Milvum Engineering
Services Ltd

The Milvum Group Ltd.
71-75 Shelton Street
London WC2H 9JQ

t: +44 7472 611560
e: contact@milvumgroup.com

www.milvumgroup.com

BASEMENT IMPACT ASSESSMENT

26 & 27 KING'S MEWS
LONDON, WC1N 2JB

Ref:	MES/2310/DCL005
Rev:	02
Date:	November 2023
Prepared:	Heather Shaw BSc MSc DIC
	Giulia Forlati MEng PhD
	Chris Emm MEng
Reviewed:	Phillip Lewis BSc MSc CGeol FGS
	Corrado Candian MEng CEng MICE

Table of Contents

1.0 Non-Technical Summary

At the request of DCL Consulting Engineers, on behalf of 1156 Limited, a Basement Impact Assessment (BIA) has been undertaken for 26 & 27 Kings Mews, London, WC1N 2JB (the site) in support of a planning application for a proposed new basement development to existing terraced commercial buildings, with existing basement below No. 27 King's Mews. The proposed basement will be beneath the full footprint of No. 26 Kings Mews, at the same depth as the existing basement to No. 27. Basement retaining walls will be formed using underpinning techniques.

The assessments have been undertaken by appropriately qualified professionals, including a Chartered Hydrogeologist (CGeol FGS) and Chartered Civil Engineer (CEng MICE).

A Desk Study, Screening and Scoping Assessments are reported separately (ref MES/2309/CDL004).

Site investigation confirms that the site is underlain by the River Terrace Deposits (Lynch Hill Gravel Member) and the London Clay Formation. The underlying soils will provide suitable bearing capacity for the proposed development's foundations.

The London Clay has potential to shrink and swell with moisture variation. The risk of movement and damage to this development due to moisture variation is negligible.

The River Terrace Deposits are classified as a Secondary A Aquifer and the London Clay is designated Unproductive Strata. Groundwater is present within the River Terrace Deposits. By adopting appropriate structural waterproofing, there is negligible risk of groundwater flooding of the basement.

Considering the ground and groundwater conditions, groundwater flow direction and construction formation levels there will be negligible impact or cumulative impact to the wider hydrogeological environment. During construction, localised groundwater control will be adopted to maintain stability.

The site and the adjacent properties have not been impacted by flooding and there is a reported very low risk from all sources. The SuDS strategy and flood risk assessment indicates the proposed basement does not impact the wider hydrological environment.

There will be no impact to slopes due to the proposed development. The main site is level and is not situated in a wider hillside environment of slopes of 7° or more.

Ground movements caused by the excavation and construction of the proposed development will be minimal. Damage impact to adjacent structures is assessed to be a maximum of Very Slight (Category 1 in accordance with the Burland Scale) with impact to the highway and underlying utilities assessed to be negligible.

It is recommended that structural movement monitoring is undertaken and mitigation actions implemented if movement trends indicate structural tolerances could be exceeded.

The BIA demonstrates that the proposed development will not cause adverse impacts relating to land stability, hydrogeology and surface water flow, and is at very low risk of flooding.

2.0 Introduction

At the request of DCL Consulting Engineers, on behalf of 1156 Limited, the following scope of works has been undertaken in order to inform a Basement Impact Assessment (BIA) for 26 & 27 Kings Mews, London, WC1N 2JB (the site) in support of a planning application for a proposed new basement development to existing terraced commercial buildings, with existing basement below No. 27 Kings Mews:

- a Ground Movement Assessment (GMA);
- a Flood Risk Assessment (FRA);
- a Drainage Strategy;
- and a Basement Impact Assessment (BIA).

A preliminary BIA (ref MES/2309/DCL004) was undertaken to provide the baseline information and preliminary assessments to inform the full BIA:

- a Desk Study;
- Screening;
- and Scoping of additional investigations and assessments.

A site investigation and geotechnical assessment has also been undertaken (by others):

- Ground Investigation (Ref C13870A) dated September 2023 prepared by Ground Engineering Limited.

The Structural Engineer, DCL Engineering Ltd, has provided temporary and permanent works information for assessment purposes, including proposed bearing pressures, sequencing and propping arrangements.

2.1 Summary of Screening and Scoping

Groundwater Flow

The site is located above an aquifer. A site investigation is therefore required to determine the presence or absence of perched water and / or groundwater. If it is assessed that an impact or cumulative impact may result, suitable mitigation should be proposed.

A drainage strategy is required to establish whether more surface water will be discharged to the ground, potentially impacting the hydrogeological environment.

Land Stability

The underlying soils should provide suitable bearing capacity for the proposed development's foundations. A site investigation is required with appropriate geotechnical assessment to ensure a suitable foundation design.

A ground movement assessment will be required to assess potential impacts, with reference to the underlying ground conditions (to be confirmed by site investigation) and the structural engineering proposals.

Surface Water Flow and Flooding

It is noted that the site is within a designated Critical Drainage Area. Potential impacts should be considered and confirmed by a drainage strategy and flood risk assessment, with appropriate mitigation measures adopted, as required.

2.2 Authors

The assessment has been reviewed and approved by Chartered Civil Engineer Corrado Candian, MEng CEng MICE and Chartered Hydrogeologist Philip Lewis, BSc CGeol FGS, who both have more than 20 years' relevant experience of design and assessment of residential and commercial developments including basements.

3.0 Site Investigation

3.1 Introduction

A ground investigation was undertaken by Ground Engineering Ltd comprising a single borehole. In addition, Ground Engineering reviewed site investigations undertaken at the adjacent 25 Kings Mews and 28 Kings Mews sites.

