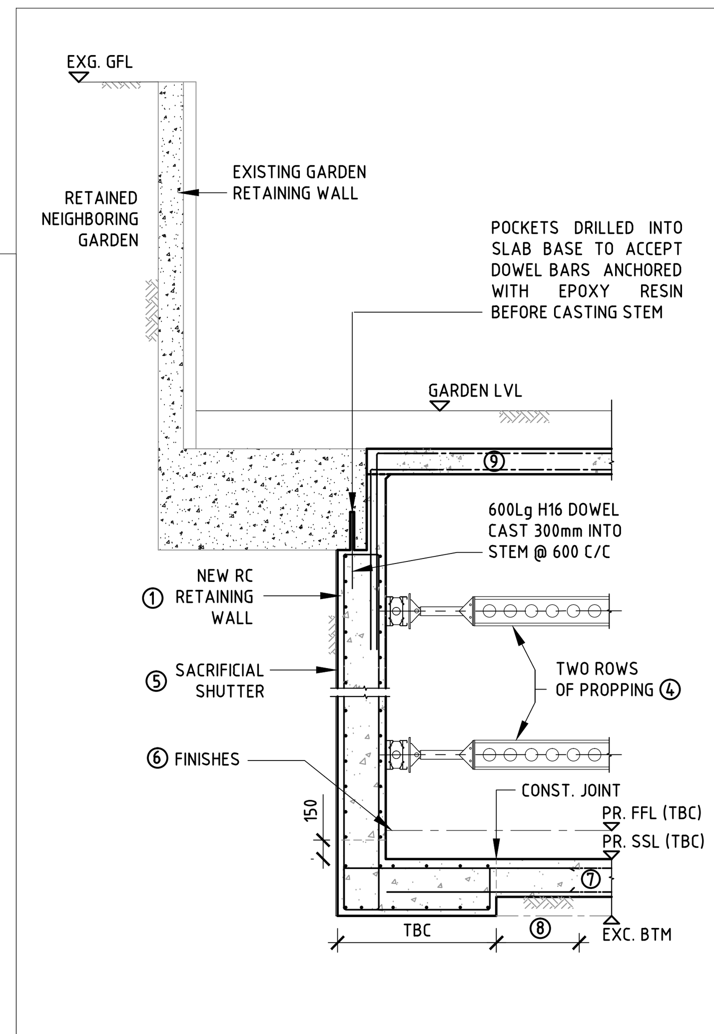
PIN SECTION C
SCALE 1:50



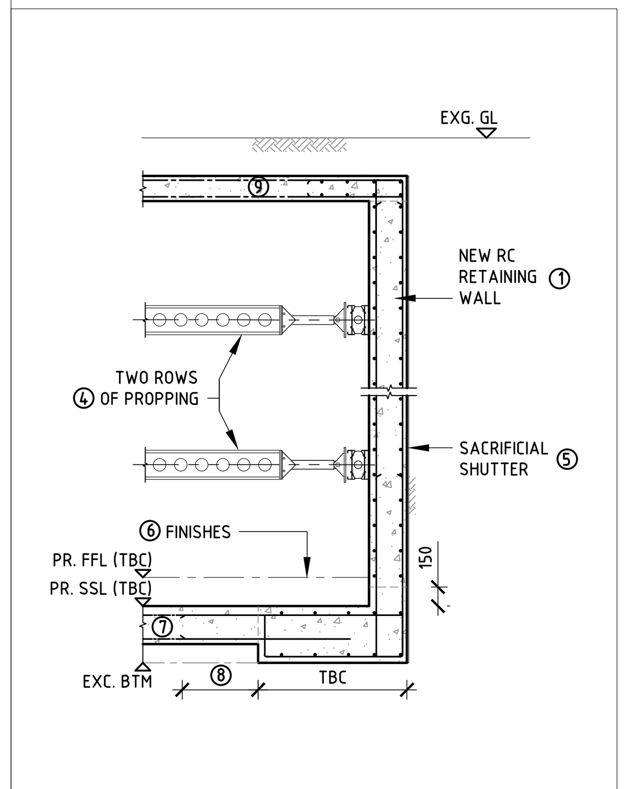
PIN SECTION D
SCALE 1:50



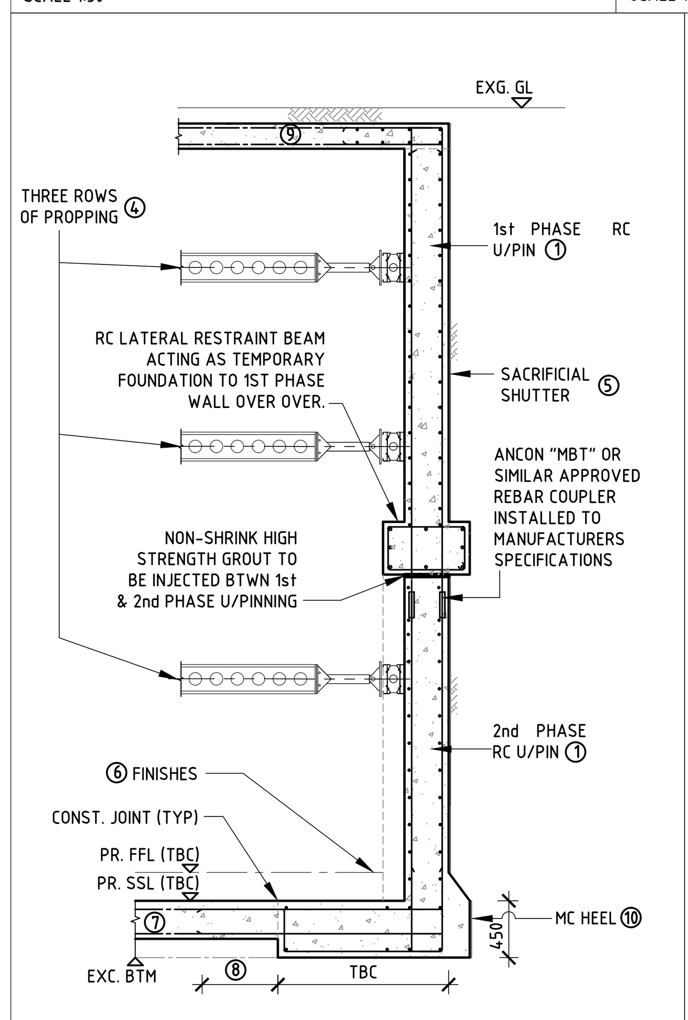
PIN SECTION E
SCALE 1:50



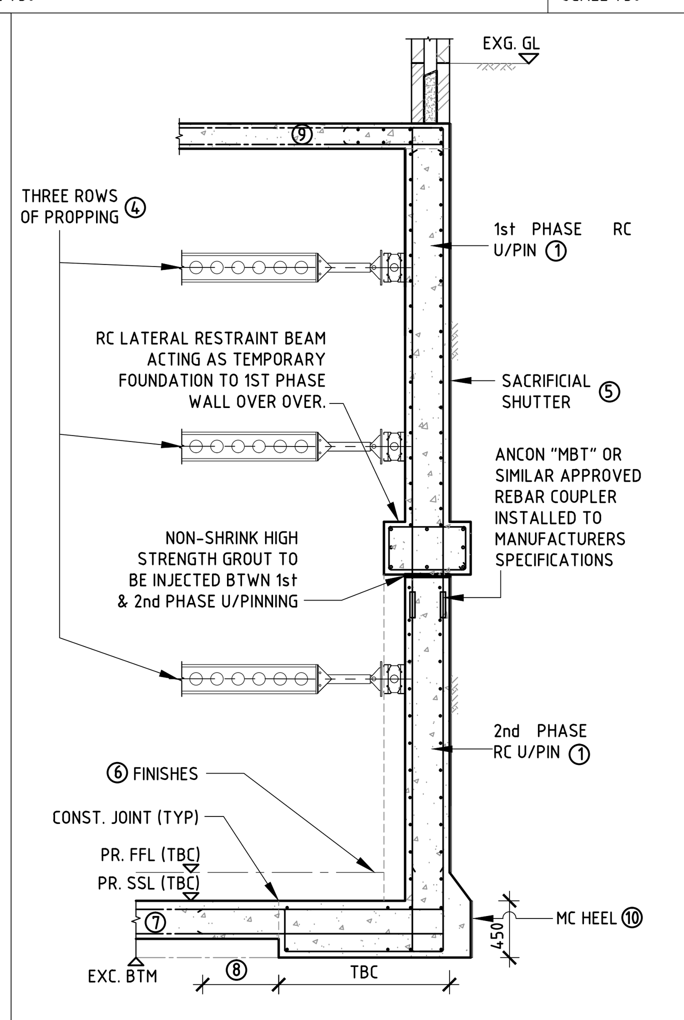
PIN SECTION F
SCALE 1:50



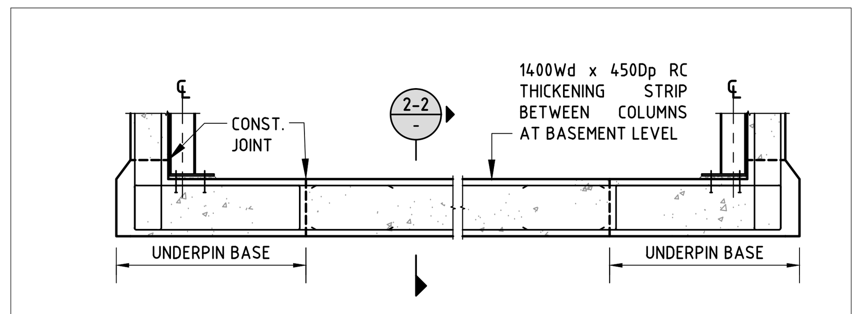
PIN SECTION G
SCALE 1:50



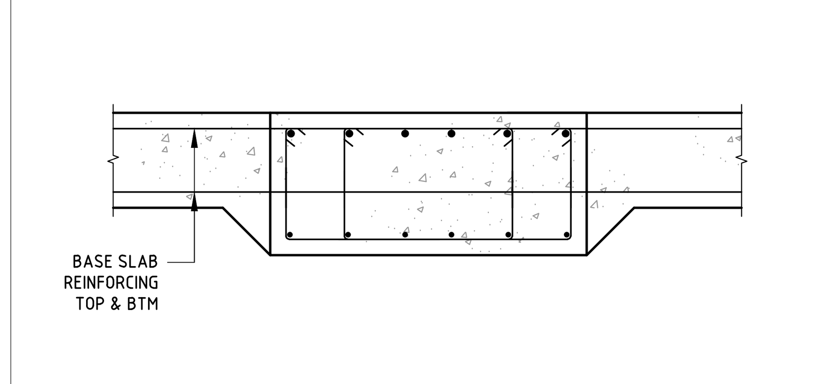
PIN SECTION H
SCALE 1:50



PIN SECTION J
SCALE 1:50

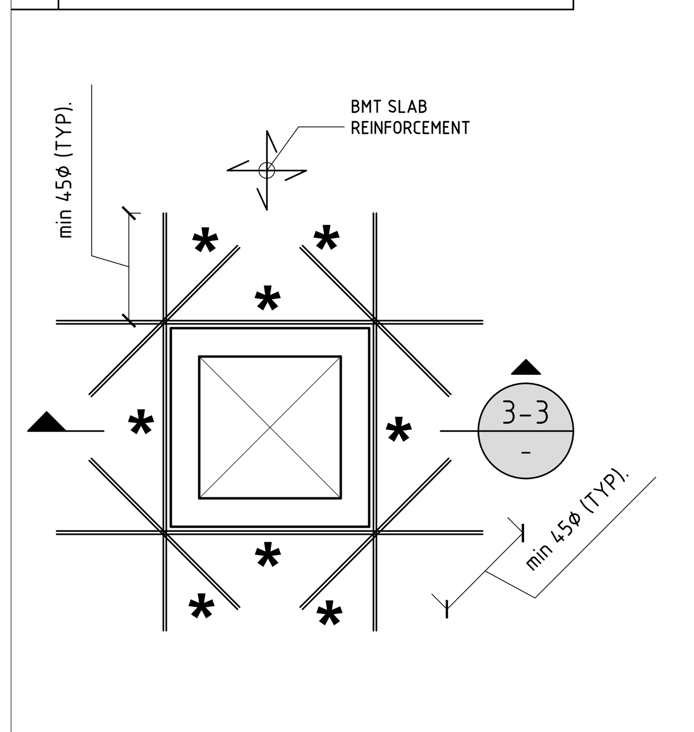


BASEMENT SLAB THICKENING (SECTION K)
SCALE 1:50



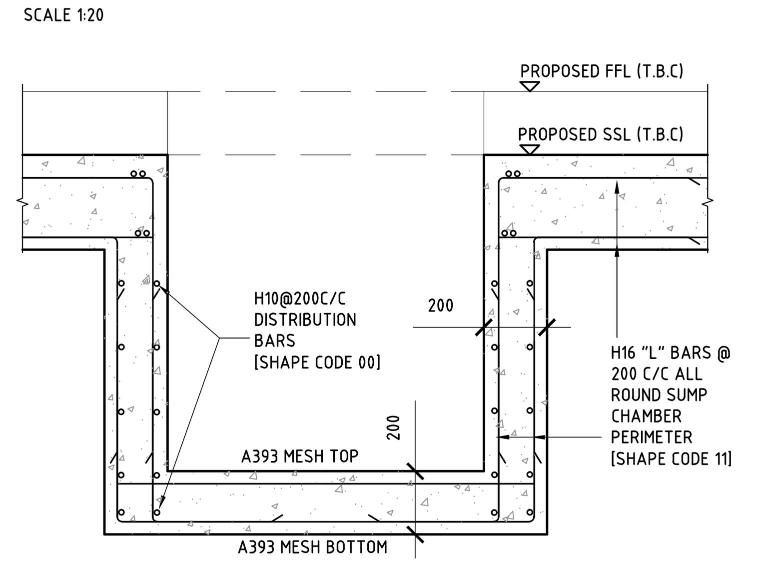
SECTION 2-2
SCALE 1:20

* 2No. H16 BARS TOP AND BOTTOM TIED IN TO SLAB MESHING



SUMP CHAMBER LOCAL REINFORCMENT DETAIL

SECTION 3-3 TYPICAL SUMP DETAIL
SCALE 1:20



GENERAL RC NOTES

LEVELS SHOWN ON RC SECTIONS ARE DERIVED DIRECTLY FROM THE ARCHITECTURAL SCHEME AND MUST BE CONFIRMED WITHIN THE FINAL ARCHITECT'S PACKAGE.