3.2 Ground Conditions

At 25 and 28 Kings Mews, the following ground and groundwater conditions were encountered:

- Made Ground to between 3.60m to 4.00m below ground level (bgl);
- Lynch Hill Gravel (sand and gravel) to between 5.10m to 6.00m bgl;
- London Clay to greater than 25.00m bgl;
- Groundwater at 4.00m bgl.

The ground conditions encountered in the borehole on site were as expected from the known history of the site and the adjacent geological records, with a significant thickness of Made Ground underlain by the Lynch Hill Gravel at 3.30m bgl. This superficial deposit was underlain by the solid geology of the London Clay at 6.40m depth. The latter was proven to 15.00m bgl.

Made Ground

The concrete floor slab was 0.30m thick and underlain by dark brown, slightly clayey, ashy sand and gravel with occasional brick cobbles. The base of the fill was proved at 3.30m bgl.

Lynch Hill Gravel

The superficial Lynch Hill Gravel Member was initially a medium dense, orange brown, silty sand and gravel, with a gravel fraction of angular to sub-rounded flint. Below 4.00m bgl it became very dense, to a depth of 5.90m bgl.

Between 5.90m and 6.40m a stiff brown gravelly clay was encountered, which Ground Engineering report as 'Reworked London Clay'. Triaxial testing indicates this has a shear strength of 91 kN/m^2.

London Clay Formation

The London Clay is indicated to be initially stiff becoming very stiff, with triaxial testing indicating a range of shear strength between 89 and 199 kN/m^2, typically increasing with depth.

3.3 Groundwater

During the ground investigation, water was recorded by the driller at 4.00m rising to 3.60m bgl in fifteen minutes. This water was sealed out of the borehole once the casing entered the underlying London Clay, and the 15.00m deep borehole was 'damp' on completion. The water level recorded in the 7.00m deep standpipe three weeks after installation was 3.04m bgl.

The groundwater recorded during the ground investigation works is considered to be representative of the Secondary (A) Aquifer of the Lynch Hill Gravel Member. It is well documented that natural recharge mechanisms for this aquifer have been limited by the mass urban development so that the groundwater is likely to be recharged via leaking drains / mains as much as rainfall infiltration. This is problematic in terms of meaningful appraisal of groundwater flow.

Whilst the groundwater recorded may be due to infiltrating surface water from the surrounding area rather than a baseflow within the superficial deposit as an aquifer, to be conservative impact to the hydrogeological environment has been considered assuming groundwater is present and flowing.

The site is located within approximately 200m of a former tributary of the 'lost' River Fleet which would have flowed in an easterly direction with the Fleet flowing in an approximately southerly direction toward the River Thames which is located approximately 1.2km south of the site. On this basis it could be assumed that groundwater flow in the Lynch Hill Gravel Member in this area is in a south easterly direction, although it is acknowledged that locally there could be flow to the east and north east.

The combination of the proposed new basement beneath 26 Kings Mews along with the existing basements beneath the adjacent properties and the basements under construction beneath 21-23 Kings Mews means that there will be a line of basement structures running along an approximate 35m section of Kings Mews.

To assess the potential for this line of basement structures to result in a cumulative impact and a local increase in groundwater elevation, consideration has been given to the local ground and groundwater conditions. Information from the ground investigation works (both on site and adjacent properties) suggests that locally groundwater within the Lynch Hill Gravel Member is approximately 3.00m to 4.00m bgl; with the London Clay Formation encountered at approximately 5.50m to 6.00m bgl, this suggests an aquifer thickness of approximately 2.00m to 3.00m.

The proposed formation level for the new basement is approximately 3.85m bgl, formed on the Lynch Hill Gravel Member and (based on the site specific data) there will be a saturated aquifer thickness of approximately 2.00m beneath the underside of the basement and the top of the London Clay Formation i.e. the basement formation will not effect an hydraulic cut off.

The existing basements immediately adjacent to the north and south of the site effectively already obstruct any groundwater flow, if present, so the proposed basement neither individually nor cumulatively increases this impact, with available soil volumes adjacent to the west (below the road, Kings Mews, and properties beyond without basements) and to the east (with a distance of approximately 8m between the proposed basement and the existing basement below 39 to 45 Gray's Inn Road), as well as below the basement slab.

There is potential for some minor diversion of groundwater flow, but it is considered unlikely that this would result in any significant cumulative effect from increase in local groundwater levels that could impact neighbouring properties via groundwater flooding.

3.4 Geotechnical Design parameters

Discussion on bearing capacity and geotechnical parameters for design of retaining walls are provided in the Ground Engineering report. Notwithstanding the bearing capacity recommendations provided, its understood that the structural engineer will limit bearing pressure to a maximum of 100kPa (see Section 5).

4.0 Flood Risk Assessment and Drainage Strategy

4.1 Sources of Flooding

Fluvial (Rivers and Seas)

The Environment Agency's Flood Map for Planning (Figure 1) shows the site to be in flood zone 1. This is defined as 'land having a less than 1 in 1,000 annual probability of river or sea flooding' and the property can therefore be considered to have a very low probability of fluvial flooding.

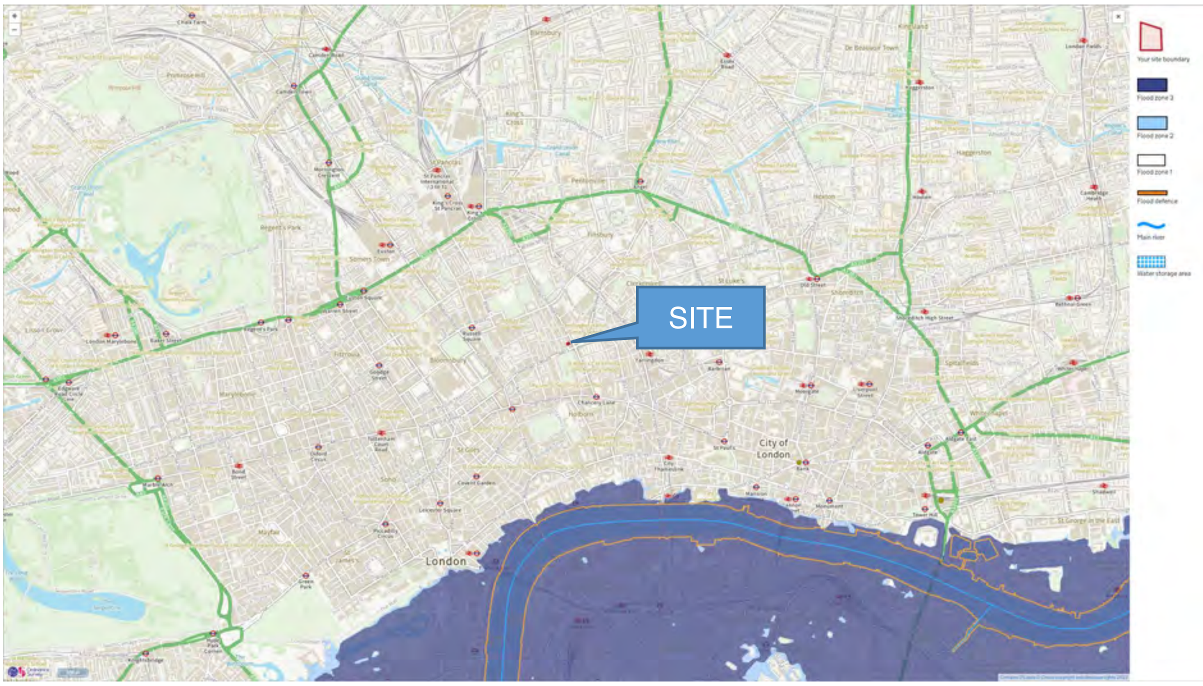


Figure 1: EA Flood Map for Planning[1]

Pluvial (Surface Water)

The Long-Term Flood Risk Map for Surface Water (Figure 2) does not show the subject property to be at risk of flooding from surface water. It can therefore be considered to be at very low risk of surface water flooding, considered to be land that each year this area has a chance of flooding of less than 0.1% (1 in 1,000).

With reference to LB Camden's Strategic Flood Risk Assessment (SFRA), and the Guidance to Subterranean Development (Figure 3), the Holborn area did not flood in 1975 nor 2002 and is not in an area with the potential to be at risk of surface water flooding.

[1] https://flood-map-for-planning.service.gov.uk/confirm-location?easting=530943&northing=182002

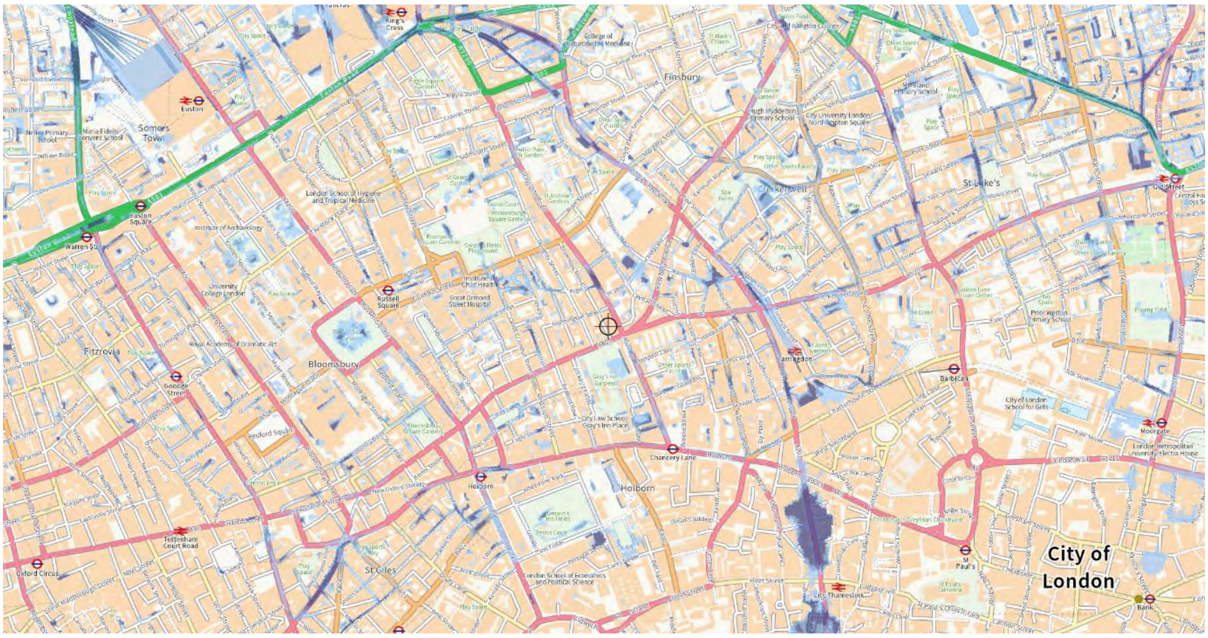


Figure 2: Long-Term Flood Risk Map - Surface Water[2]