MINIMUM LAP LENGTH TO BE 40xØ OF THE SMALLER LAPPED BAR.

PIN EXCAVATION MUST BE STAGGERED FOLLOWING SEQUENCE SHOWN.

UNDER NO CIRCUMSTANCE SHOULD TWO ADJACENT PINS BE EXCAVATED SIMULTANEOUSLY, THE CONTRACTOR IS TO ENSURE THAT A PIN HAS CURED SUFFICIENTLY BEFORE OPENING A NEIGHBORING PIN.

WHERE RETAINING WALL CORNERS ARE FORMED, DIAGONAL PROPPING ACROSS ADJACENT PINS IS TO BE INSTALLED.

ALL PROPPING AND TEMPORARY WORKS TO CONTRACTOR'S DESIGN AND METHOD STATEMENT.

THE SECTIONS SHOWN ARE INDICATIVE DRAWINGS ONLY. REINFORCEMENT AND CONSTRUCTION DETAILS AND DRAWINGS TO BE PROVIDED IN FULL AT FINAL CONSTRUCTION DRAWING STAGE TO BE REQUESTED BY CONTRACTOR PRIOR TO CONSTRUCTION.

ALL DIMENSIONS SHOULD BE OBTAINED & VERIFIED ON SITE BEFORE CONSTRUCTION BEGINS.

ANY VOIDS BEHIND UNDERPINNING TO BE FILLED AND PACKED WITH DRY SAND PRIOR TO CASTING UNDERPINS.

50mm COVER TO BACK FACE OF UNDERPINNING. 70mm COVER TO BOTTOM OF PINS. 35mm COVER TO INTERNAL FACES.

ALL DIMENSIONS SHOULD BE OBTAINED & VERIFIED ON SITE BEFORE CONSTRUCTION BEGINS.

UNDER NO CIRCUMSTANCES SHOULD DIGGERS BE USED FOR ANY EXCAVATIONS ALL WORKS TO BE HAND DUG.

THIS DRAWING FORMS PART OF A INITIAL DRAFT SCHEME PRODUCED PRIOR TO ANY STRUCTURAL CALCULATIONS BEING CARRIED OUT BY MINT STRUCTURES, IT IS THEREFORE FOR INDICATIVE PURPOSES ONLY FOR USE SOLE IN SUPPORT OF A PLANNING APPLICATION. ALL STRUCTURAL ELEMENTS SHOWN ARE PRELIMINARY AND SUBJECT TO CHANGE AT FINAL CONSTRUCTION ISSUE STAGE. THIS DRAWING SHOULD NOT BE USED FOR ANY CONSTRUCTION PURPOSES.

DRAWING NOTES

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2. THIS DRAWING SHOULD NOT BE SCALED IN EITHER PAPER OR DIGITAL FORMAT.
3. ALL DIMENSIONS AND LEVELS TO BE CONFIRMED BY THE ARCHITECT.
4. ANY DISCREPANCIES IN DRAWINGS OR DETAILS TO BE IMMEDIATELY REPORTED TO MINT STRUCTURES.
5. THIS DRAWING REMAINS THE PROPERTY OF MINT STRUCTURES AND MUST NOT BE REPRODUCED WITH OUT PRIOR WRITTEN CONSENT.
6. ALL DETAILS ARE SUBJECT TO BUILDING REGULATIONS APPROVAL.
7. TO ENSURE THIS DRAWING HAS BEEN PRINTED CORRECTLY THE BAR BELOW SHOULD MEASURE 50mm:

   0mm 25mm 50mm
8. ALL WORK CARRIED OUT SHOULD ADHERE TO SPECIFIC DRAWING NOTES & GUIDANCE AND COMPLY WITH CURRENT RELEVANT HSE & CDM REGULATIONS TO ENSURE SAFE SITE PRACTICE IS MAINTAINED IN ACCORDANCE WITH DETAILS PRODUCED BY OTHERS.
9. FOR GENERAL NOTES SEE DRAWING: M20202/GN01.
10. ALL DIMENSIONS SHOWN ARE IN mm U.N.O.

DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
JAN 21	A	SECTIONS B, D ,E REVISED	LS
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

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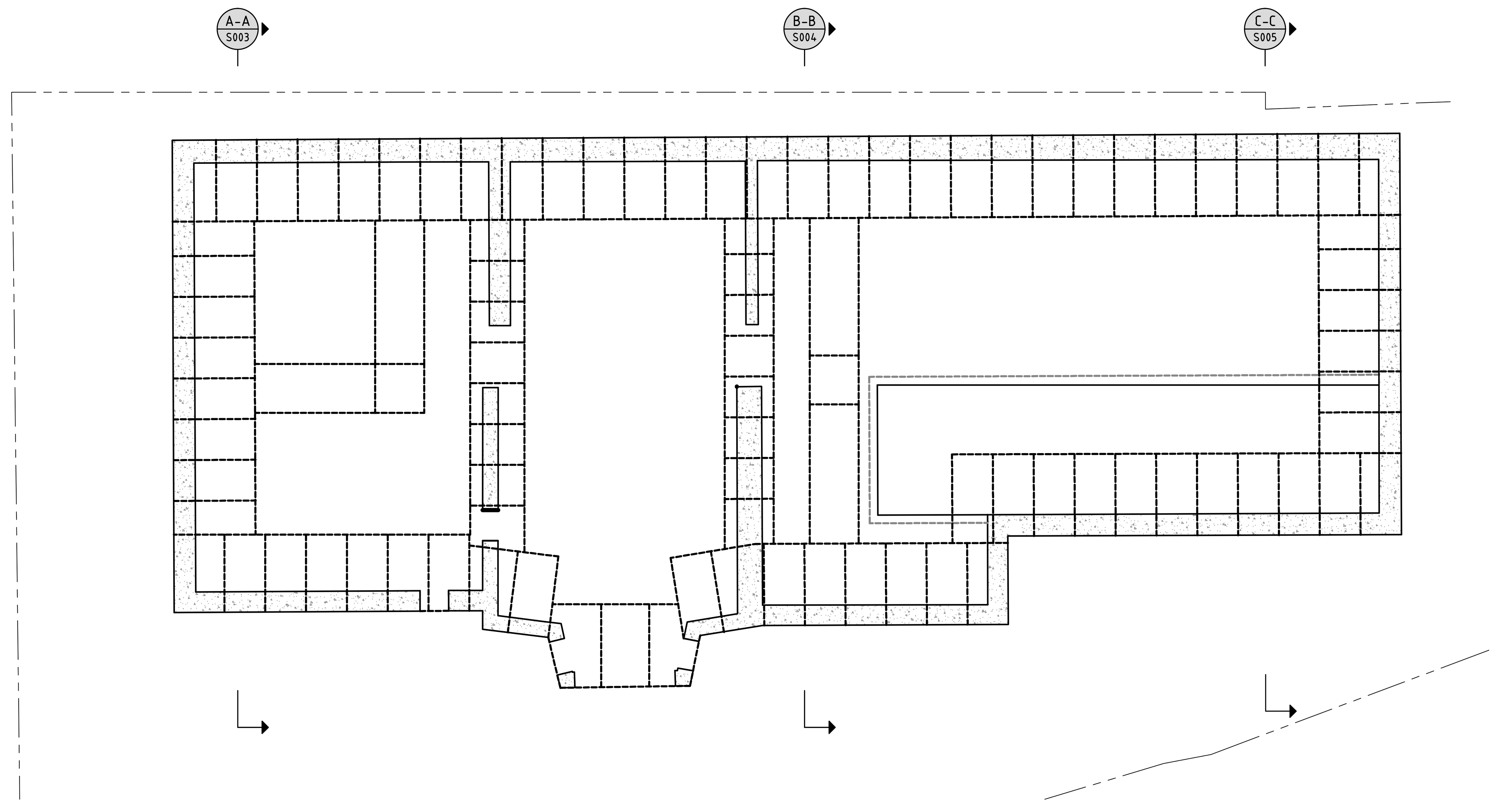
DRAWING TITLE
BASEMENT SECTION AL DETAILS

SCALE	DRAWN
1:50 @ A1	LS
	ENGINEER LS
DATE JAN 2021	CHECKED --
DRAWING No. M20202/ B2	REVISION A

PROP NOTE
ALL WAILINGS TO BE "MABEY MASS" GRILLAGE BEAMS INSTALLED TO MANUFACTURERS SPECIFICATIONS.

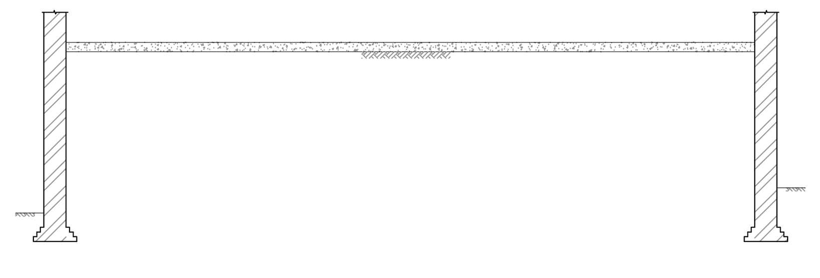
CONSTRUCTION SEQUENCES KEY PLAN

SCALE 1:100

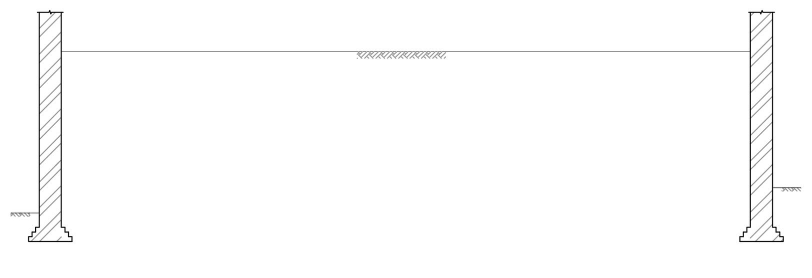


BASEMENT CONSTRUCTION SEQUENCE A-A

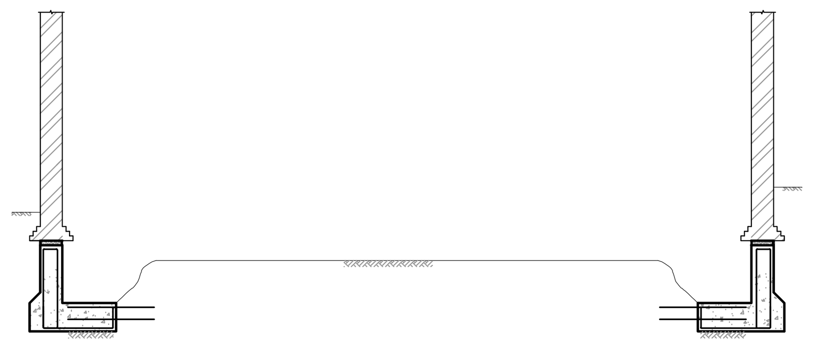
SCALE 1:100



1. EXTG SITE CONDITION.

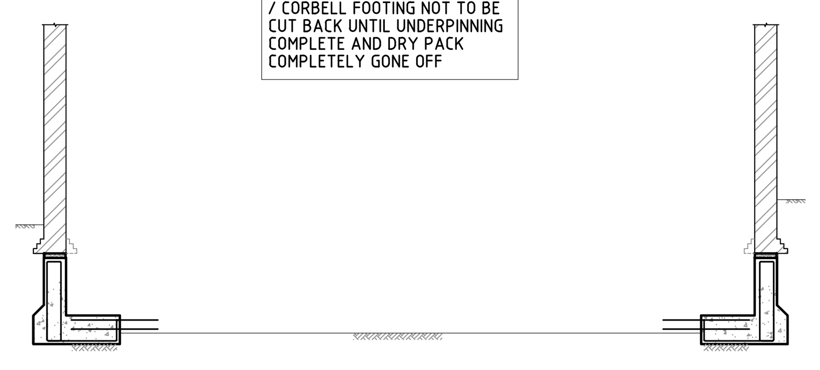


2. REMOVE EXTG FLOOR SLAB.

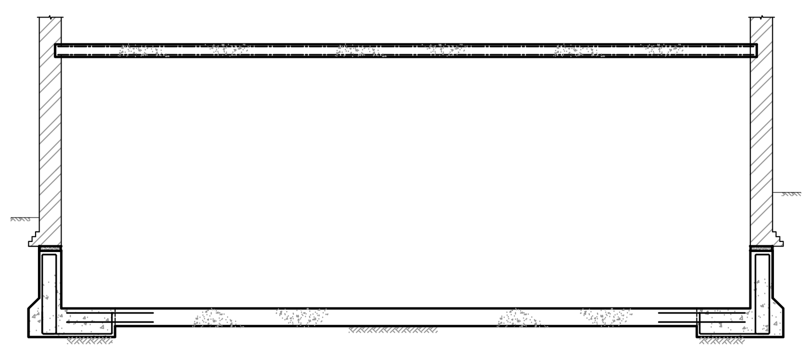


3. EXCAVATE TO FORMATION LEVEL PROVIDING RETENTION SHORING WHERE NEEDED TO EXPOSED FACES OF ACCESS PIT. CAST RC BASE, STEM & AND DRY PACK BTWN EXTG IN ACCORDANCE WITH WRITTEN METHOD STATEMENT.

NOTE:
EXTG PROJECTING FOUNDATION / CORBELL FOOTING NOT TO BE CUT BACK UNTIL UNDERPINNING COMPLETE AND DRY PACK COMPLETELY GONE OFF



4. REDUCE LEVEL OF CENTRAL EARTH MASS EXCAVATING TO BASE SLAB FORMATION LEVEL.



5. CAST RC BASE & GROUND FLOOR SLAB.

NOTES:

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2. CONTRACTOR TO CARRY OUT UNDERPINNING ENSURING PIN SECTIONS WIDTHS DO NOT EXCEED 1.0m WITH A MAXIMUM OF 20% OF THE WALL BEING OPEN AT ANY TIME.
3. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO PROVIDE A FULL T.W. PACKAGE INCLUDING PROPPING PLAN & T.W. DESIGN CALCULATIONS. THIS DRAWING AND ANY SUPPORTING SKETCH CALCULATIONS SHOULD NOT BE RELIED ON FOR THE REQUIRED T.W. PROVISIONS.
4. ALL CONSTRUCTION DETAILS TO BE AS RELEVANT ENGINEER'S AND ARCHITECTS'S DETAILS.
5. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH WRITTEN METHOD STATEMENT.
6. UNDER NO CIRCUMSTANCES SHOULD DIGGERS BE USED FOR ANY EXCAVATIONS ALL WORKS TO BE HAND DUG.