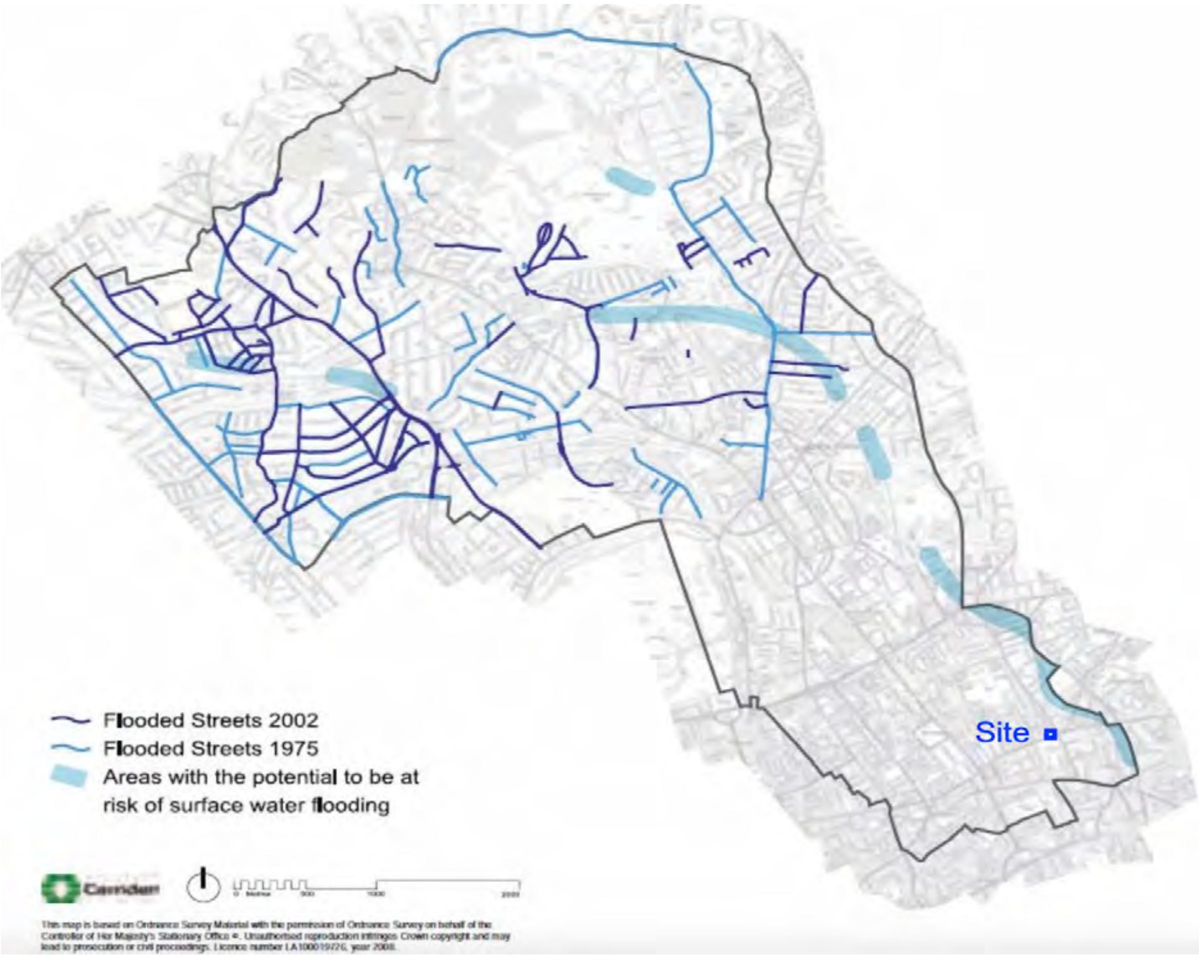


Figure 3: LB Camden's SFRA (reproduced from Figure 15) – Surface Water Flood Risk

[2] https://check-long-term-flood-risk.service.gov.uk/map?easting=530943&northing=182002&map=SurfaceWater

Reservoir

The Long-Term Flood Risk Map for Reservoir Flooding (Figure 4) does not show the subject property to be in the extent of flooding that could occur in the event of breach failure of a reservoir. This is considered to be the largest area that might be flooded if a reservoir were to fail and release the water it holds. Since this is a prediction of a credible worst-case scenario, it's unlikely that any actual flood would be this large.

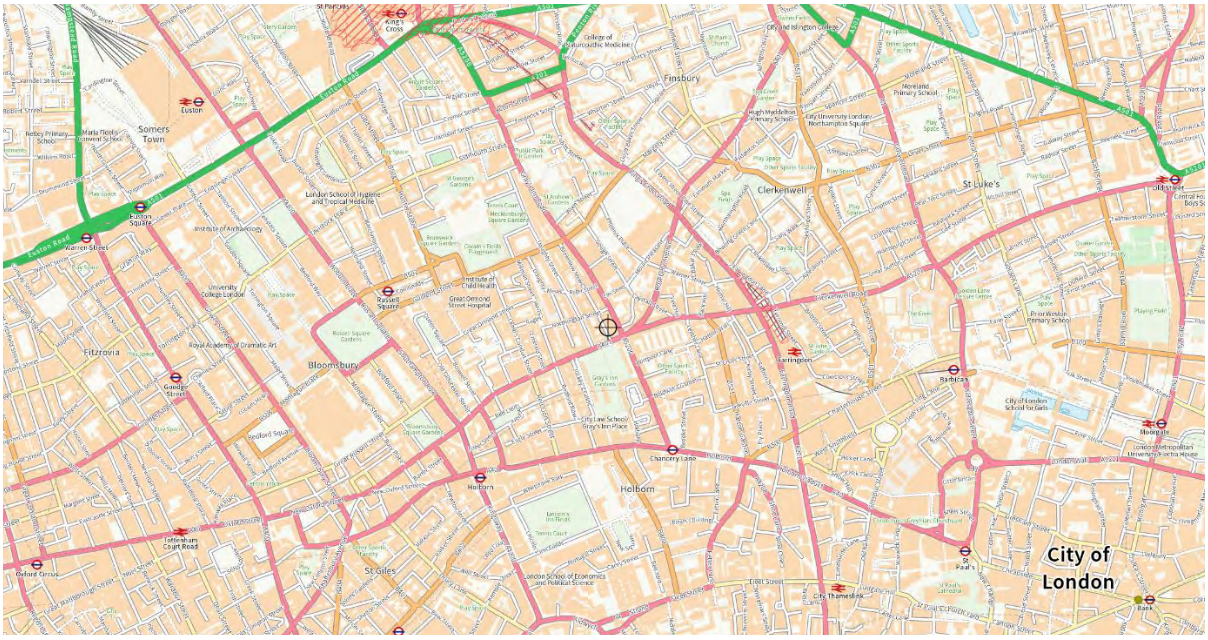


Figure 4: Long-Term Flood Risk Map - Reservoir[3]

Groundwater

A desk top study has been undertaken to review online data sets. British Geological Survey (BGS) maps record superficial deposits at the property location as Lynch Hill Gravel Member comprising Sand and Gravel and show bedrock geology to be London Clay Formation comprising Clay, Silt and Sand. The bedrock is designated[4] as 'unproductive'. The superficial drift is designated as a 'Secondary A' aquifer which is defined *as* 'Permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as minor aquifers'. The superficial drift aquifer designation status relating to groundwater vulnerability[5] is 'Low'.

The property is not located within a groundwater source protection zone.

Soilscape[6] mapping shows the property to be in an area with 'freely draining slightly loamy soils' that is 'freely draining' to 'local groundwater and rivers'.

[3] https://check-long-term-flood-risk.service.gov.uk/map?easting=530943&northing=182002&map=Reservoir

[4] https://data.gov.uk/dataset/616469ae-3ff2-41f4-901f-6686feb1d5b6/aquifer-designation-dataset-for-england-and-wales

[5] https://data.gov.uk/dataset/42d7d021-538c-46e2-abbb-644e01c63551/groundwater-vulnerability-maps-2017-on-magic

[6] http://www.landis.org.uk/soilscapes/#

The nearest BGS borehole records are located 55m to the southeast[7] and 90m to the northwest[8] of the site dating from 1859 and 1908 respectively. Both confirm the superficial and bedrock geology mapping referred to above with made ground over sand and gravel to a depth of 20ft 6in (6.248m) over clay to the southeast and Made Ground over loamy sand and gravel to a depth of 21ft (6.4m) to the northwest. The BGS borehole records do not make reference to groundwater.

Figure 4e in the SFRA[9] presents a map showing areas where there is an '*Increased Potential for Elevated Groundwater*'. The property is not located within such an area. The map also shows the locations of historic flooding from groundwater sources and Environment Agency groundwater flood incidents. The property is similarly not in proximity to these areas with the nearest being at High Holborn, approximately 0.58km to the southwest.

Intrusive ground investigation was undertaken as part of proposals for works at the adjacent property, no 25 Kings Mews. An extract from the Desk Study and Ground Investigation Report (GIR) by GEA[10] is included in the Basement Impact Assessment that supported the planning application[11]. GEA's borehole logs do not record groundwater and the GIR advises that groundwater has been recorded in the Lynch Hill Gravel during monitoring visits at depths of 3.90m and 4.20m bgl.

Groundwater records from the site investigation and adjoining developments indicate groundwater at the site between 3.00m and 4.00m bgl. This is at or just above the formation level of the proposed basement and as such there is potential for groundwater ingress during construction, to be managed by localised groundwater control methods. Ingress and flood risk to the permanent basement structure is negligible, considering the proposed structural waterproofing.

As Section 3.3, there will be no impacts (or cumulative impacts) to the wider hydrogeological environment due to the proposed basement.

Sewer

Thames Water sewer records show that there are combined water sewers in the area with the nearest located below the road Kings Mews to the west of the property. This is shown to be 381mm in size with an approximate depth of up to 4.00m bgl (ground level 20.81mOD and invert level of 15.64mOD at intersection of Northington Street and Kings Mews where sewer changes from 381mm to 1,372mm x 864mm). There are no local sewer flooding incidents recorded (LB Camden's SFRA, Figures 5a and 5b).

4.2 Risk of Flooding to and from the Development

From a review of the sources of flooding presented in the foregoing, it is considered that there is a low risk of flooding from all sources.

The predicted effects of climate change generally result in exacerbation of current day flooding due to increases in the rate and volume of flood water that can occur and the reduced frequency of flood events.

---

[7] BGS Reference TQ38SW156

[8] BGS Reference TQ38SW143

[9] LBC SFRA Report by URS, ref 47070547, Rev 2, dated July 2014

[10] Geotechnical & Environmental Associates Ltd, Report Ref J12150, Issue 1, 7th August 2012

[11] London Borough of Camden Council planning application reference 2012/0972/P

However, it is not considered that the effects of climate change will significantly alter the potential for flooding from the sources discussed other than locally in respect of surface water run-off management.

It follows that mitigation measures other than those inherent to standard building practice are not required. However, a drainage strategy should be considered in line with best practice and appropriate polices.

4.3 Drainage Strategy

Chapter 9 of The London Plan 2021 includes Policy SI 13 relating to Sustainable Drainage. It presents the following drainage hierarchy:

1) rainwater use as a resource (for example rainwater harvesting, blue roofs for irrigation);
2) rainwater infiltration to ground at or close to source;
3) rainwater attenuation in green infrastructure features for gradual release (for example green roofs, rain gardens);
4) rainwater discharge direct to a watercourse (unless not appropriate);
5) controlled rainwater discharge to a surface water sewer or drain;
6) controlled rainwater discharge to a combined sewer.