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DRAWING NOTES

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   0mm 25mm 50mm
8. ALL WORK CARRIED OUT SHOULD ADHERE TO SPECIFIC DRAWING NOTES & GUIDANCE AND COMPLY WITH CURRENT RELEVANT HSE & CDM REGULATIONS TO ENSURE SAFE SITE PRACTICE IS MAINTAINED IN ACCORDANCE WITH DETAILS PRODUCED BY OTHERS.
9. FOR GENERAL NOTES SEE DRAWING: M20202/GN01.
10. ALL DIMENSIONS SHOWN ARE IN mm U.N.O.

DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
JAN 21	A	GLF SLAB NOTE ADDED	LS
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

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PROJECT
30 FERNCROFT AVENUE
LONDON
NW3 7PH

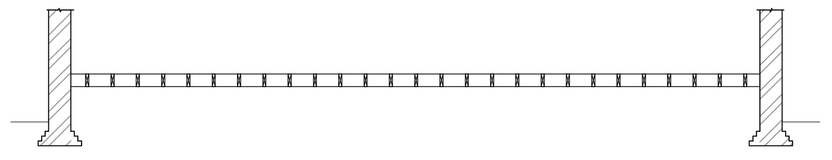
DRAWING TITLE
KEY PLAN & CONSTRUCTION SEQUENCE A-A

SCALE: 1:100 @ A1
DRAWN: LS
ENGINEER: LS
DATE: JAN 2021
CHECKED: --
DRAWING No.: M20202/ B3
REVISION: A

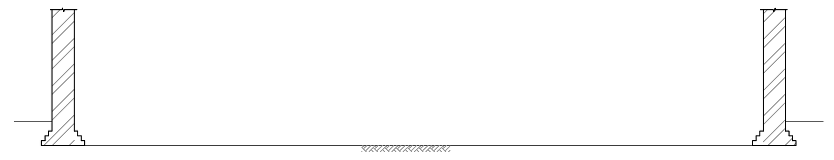
BASEMENT CONSTRUCTION SEQUENCE B-B

SCALE 1:100 @ A1

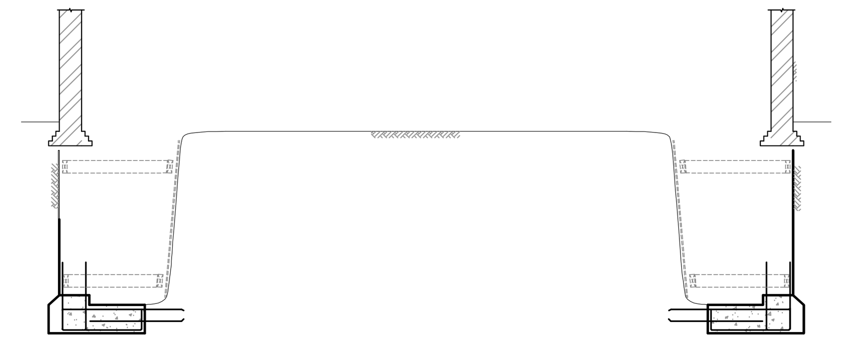
PROP NOTE
ALL WAILINGS TO BE "MABEY MASS" GRILLAGE BEAMS INSTALLED TO MANUFACTURERS SPECIFICATIONS.



1. EXTG SITE CONDITION.

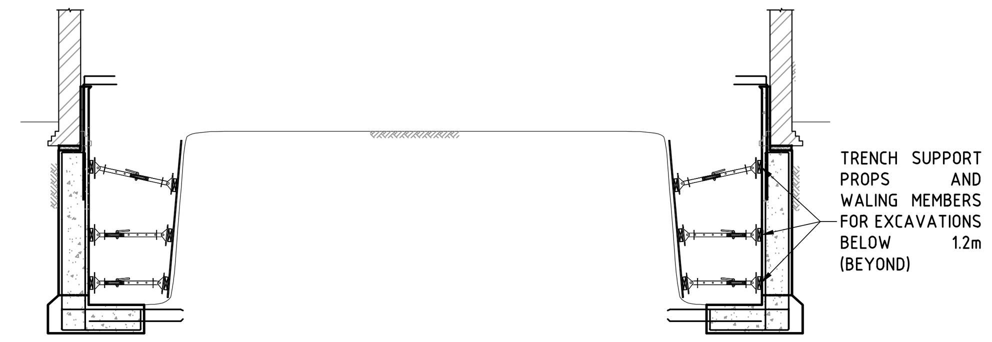


2. REMOVE EXTG FLOOR STRUCTURE & EXCAVATE TOPSOIL TO LVL OF EXTG FOOTINGS.

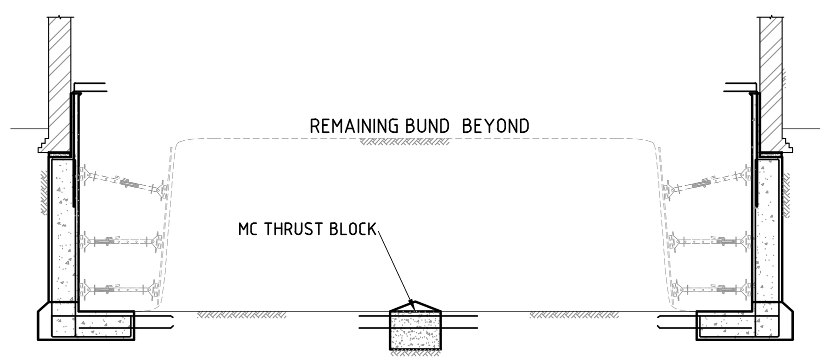


3. FOLLOWING INSTALLATION OF TEMP WORKS & DEMOLITIONS. EXCAVATE TO FORMATION LEVEL PROVIDING RETENTION SHORING TO EXPOSED FACES OF ACCESS PIT. CAST RC BASE & KICKER IN ACCORDANCE WITH WRITTEN METHOD STATEMENT.

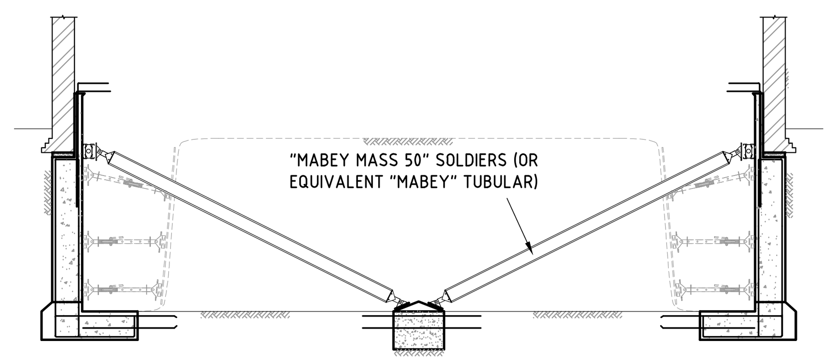
NOTE:
EXTG PROJECTING FOUNDATION / CORBELL FOOTING NOT TO BE CUT BACK UNTIL UNDERPINNING COMPLETE AND DRY PACK COMPLETELY GONE OFF



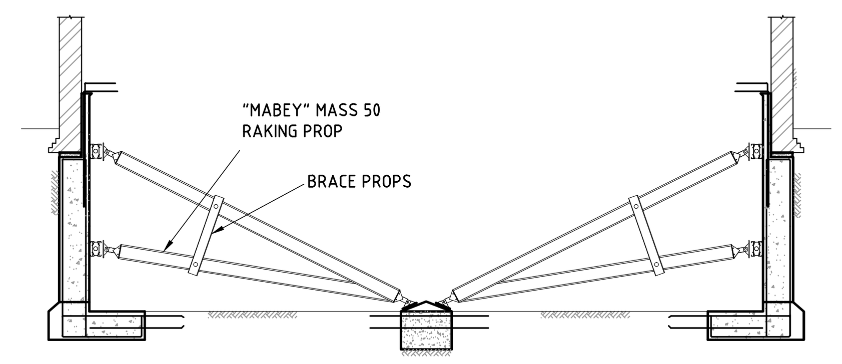
4. CAST WALL STEM AND DRY PACK BTWN EXTG IN ACCORDANCE WITH WRITTEN METHOD STATEMENT.



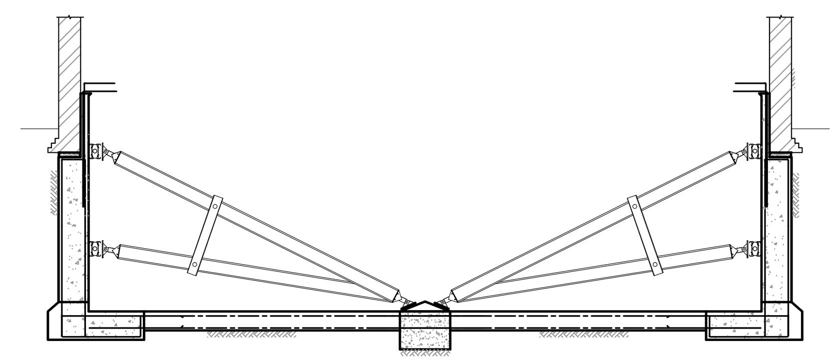
5. EXCAVATE ACCESS CHANNEL TO ALLOW CASTING OF CENTRAL THRUST BLOCK. SIDES OF CHANNEL TO BE FULLY SHORED WITH ACROW PROPPED TRENCH SHEETING EITHER SIDE. (BUND TO REMAIN UNEXCAVATED IN ALL AREAS BARRING ACCESS CHANNEL)



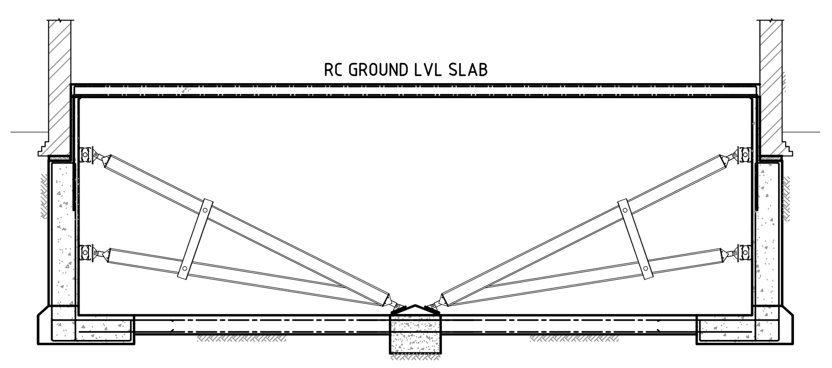
6. INSTALL HIGH LEVEL "MABEY MASS 50" RAKING PROPPING AND WAILINGS.



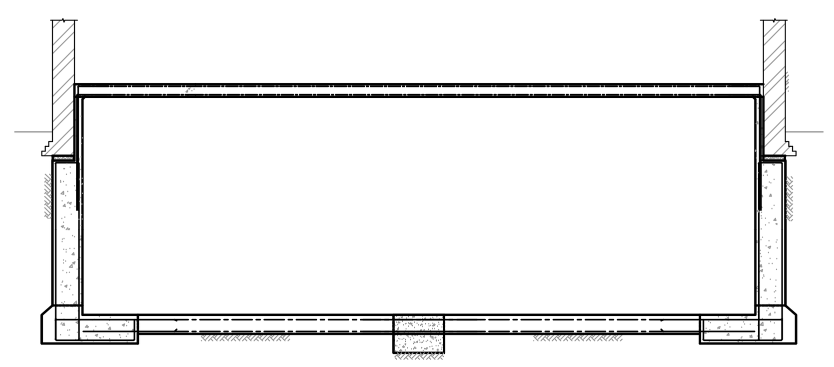
7. INSTALL LOWER LEVEL "MABEY MASS 50" RAKING PROPPING AND WAILINGS AND EXCAVATE REMAINING BUND SURROUND.



8. CAST RC BASE SLAB.



9. CAST GROUND FLOOR SLAB TO TOP OF RC WALL INSTALLED TO DESIGN ENGINEER'S DETAILS AND SPECIFICATION.



10. REMOVE TEMPORARY PROPS AND RE-INSTATE GROUND FLOOR STRUCTURE AS NECESSARY.

NOTES:

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0mm 25mm 50mm

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9. FOR GENERAL NOTES SEE DRAWING: M20202/GN01.
10. ALL DIMENSIONS SHOWN ARE IN mm U.N.O.

DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

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E: advice@mintstructures.co.uk
W: www.mintstructures.co.uk

PROJECT
30 FERNCROFT AVENUE
LONDON
NW3 7PH

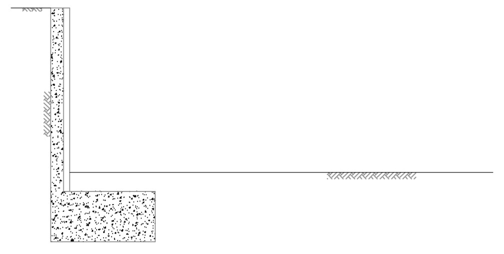
DRAWING TITLE
BASEMENT CONSTRUCTION SEQUENCE B-B

SCALE	DRAWN
1:100 @ A1	LS
	ENGINEER: LS
DATE: JAN 2021	CHECKED: - -
DRAWING No.: M20202/ B4	REVISION: -

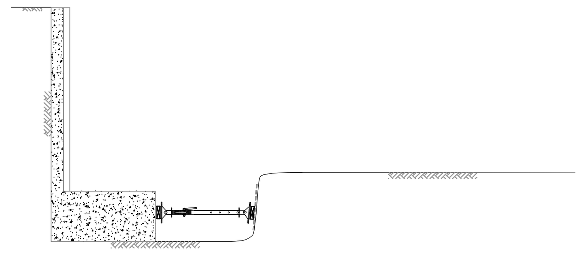
BASEMENT CONSTRUCTION SEQUENCE C-C

SCALE 1:100 @ A1

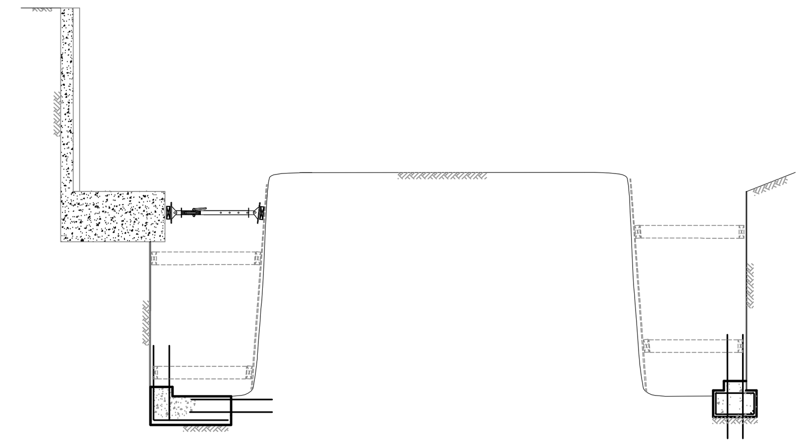
PROP NOTE
ALL WAILINGS TO BE "MABEY MASS" GRILLAGE BEAMS INSTALLED TO MANUFACTURERS SPECIFICATIONS.