The SFRA provides guidance in relation to surface water management. Figure 4c of the SFRA presents a map showing the infiltration potential across LB Camden based on BGS data. The property is in an area shaded to signify '*probably compatible for infiltration SuDS*'.

There are no paved areas associated with the development which is restricted to the curtilage of the property that comprises only the existing building structure. The footprint of the building and therefore the plan area of roof will not change as a result of the development.

The roof presents the most realistic opportunity for betterment of the current surface water regime and to allow for mitigation in changes to runoff over the course of the lifetime due to the predicted effects of climate change. Green, brown and blue roofs all offer differing degrees of interception and attenuation capacity. The proposed roof form is to be a flat terrace rather than traditional non-accessible pitched roof area and therefore the viability of surface water management at roof level should be considered. However, it is noted that the paved flat roof to No. 27 is existing and will be retained without change as a result of the proposed development, whilst the roof area to No. 26 will be new and is intended to provide an extension to the existing paved flat roof area.

The proposed office use of the building may merit the use of rainwater harvesting to minimise the need for potable water for toilet flushing and a viability assessment should be undertaken to confirm supply and demand data together with technical and financial considerations such as whether a high level tank can be incorporated to allow a gravity storage system given the lack of below ground area for an underground tank. The use of rainwater harvesting is a method of source control that provides good interception but cannot be relied on for management of extreme events where high intensity or prolonged rainfall occurs. Therefore, the need to implement another form of SuDS technique may be required to balance discharge from the property drainage system so that the status quo of existing flow is maintained or ideally reduced.

The drainage system should also be appraised for the effects of climate change over the lifetime of the development. Current guidance for peak rainfall intensity increase allowances states that drainage system should be design to make sure there is no increase in the rate of runoff discharged from the site for the upper end allowance. Planning Practice Guidance for the National Planning Policy Framework advises[12] that '*the lifetime of a non-residential development depends on the characteristics of that development but a period of at least 75 years is likely to form a starting point for assessment*'. On this basis, the central allowance for the 2070s epoch (2061 to 2125) of 25% should be applied to 1 in 100 year rainfall intensities when assessing the drainage system.

The existing roof area to Nos. 26 and 27 Kings Mews is approximately 168sqm and will not increase as a result of the development. It is expected that the existing drainage is unrestricted and as such, a pre-development discharge rate of approximately 2.3l/s would occur under a rainfall intensity of 50mm/hr. Section 9.13.12 of The London plan 2021 advises that '*development proposals should aim to get as close to greenfield run-off rates as possible depending on site conditions*'. LBC Local Plan Policy CC3 also advises that development is required to '*utilise Sustainable Drainage Systems (SuDS) in line with the drainage hierarchy to achieve a greenfield run-off rate where feasible*'.

Drainage calculations are presented in Appendix 5. A greenfield runoff rate of qbar = 1.6l/s/ha (approx. 1 in 2 year) has been determined, which for the overall site area of approximately 0.017ha (168sqm) is equivalent to 0.027l/s. This is a very low rate that would not be practical to achieve due to the low size of flow control aperture that would be needed which would be inherently susceptible to blockage.

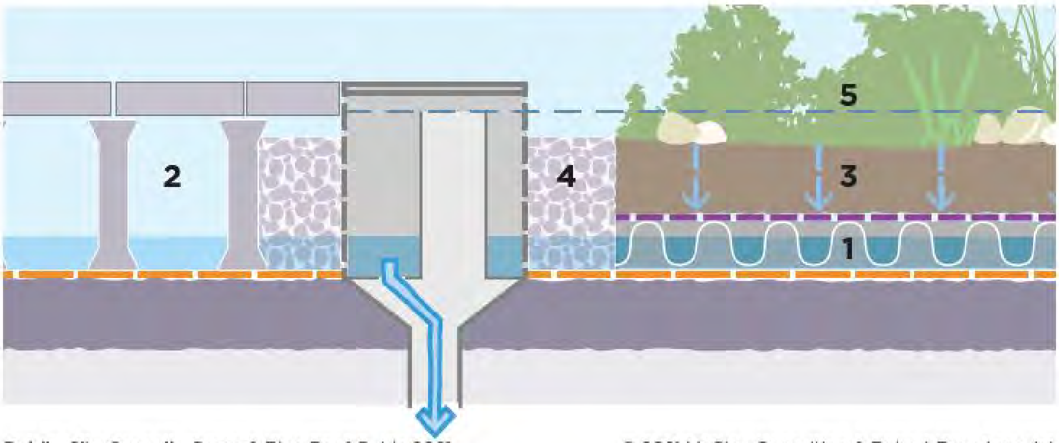
Therefore, a lowest practical flow rate should be used, in this case allowing for an orifice size of 50mm diameter. This is to suit the use of attenuation at roof level via void space created either with a pedestal or crated system (examples from Bauder and Alumasc literature included in Appendix 5). The following illustration (Figure 5) taken from guidance by McCloy Consulting & Robert Bray Associates shows typical construction profiles for both blue and green roofs. Given the proposed paved surfacing shown on the application drawings, it is expected that the blue roof detail will be compatible.

The appended drainage calculations demonstrate that a roof area of 5m wide x 12m long (ie not including the existing roof area to no 27 and allowing a margin) with a storage layer of 150mm (95% voids) will be adequate to balance runoff from both roof areas (up to 170sqm) under 1 in 100 year + 25% rainfall intensities and without exceeding 1.8l/s discharge rate (ie approx. 22% reduction of the pre development rate of 2.3l/s determined above).