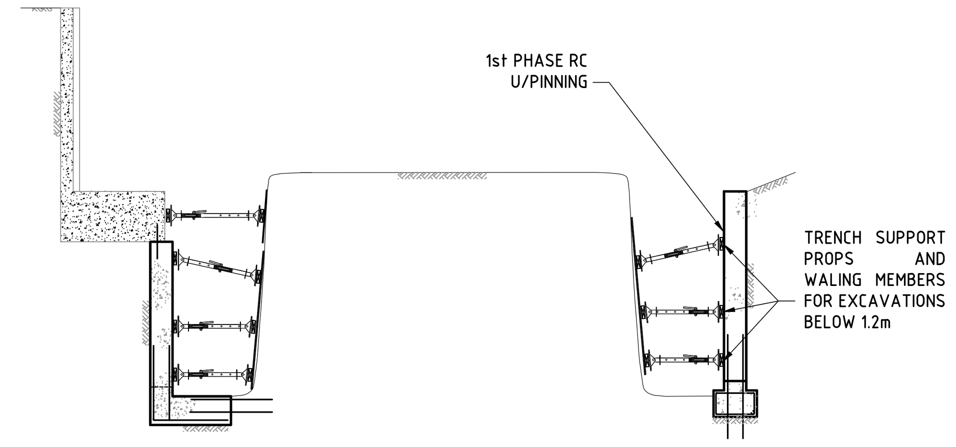
1. EXTG SITE CONDITION.



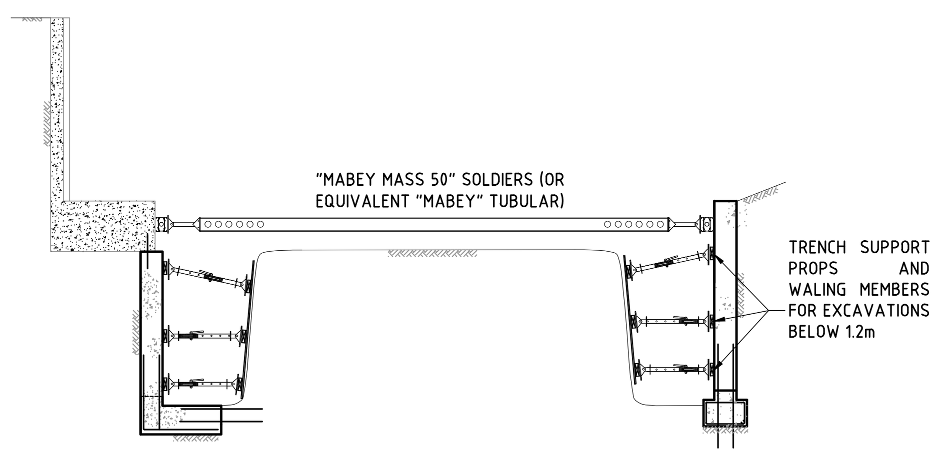
2. REMOVE EXTG GARDEN SURFACING & EXCAVATE TOPSOIL TO LVL OF EXTG FOOTINGS PROVIDING SHORING.



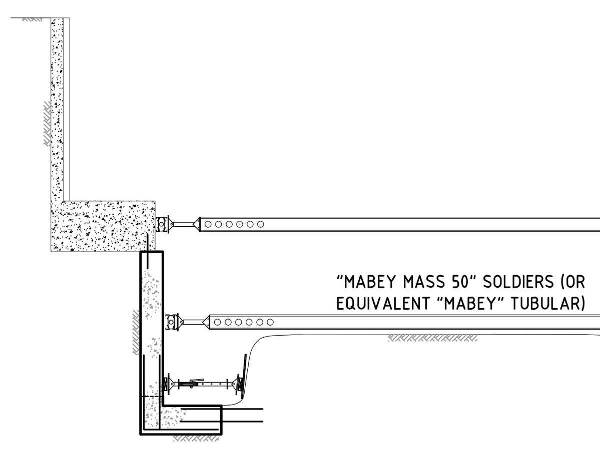
3. EXCAVATE TO UPPER FORMATION LEVEL PROVIDING RETENTION SHORING TO EXPOSED FACES OF ACCESS PIT. CAST RC BASE & LATERAL RC BEAM (BEAM TO ALSO ACT AS 1ST PHASE BOOT IN TEMPORARY CASE) IN ACCORDANCE WITH WRITTEN METHOD STATEMENT.



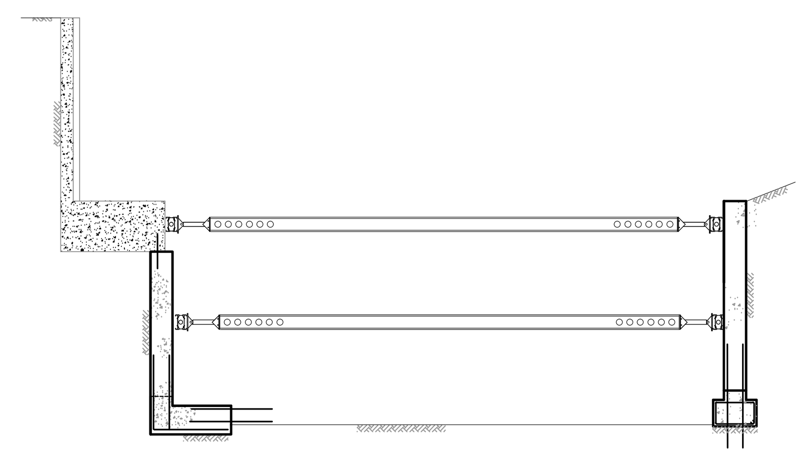
4. CAST WALL STEM AND DRY PACK BTWN EXTG IN ACCORDANCE WITH WRITTEN METHOD STATEMENT.



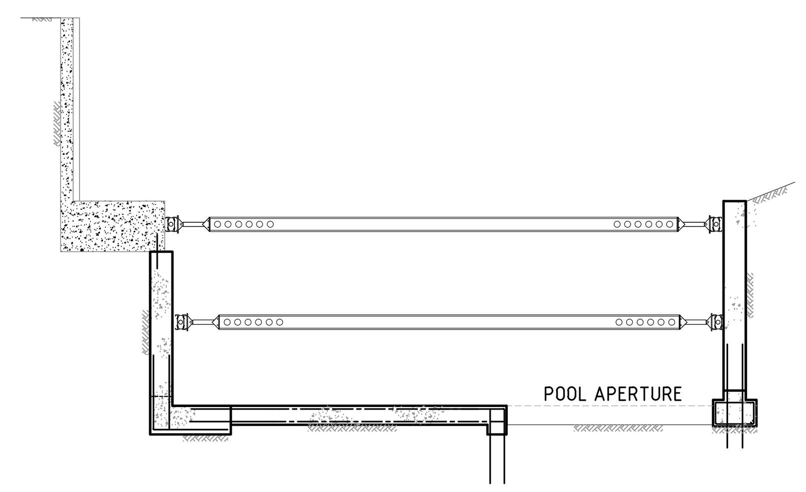
5. REDUCE LEVEL OF CENTRAL EARTH MASS TO ALLOW INSTALLATION OF HIGH LEVEL "MABEY MASS 50" PROPPING AND WAILINGS.



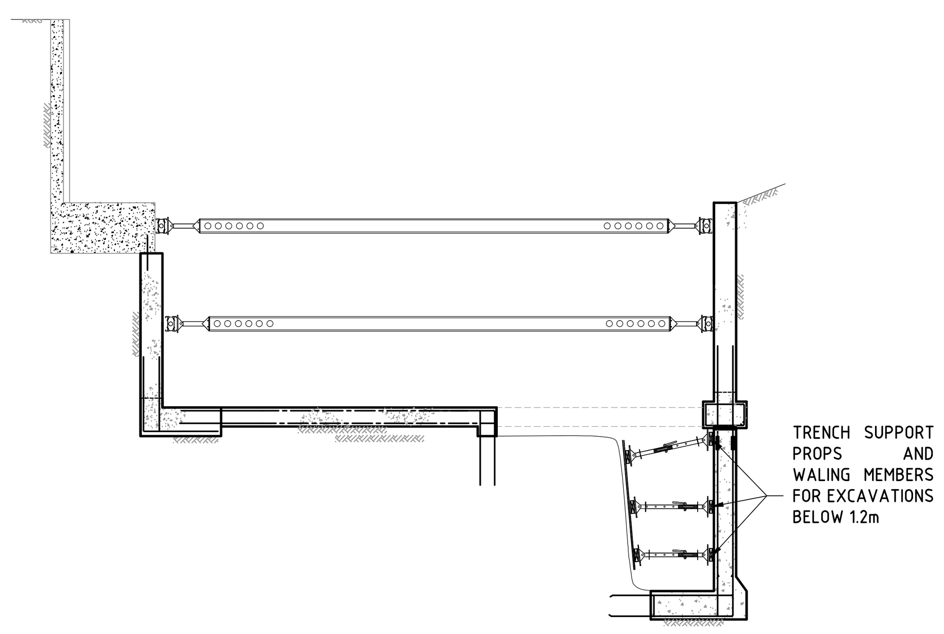
6. PROGRESS EXCAVATION TO 1/2 HEIGHT AND INSTALL 2nd ROW OF PROPPING ACROSS SITE. ALL PROPPING AND SHORING TO REMAIN IN PLACE UNTIL ALL REQUIRED UNDERPINNING IS COMPLETE AND BASE SLAB HAS BEEN CONSTRUCTED.



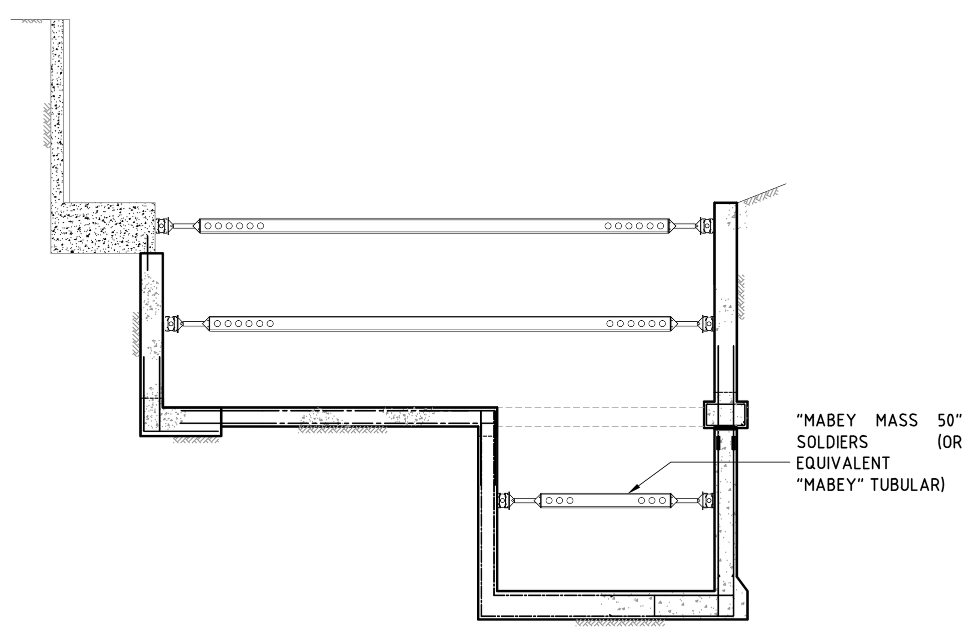
7. EXCAVATE TO BASE SLAB FORMATION LEVEL.



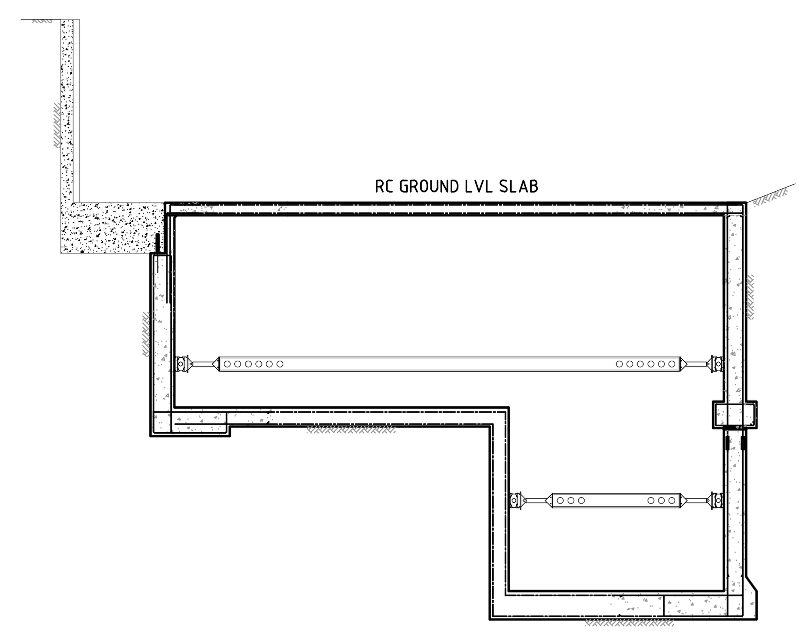
8. CAST RC BASE SLAB.



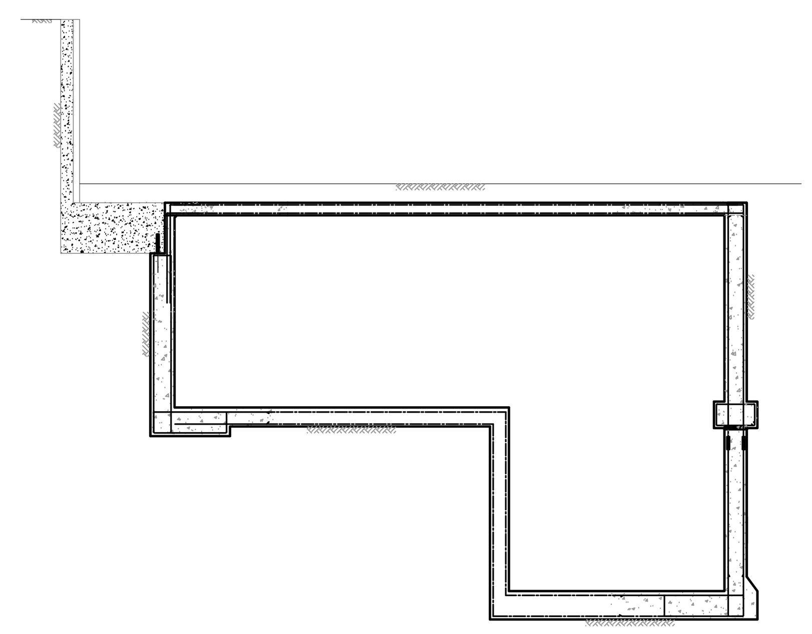
9. PROGRESS EXCAVATION TO FORM LOWER LEVEL POOL UNDERPINNING BELOW LATERAL SUPPORT BEAM.



10. COMPLETE INTERNAL POOL WALLS LINKED TO MAIN BASE SLAB.



11. CAST GROUND FLOOR SLAB INTO TOP OF RC WALL INSTALLED TO DESIGN ENGINEER'S DETAILS AND SPECIFICATION.



12. REMOVE TEMPORARY PROPS AND RE-INSTATE GROUND FLOOR STRUCTURE AS NECESSARY.

NOTES:

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10. ALL DIMENSIONS SHOWN ARE IN mm U.N.O.

DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
-	-	-	-
-	-	-	-
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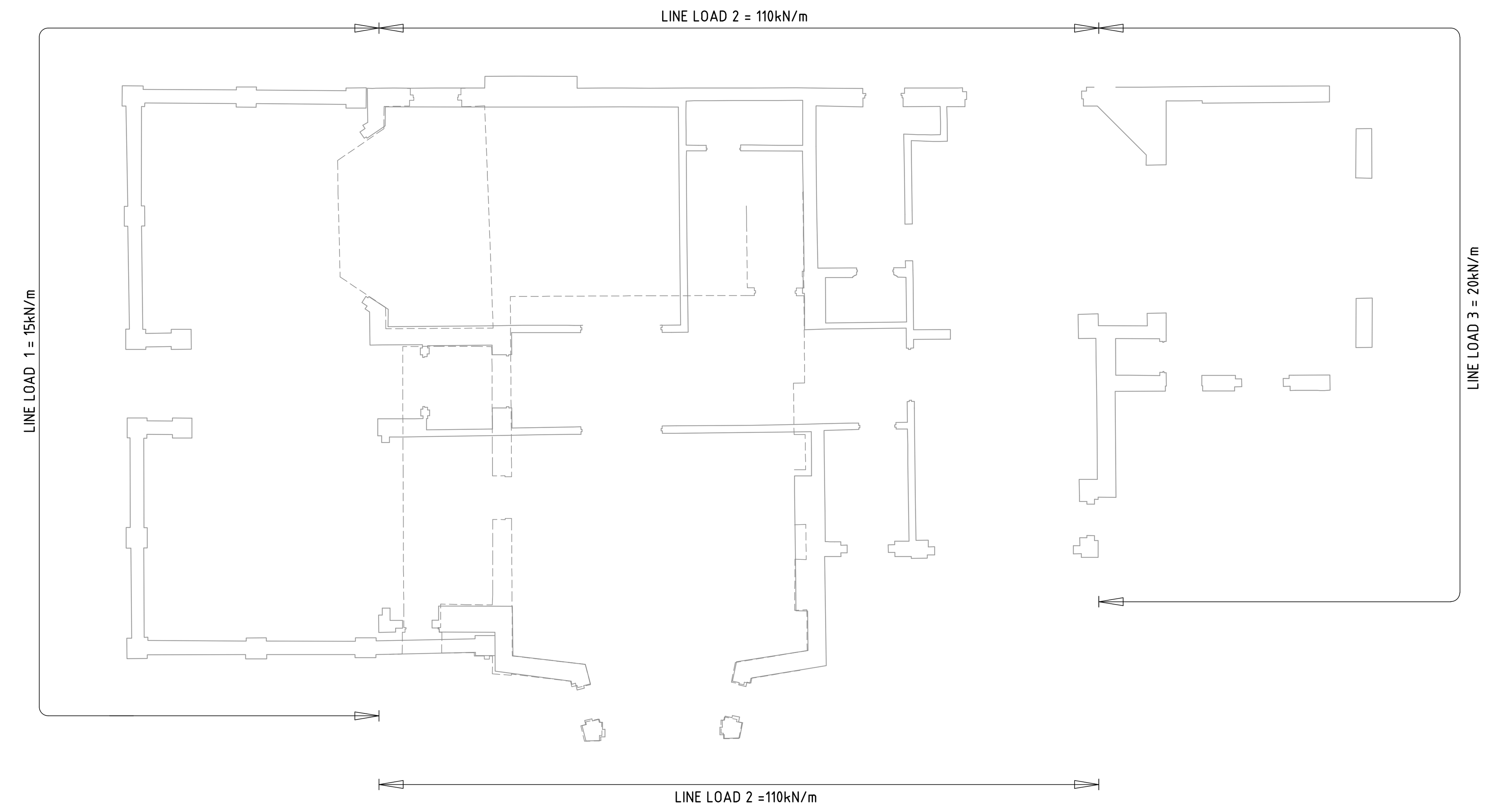
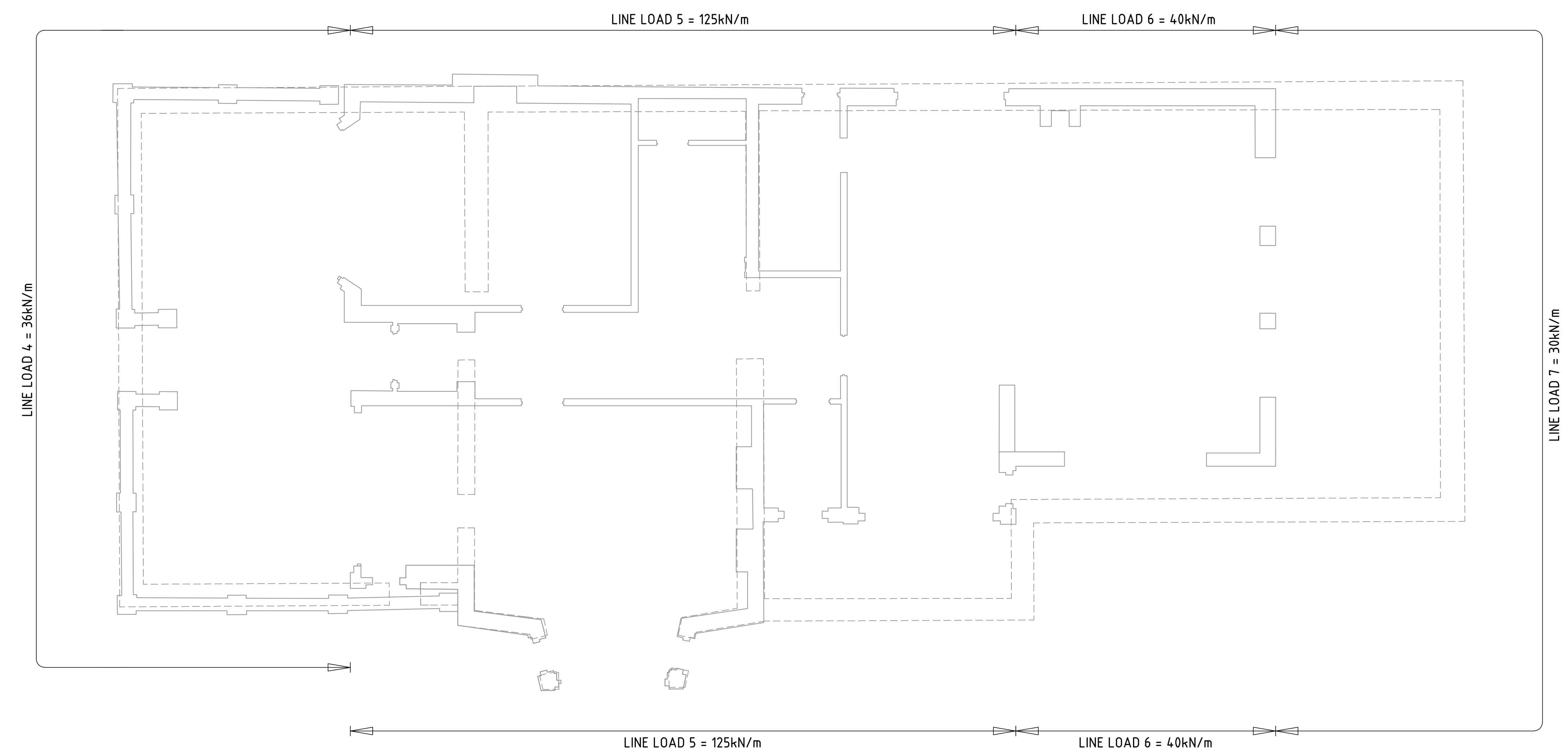
PROJECT
30 FERNCROFT AVENUE
LONDON
NW3 7PH

DRAWING TITLE
BASEMENT CONSTRUCTION SEQUENCE C-C

SCALE 1:100 @ A1	DRAWN LS
	ENGINEER LS
DATE JAN 2021	CHECKED --
DRAWING No. M20202/ B5	REVISION -



EXISTING LOADING CONDITION



PROPOSED LOADING CONDITION

KEY	
- - - - -	BASEMENT LEVEL OUTLINE
———	GROUND FLOOR LEVEL OUTLINE

NOTES:
1. THIS DRAWING IS INTENDED AS INDICATIVE PRELIMINARY INFORMATION ONLY.
2. LINE LOADS GIVEN ARE UNFACTORED LOADS AT GROUND LEVEL I.E. NOT INCLUDING FOUNDATION/BASE SLAB SELF WEIGHT.
3. LINE LOADINGS SHOWN ARE BASED ON SIMPLE AREA LOADING AS REQUIRED FOR THE PREPLANNING ANALYSES. ALL DESIGN LOADING CALCULATIONS AT DETAILED DESIGN STAGE WILL SUPERSEDE THE VALUES GIVEN HERE.
4. FINAL CONSTRUCTION DETAILS MUST NOT RELY ON VALUE ON THIS DRAWING.
5. ALL CONSTRUCTION DETAILS SHOULD ADHERE TO FINAL STRUCTURAL ENGINEER'S AND ARCHITECT'S DETAILS ONLY.

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DRAFT ISSUE

DATE	REV	DETAIL	DRAWN
-	-	-	-
-	-	-	-
-	-	-	-
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-	-	-	-

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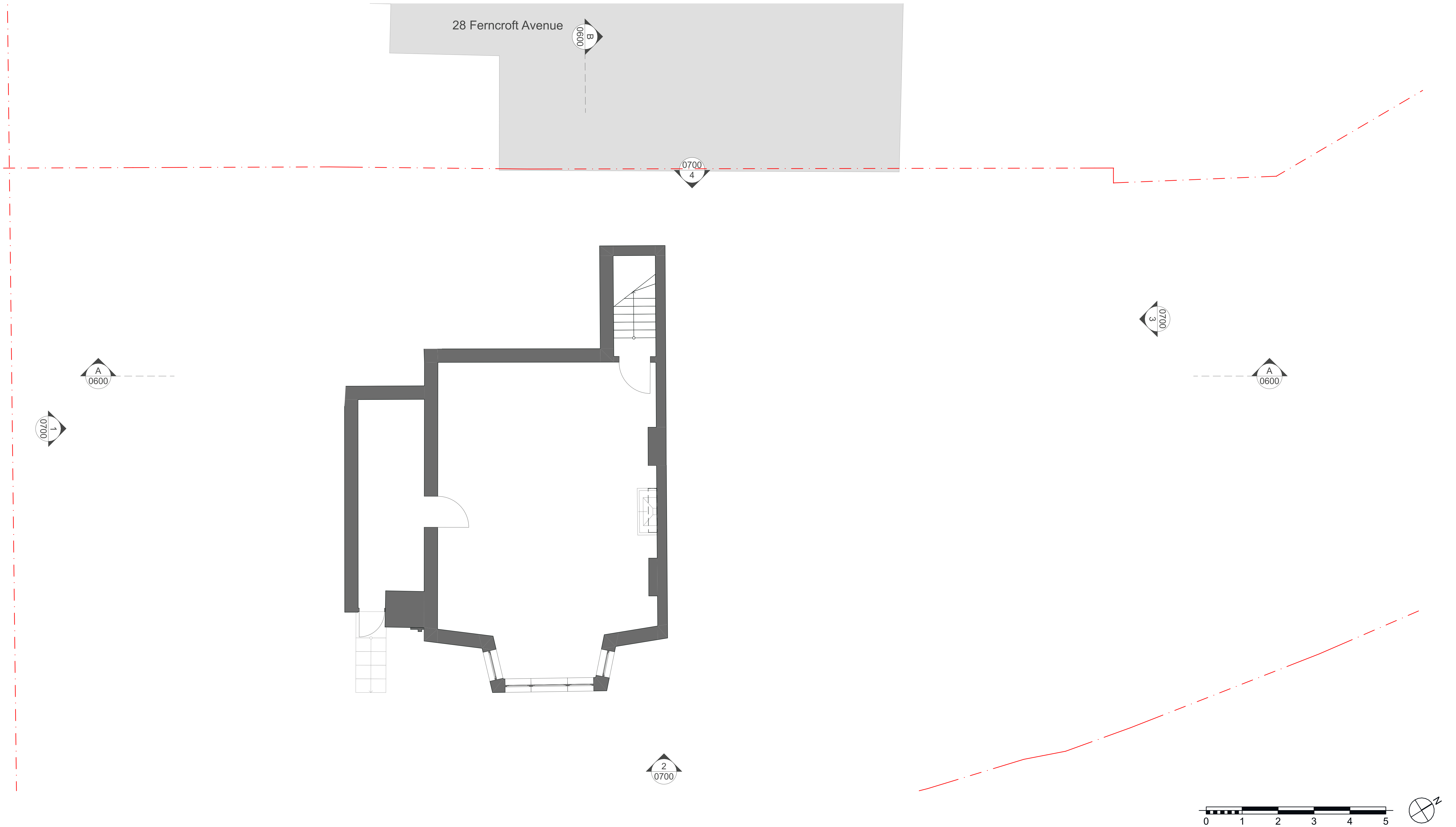
PROJECT
30 FERNCROFT AVENUE
LONDON
NW3 7PH