In principle, the above is a viable drainage strategy that demonstrates that a blue roof form of construction with attenuation via a crated or pedestal layer and simple orifice flow control can manage runoff for 1 in 100 year + 25% rainfall and restrict discharge to approximately 78% of the pre-development peak rate.

[12] https://www.gov.uk/guidance/flood-risk-and-coastal-change#para6

Typical construction profiles.



Dublin City Council Green & Blue Roof Guide 2021

© 2021 McCloy Consulting & Robert Bray Associates

Note: Void porosity figures presented below are provided as initial guidance and detailed figures should be informed by manufacturers specification.

1. Drainage board / Reservoirs – assume reservoirs are full for purposes of attenuation storage calculation (50% voids)
2. Storage layer provided by geocellular structures or pedestals – as per manufacturers specification (90-95% voids)
3. Growing medium layer (15-25% voids)
4. Stone (30% voids)
5. Where flows are stored above the vegetation layer (100% voids), the designer should confirm that:
   - the growing medium will not be subject to flotation
   - surface ponding on the roof will only come into effect during extreme rainfall and any ponding will dissipate after a short time period

Figure 5: Illustration of Blue and Green Roofs

4.4 FRA and Drainage Strategy – Non-Technical Summary

From a review of the sources of flooding that could influence the proposed works at 26 & 27 Kings Mews, it has been determined that there is a very low risk of flooding to the development. It is not considered that the proposals would result in an increased risk of flooding at the property location or surrounding area or that the effects of climate change will significantly change the current day regime.

The management of surface water will be undertaken utilising attenuation SuDS to improve the off-site run-off rate.

There will be no impacts (or cumulative impacts) to the wider hydrological and hydrogeological environments due to the proposed basement.

5.0 Ground Movement Assessment

5.1 Introduction

On the basis of the records from the construction of the existing basement at 27 Kings Mews and the adjacent basements at 25 and 28 Kings Mews, existing Party Wall foundations for the building are already at the same depth as the proposed formation level of the basement at 26 Kings Mews. Deep foundations to both the subject building and neighbouring buildings will reduce both the magnitude of ground movements generated by the proposed basement and the impacts of those movements on nearby structures.

On that basis, by inspection, damage to neighbours is likely to be Negligible (Category 0 in accordance with the Burland scale). Indeed for 25 Kings Mews, with the existing basement comprising reinforced concrete liner walls, basement slab and ground floor slab, with a formation level at or very close to the proposed formation level of the proposed basement, no further assessment is considered necessary. Similarly, the existing basement at 27 Kings Mews will entirely shield 28 Kings Mews (which also has a basement) from ground movements and no further assessment is considered necessary.

In order to undertake a conservative assessment, foundations to the other neighbouring buildings have been assumed to be shallow for the purposes of the BIA.

5.2 Assessment Methodologies

A ground movement assessment (GMA) has been completed utilising industry standard software (Oasys XDisp). Using the data from the analysis, an assessment has been made of the potential impact on neighbouring buildings in accordance with the Burland Scale. Calculations and GMA outputs are provided in Appendix 6.

5.3 Ground Movements Generated by Proposed Development

The following construction processes are likely to give rise to the majority of ground movements:

1. Installation of the underpins.
2. Excavation of the new basement.

It should be noted that based on the existing basement structures, underpinning of the flank Party Walls is not required, with only the front and rear walls requiring underpinning. However, in order to generate a conservative range of movements, for the purposes of assessment all walls have been adopted as being underpinned.

The top of the proposed basement slab will be 3.35m bgl. The basement slab including blinding is 500mm. Therefore, a formation level of 3.85m bgl has been adopted for assessment. Underpinning will be undertaken in a single lift. The structural information indicates the temporary and permanent works will be stiffly propped at all times and the assessment adopts ‘high stiffness’ parameters.

Based on the guidance provided in CIRIA C760 for embedded retaining walls, ground movements resulting from installation of underpinned walls and excavation in front of the walls have been estimated. Whilst its noted that the guidance is intended for use with embedded walls, the methodology provides predicted ranges of movement that are consistent with movements generated during underpinning.

In order to be conservative, the depth of existing foundations has been ignored (with the exception of 25 and 28 Kings Mews, as detailed in the preceding section) and the depth of underpinning and excavation has been taken from ground level. This approach should over-estimate movements compared to those generated by the actual works.

For movement due to the underpin installations, the magnitudes of the movements are dependent on the total retaining wall depth. Maximum vertical movements occur at the wall itself. C760 indicates movements will be 0.05% of the wall depth, with negligible vertical movement at one and a half times the wall depth from the wall. On this basis, maximum vertical movements due to wall installation of <2mm are predicted with vertical movements extending to a maximum of <6m from the wall.

Anticipated maximum horizontal movements due to wall installation are 0.05% of the wall depth, with negligible horizontal movement one and a half times the wall depth from the wall. Maximum horizontal movements are therefore predicted to be <2mm with horizontal movements extending to a maximum of <6m from the wall.

For movements due to excavation in front of the retaining wall, the magnitudes of the movements are dependent on the excavation depth. Based on the Contractor adopting a stiffly propped method of excavation, C760 indicates maximum vertical movements of 0.10% of excavation depth, with negligible movement three and a half times excavation depth from the wall. Maximum vertical movements due to excavation of <4mm are predicted, extending <14m from the wall.

Anticipated maximum horizontal movement due to excavation are 0.15% of the excavation depth, with negligible horizontal movements four times the excavation depth from the wall. Maximum horizontal movements are predicted to be <6mm, extending <16m from the wall.

A summary of ground movement predictions obtained using Oasys XDisp are reported in Appendix 7, presented as contour plots. The calculations take account of the combined vertical and horizontal movements from both installation and excavation. The predicted ground movements are at ground level.