DRAWING TITLE
SUGGESTED EXISTING AND PROPOSED LINE LOADING FOR PRELIMINARY BASEMENT ANALYSIS

SCALE 1:100 @ A1	DRAWN LS
	ENGINEER LS
DATE JAN 2021	CHECKED --
DRAWING No. M20202 B6	REVISION -



No.	Notes:
1.	Dimensions to be verified on site. Only figured dimensions to be used and any discrepancies in dimensions are to be reported to the contract administrator. No dimensions are to be scaled from printed drawings. Any areas indicated on this drawing are for guidance only. No responsibility is taken for their accuracy.
2.	There is a risk of injury or death in construction if the works are not properly planned and supervised. The contractor must not undertake any element of the work without first having carried out the necessary risk assessments and prepared detailed method statements.

client: Aaaa Bbbb

project: 30 Ferncroft Avenue

drawing: BASEMENT

dwg. type: EXISTING

dwg. status: PLANNING

drawn by: ER | approved by: PR | date: 2/19/2021

scale: 1:100 @ A3 | format: A3

ARCHITECTURE

18 Greek Street
London W1D 4DS
+44 02070210332
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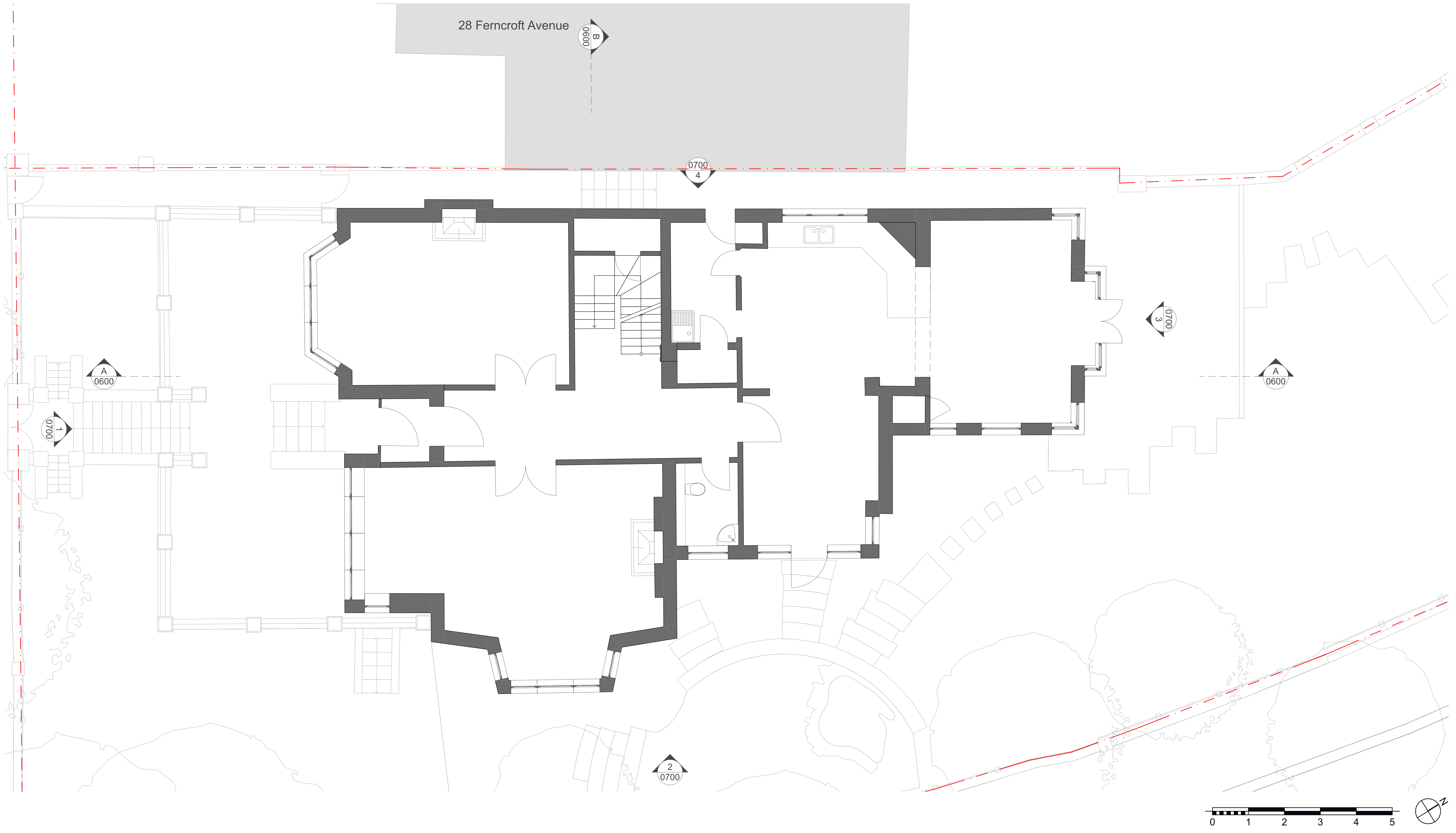
job number	drawing number	revision
309	0003	



No.	Notes:
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2.	There is a risk of injury or death in construction if the works are not properly planned and supervised. The contractor must not undertake any element of the work without first having carried out the necessary risk assessments and prepared detailed method statements.

client: Aaaa Bbbb
project: 30 Ferncroft Avenue
drawing: GROUND FLOOR PLAN
dwg. type: EXISTING
dwg. status: PLANNING
drawn by: ER approved by: PR date: 2/19/2021
scale: 1:100 @ A3 format: A3

ARCHITECTURE
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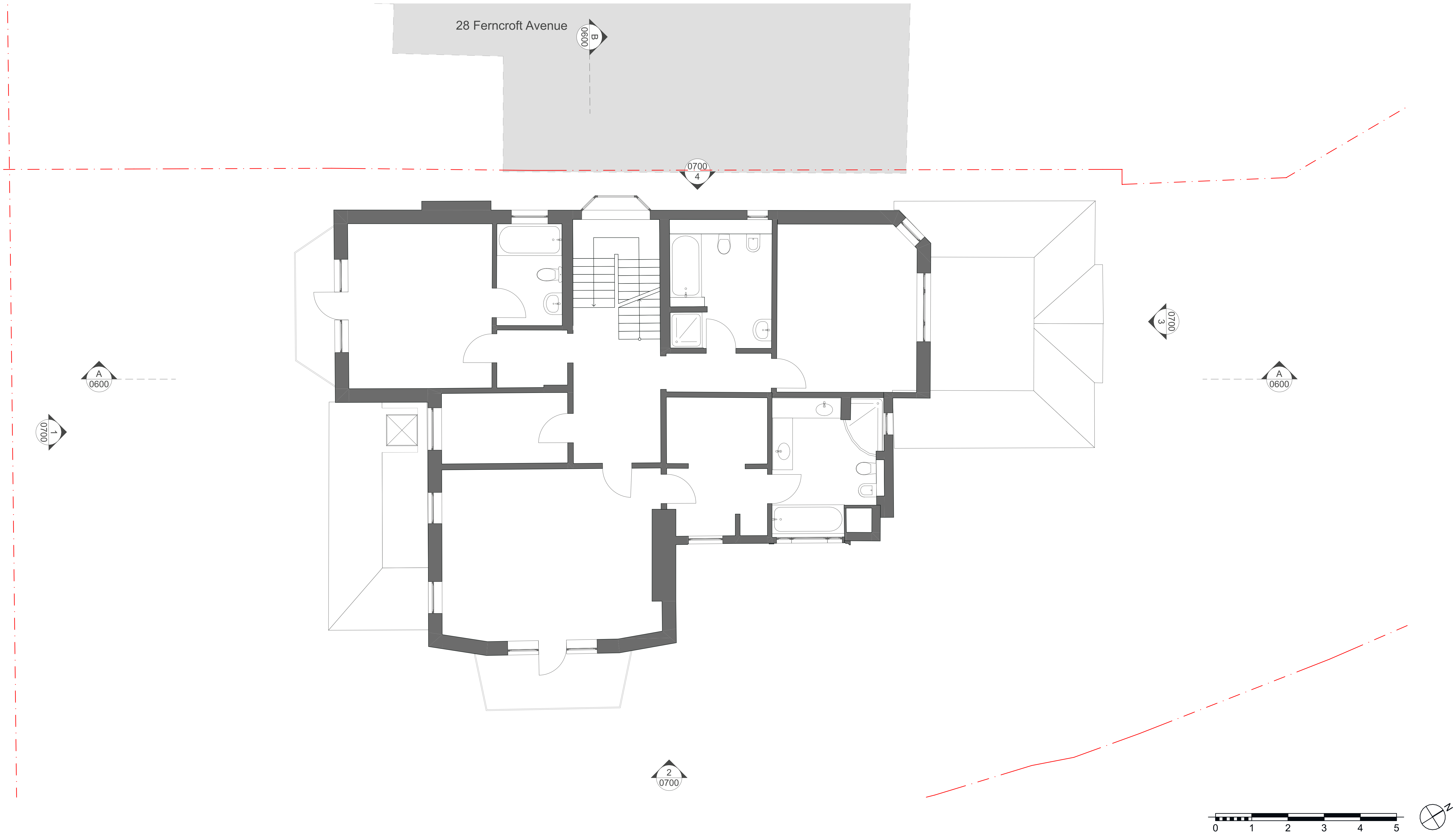
job number	drawing number	revision
309	0005	



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client: Aaaa Bbbb

project: 30 Ferncroft Avenue

drawing: FIRST FLOOR PLAN

dwg. type: EXISTING

dwg. status: PLANNING

drawn by: ER approved by: PR date: 2/19/2021

scale: 1:100 @ A3 format: A3

ARCHITECTURE

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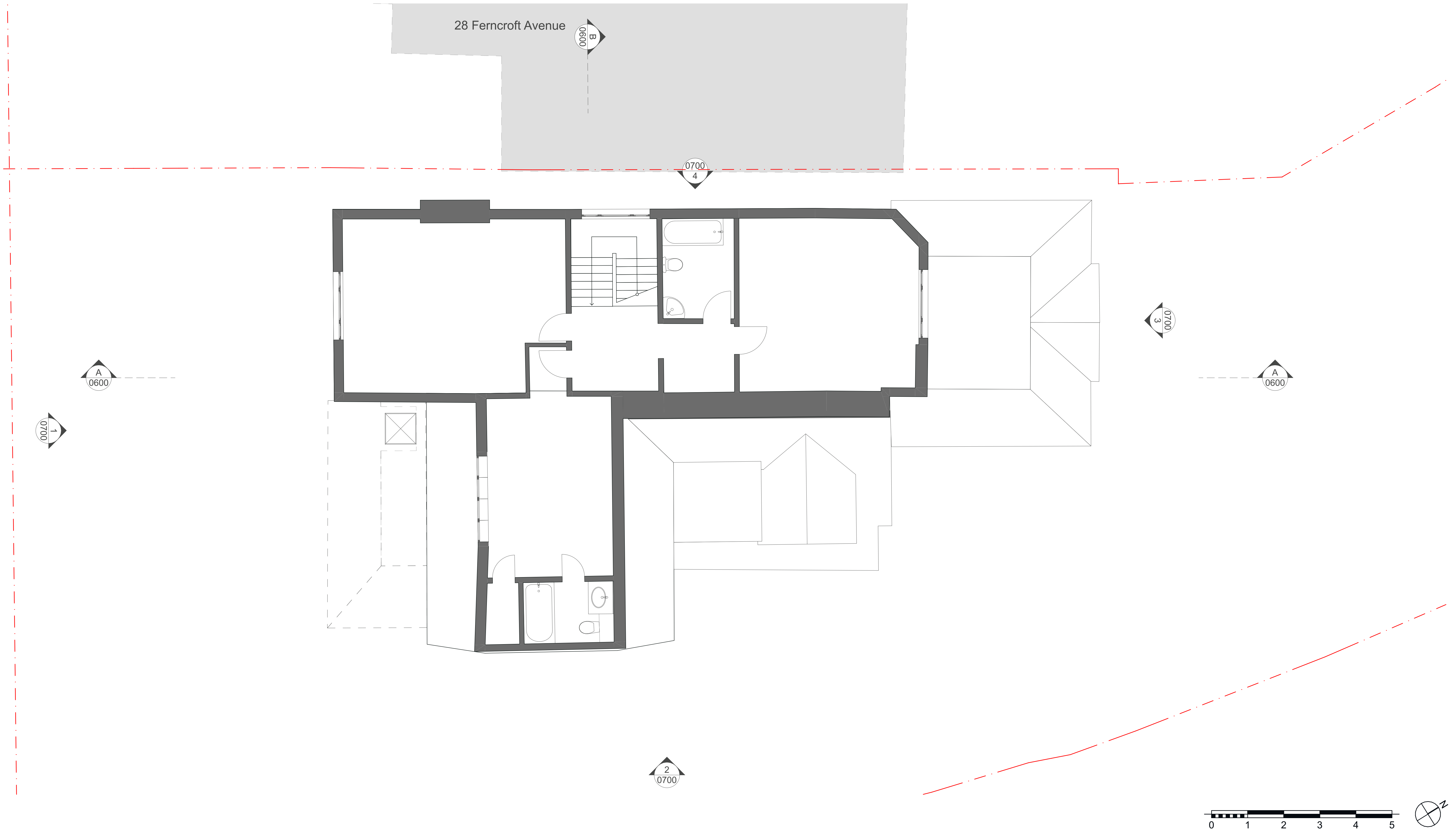
job number	drawing number	revision
309	0006	



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client: Aaaa Bbbb
project: 30 Ferncroft Avenue
drawing: SECOND FLOOR PLAN
dwg. type: EXISTING
dwg. status: PLANNING
drawn by: ER approved by: PR date: 2/19/2021
scale: 1:100 @ A3 format: A3

ARCHITECTURE
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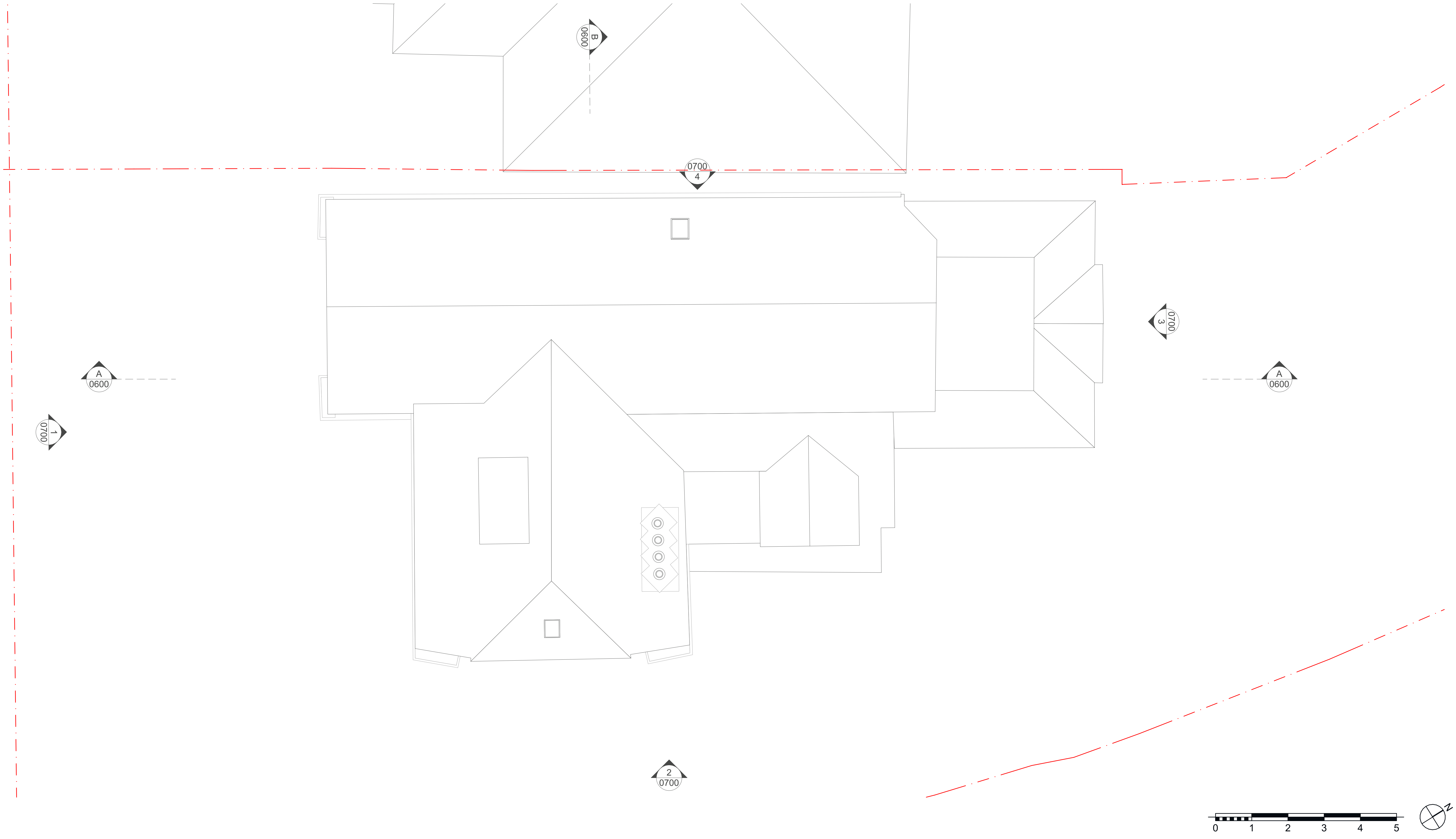
job number	drawing number	revision
309	0007	



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client: Aaaa Bbbb

project: 30 Ferncroft Avenue

drawing: ROOF

dwg. type: EXISTING

dwg. status: PLANNING

drawn by: ER approved by: PR date: 2/19/2021

scale: 1:100 @ A3 format: A3

ARCHITECTURE

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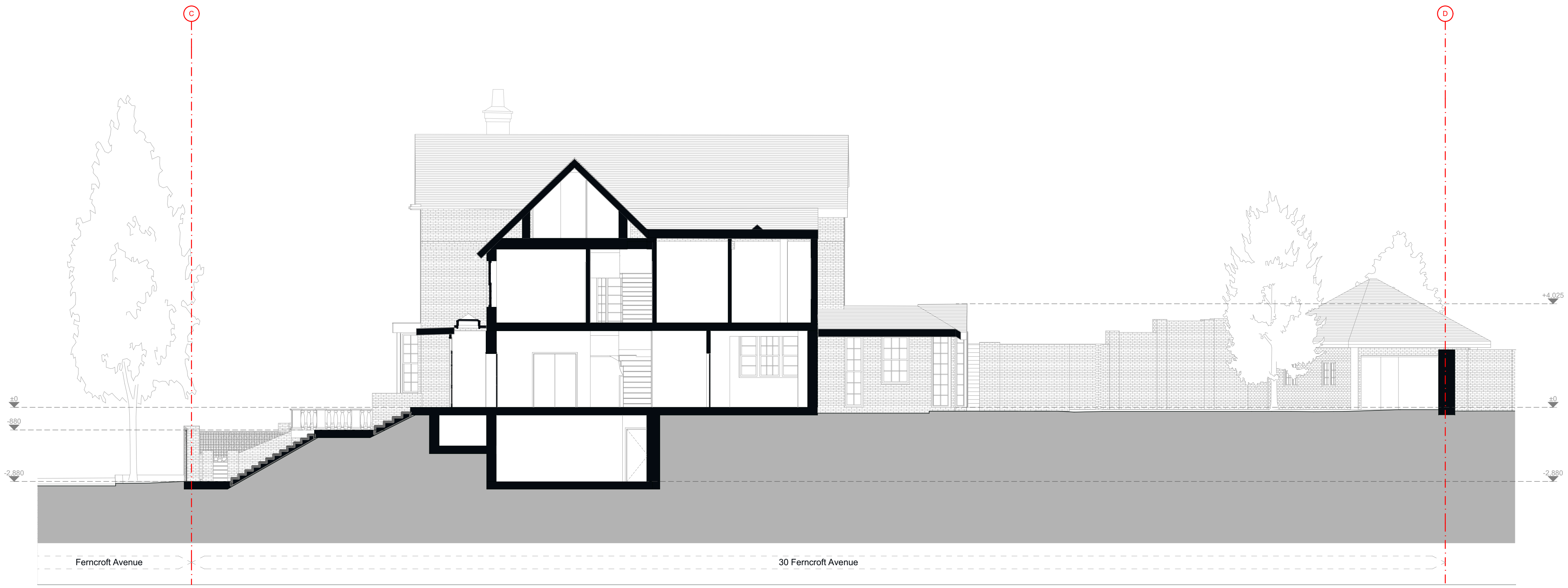
job number	drawing number	revision
309	0008	



No.	Notes:
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client: Aaaa Bbbb
project: 30 Ferncroft Avenue
drawing: SECTION C

dwg. type: EXISTING
dwg. status: PLANNING
drawn by: ER approved by: PR date: 2/19/2021
scale: 1:200 @ A3 format: A3

ARCHITECTURE
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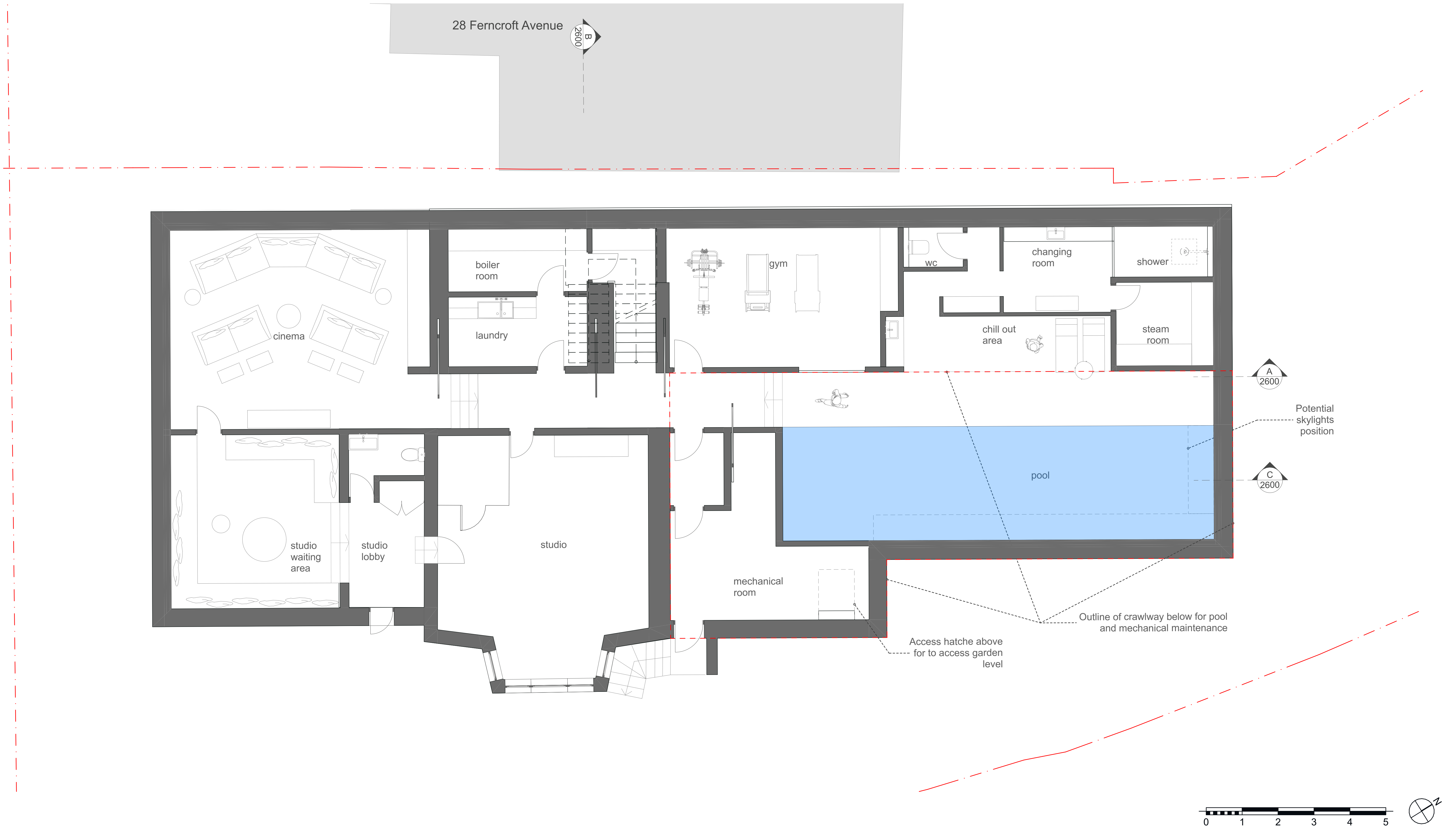
job number	drawing number	revision
309	0602	



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client: Aaaa Bbbbb

project: 30 Ferncroft Avenue

drawing: BASEMENT

dwg. type: PROPOSED

dwg. status: PLANNING

drawn by: ER approved by: PR date: 19/02/2021

scale: 1:100 @ A3 format: A3

ARCHITECTURE

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job number	drawing number	revision
309	2002	

No.	Notes:
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client: Aaaa Bbbbb
project: 30 Ferncroft Avenue
drawing: GROUND FLOOR PLAN

dwg. type: PROPOSED
dwg. status: PLANNING
drawn by: ER approved by: PR date: 19/02/2021
scale: 1:100 @ A3 format: A3

ARCHITECTURE
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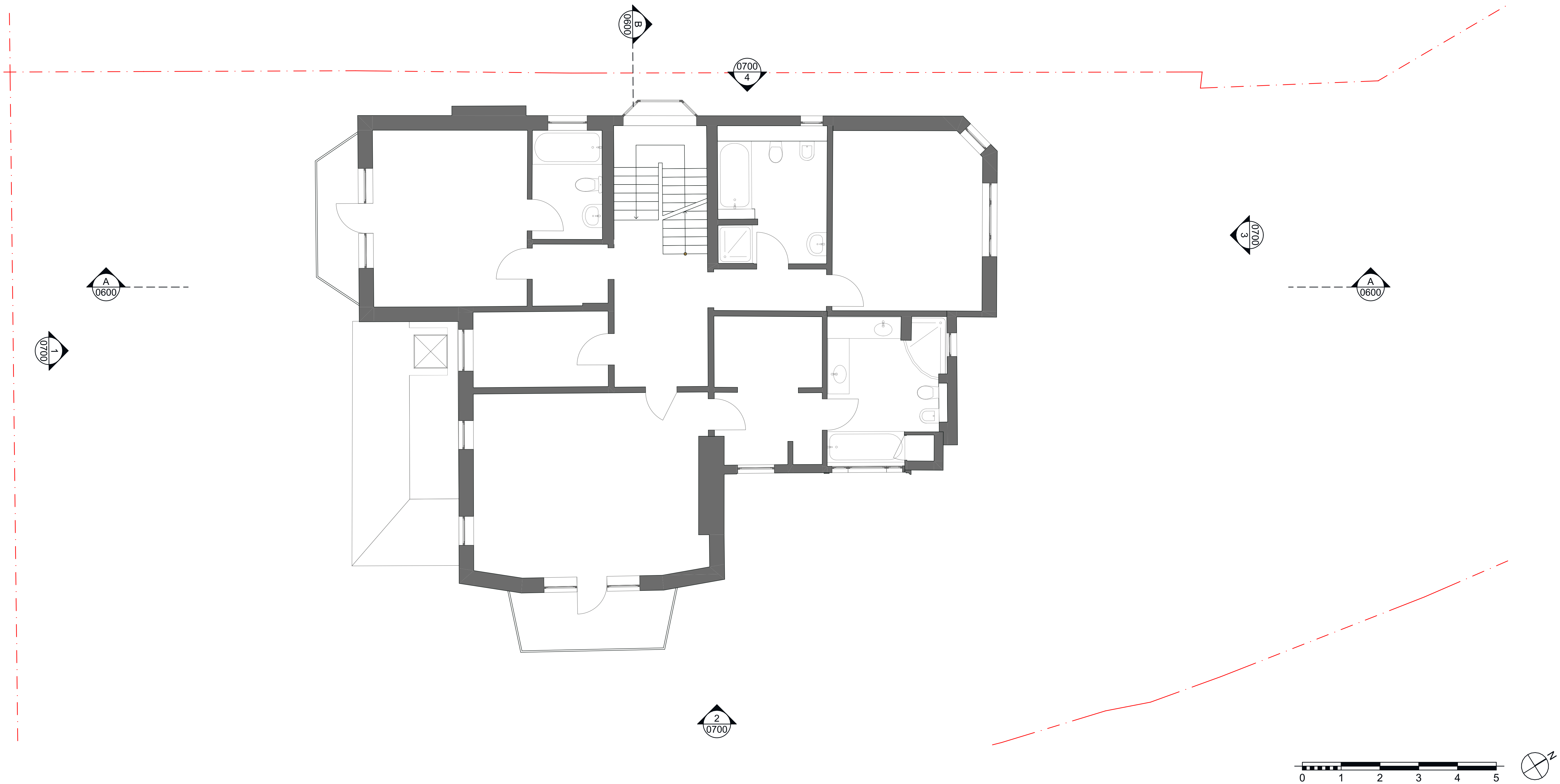
job number	drawing number	revision
309	2004	



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client: Aaaa Bbbbb
project: 30 Ferncroft Avenue

drawing: FIRST FLOOR PLAN

dwg. type: PROPOSED
dwg. status: PLANNING
drawn by: ER approved by: PR date: 19/02/2021
scale: 1:100 @ A3 format: A3

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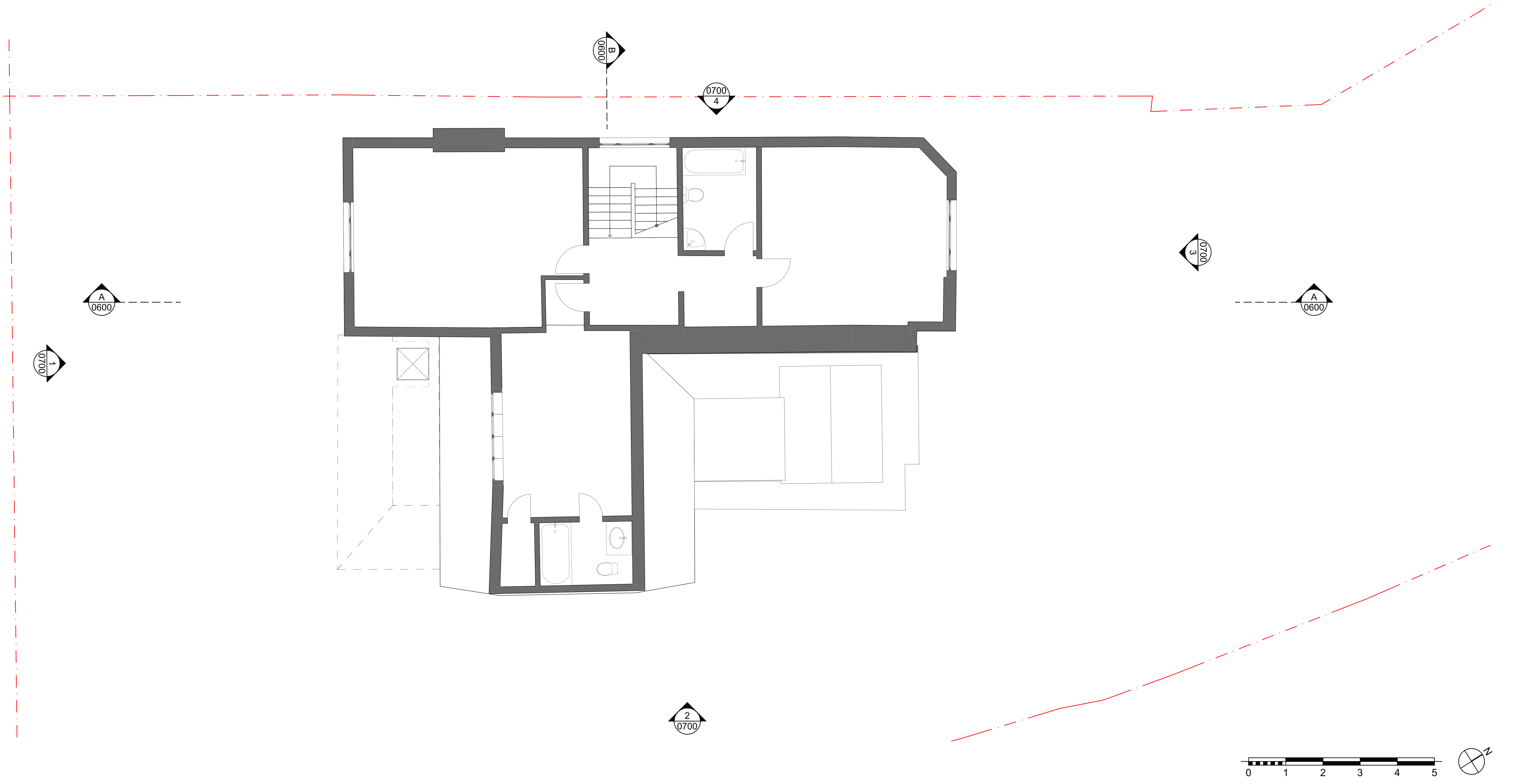
job number	drawing number	revision
309	2005	



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client: Aaaa Bbbbb
project: 30 Ferncroft Avenue
drawing: SECOND FLOOR PLAN
dwg. type: PROPOSED
dwg. status: PLANNING
drawn by: ER approved by: PR date: 19/02/2021
scale: 1:100 @ A3 format: A3

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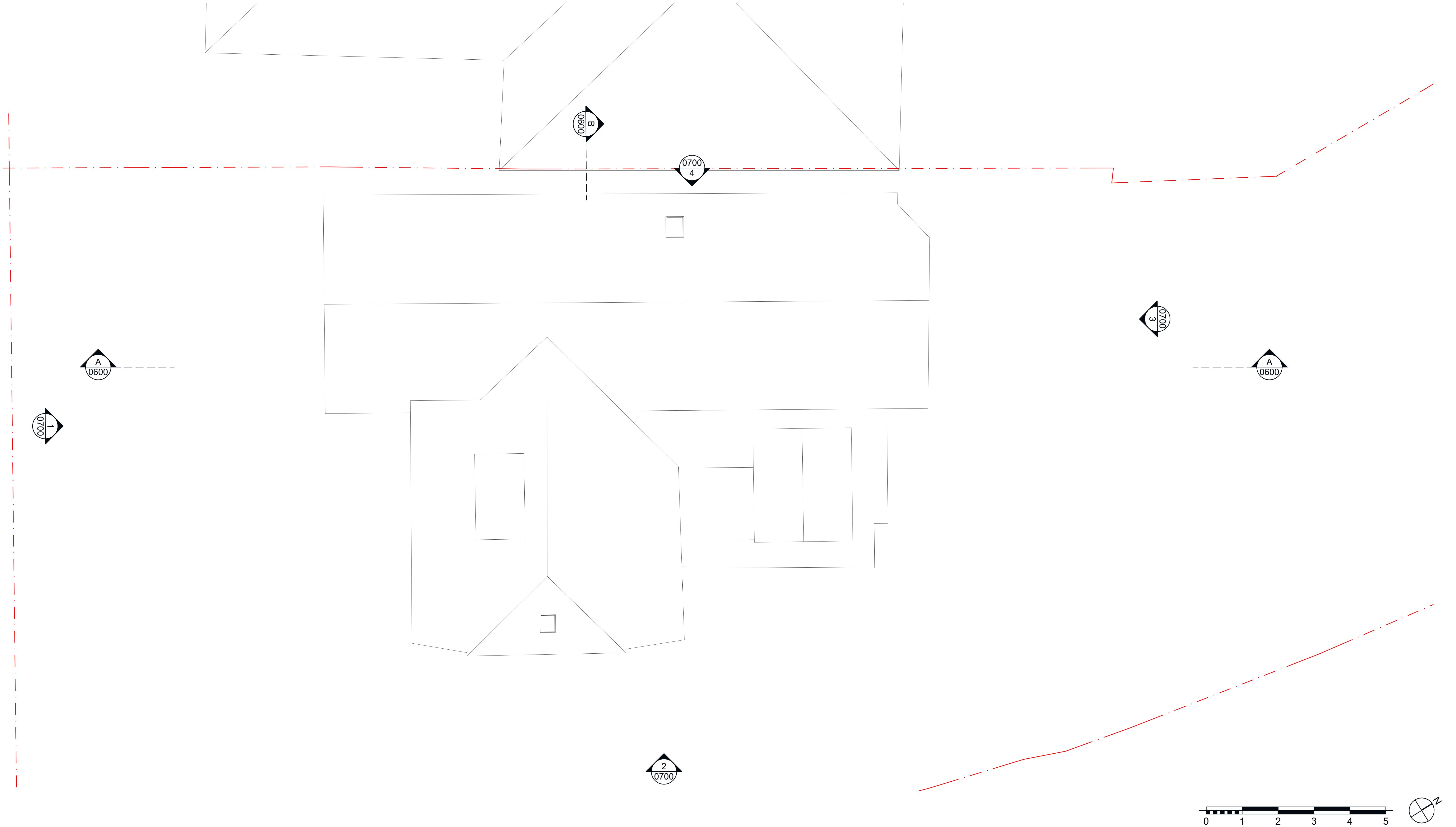
job number	drawing number	revision
309	2006	



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client: Aaaa Bbbbb

project: 30 Ferncroft Avenue

drawing: ROOF

dwg. type: PROPOSED

dwg. status: PLANNING

drawn by: ER approved by: PR date: 19/02/2021

scale: 1:100 @ A3 format: A3

ARCHITECTURE

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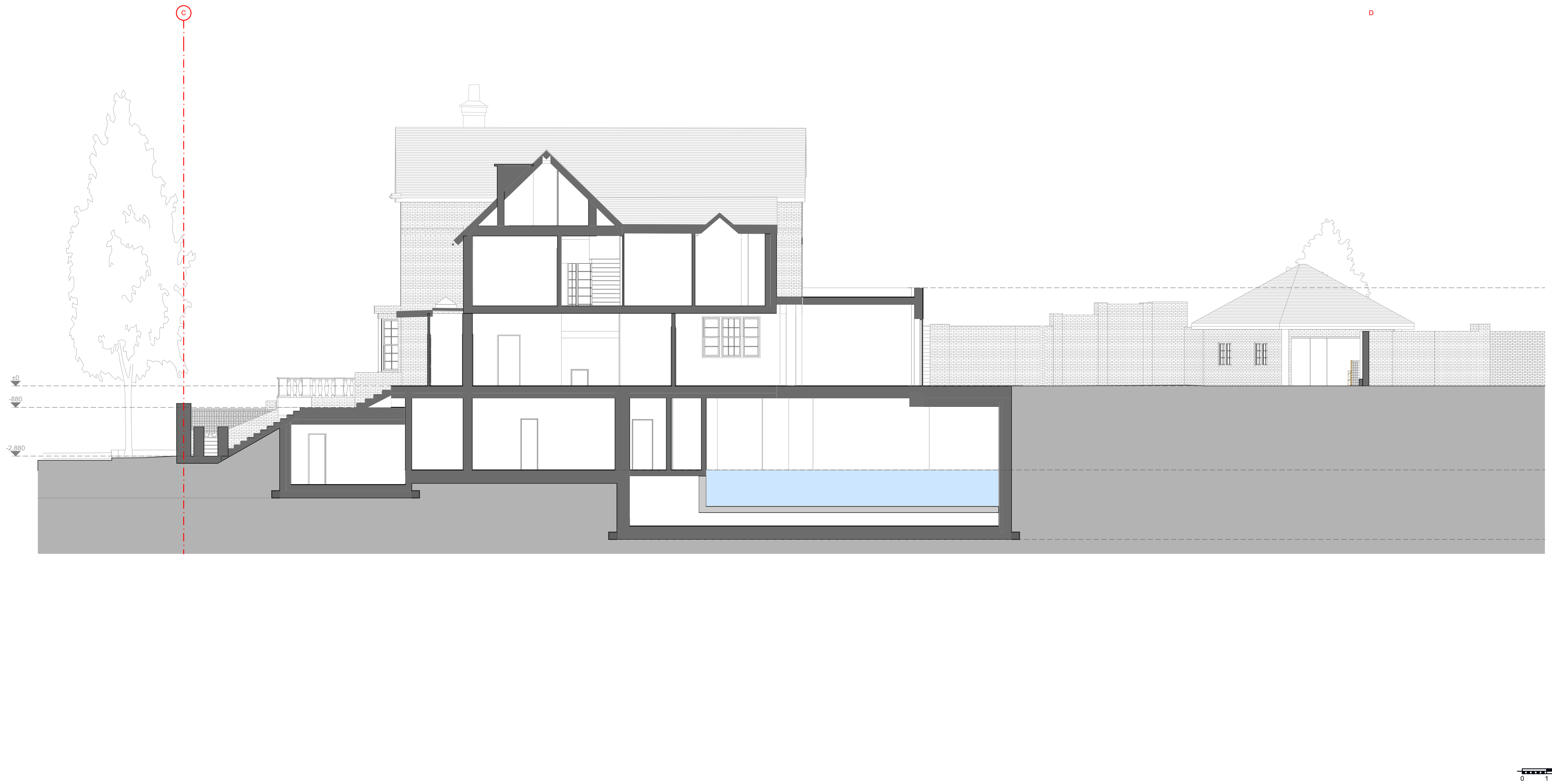
job number	drawing number	revision
309	2007	



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client: Aaaa Bbbbb
project: 30 Ferncroft Avenue
drawing: SECTION C
dwg. type: PLANNING
dwg. status: PLANNING
drawn by: ER approved by: PR date: 19/02/2021
scale: 1:200 @ A3 format: A3

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job number	drawing number	revision
309	2602	

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w: www.mintstructures.co.uk