5.4 Adjacent Structures, Highway and Utility Assets

The buildings identified as being within the potential zone of influence from the proposed basement construction works:

- 2 Kings Mews
- 4 Kings Mews
- 25 Kings Mews
- 28 Kings Mews
- 39 – 45 Gray's Inn Road

The potential damage impacts to the buildings within the zone of influence have been assessed. A indicated in 5.1, the full depth reinforced concrete basements at 25, 27 and 28 Kings Mews are considered to mitigate risk of damage to those properties.

The highway (with underlying utilities) is located <1.0m from the proposed basement at the closest point. The most sensitive utilities to movement are considered to be the water mains and sewers. The other utilities are considered to be relatively flexible.

Although not integral to the purpose of this assessment, it should be noted that during the construction works the adjacent structures will be monitored for movements as required by Party Wall Agreements and any highway or utility asset protection agreements. The results of this monitoring provide a comprehensive feedback loop to the assessment models. This will allow contingency actions to be undertaken, if necessary, to limit movements.

5.5 Sensitivity Analysis

To provide a sensitivity check of the methodology adopted, the movement values predicted have been compared with:

- the typical range of movements reported by underpinning contractors, which is between 5mm and 10mm vertical / horizontal for an underpin constructed in a single lift;
- consideration of a 'low stiffness' construction methodology (i.e. without the use of temporary propping to restrain movements), which indicates approximately 16mm to 18mm vertical / horizontal movements (if ignoring the depth of existing foundations). The conservative 'low stiffness' range of movements could be considered a worst-case scenario, if propping was omitted for instance.

5.6 Estimates of Ground Movement using Oasys XDisp

Whilst the CIRIA C760 approach is considered conservative, it has been adopted as the underlying method of analysis precisely for this reason: the actual ground movements generated during the works should be less onerous than those predicted. The geometries of the site have been imported into XDisp and ground movements modelled based on C760.

The displacement profiles and damage assessments derived using XDisp assume greenfield movements and predict movements at ground level. In relation to all buildings, the movements derived will be an overestimate of movement both with respect to adjacent foundations and assets, which are located at a depth greater than existing street levels. The XDisp contour outputs are reported in Appendix 6.

5.7 Estimates of Movement due to Settlement / Heave

The excavation of a maximum 3.85m of soil will generate an unloading of around <80kPa. Given that the new building will have a ground bearing basement slab with bearing pressure of between 50 to 100kPa, based on the structural information received to date, this will result in a very small net change in loading.

A proportion of the soil heave pressure will be dissipated in the short term / during excavation, before the base slab is cast and structural loads imposed, due to undrained deformation and other short term effects. In the long term, as the clay swells, the base slab will have a pressure exerted on it.

The CIRIA C760 calculations, as empirically derived formulae, are considered to include the short term heave / settlement movements. In the long term, the net change in loading is considered negligible and no further movements of a magnitude that could generate impacts to surrounding structures are likely.

Experience suggests that heave movements tend largely to be restricted to within the basement excavation when excavations are created within embedded retaining walls. Whilst no embedded walls will be utilised, the existing adjacent basements will mitigate heave movements to an extent around the perimeter of the basement, so it is not anticipated that the changes in loading at basement level will have a significant impact on the neighbouring structures.

5.8 Impact Assessment of Neighbouring Buildings, Highway and Utilities

The ground movements have been used to assess the resultant potential damage that may be experienced by neighbouring structures. The methodology proposed by Burland and Wroth, and later supplemented by the work of Boscardin and Cording, has been used, as described in CIRIA C760 (and preceding CIRIA publications). The 'Burland Scale' damage categories are presented in Table 1.

Based on the ground movements calculated, the following impacts are predicted in accordance with the Burland Scale:

- 2 Kings Mews – Category 0 (Negligible)
- 4 Kings Mews – Category 0 (Negligible)
- 25 Kings Mews – Category 0 (Negligible)
- 28 Kings Mews – Category 0 (Negligible)
- 39 – 45 Gray's Inn Road – Category 1 (Very Slight)

The maximum movements predicted to be experienced at the highway are 4mm vertically / 8mm horizontally. This magnitude of movement will cause negligible impact to surfacing or underlying utilities.

It is recommended that structural movement monitoring is undertaken during the works and mitigation actions implemented if movement trends indicate predicted impacts and structural movement tolerances could be exceeded.

Category of damage	Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain, ε_{lim} (%)
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible	<0.1	0.0 to 0.05
1 Very slight	**Fine cracks that can easily be treated during normal decoration.** Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection	<1	0.05 to 0.075
2 Slight	**Cracks easily filled. Redecoration probably required.** Several slight fractures showing inside of building. Cracks are visible externally and **some repointing may be required externally** to ensure weathertightness. Doors and windows may stick slightly.	<5	0.075 to 0.15
3 Moderate	**The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable lining. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.** Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15 or a number of cracks >3	0.15 to 0.3
4 Severe	**Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.** Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Services pipes disrupted.	15 to 25, but also depends on number of cracks	>0.3
5 Very severe	**This requires a major repair, involving partial or complete rebuilding.** Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	Usually >25, but depends on numbers of cracks	

Table 1: Damage Categories on the Burland Scale

6.0 Basement Impact Assessment

The purpose of this assessment is to consider the potential impacts from basement development on the local hydrology, geology and hydrogeology and any resulting impacts to stability of adjacent structures. The assessments have been undertaken by appropriately qualified professionals in accordance with the guidance.

6.1 Geology and Land Stability

The site is underlain by the Lynch Hill Gravel Member and the London Clay Formation. These ground conditions provide a suitable bearing capacity for the proposed development's foundations. This has been confirmed by the site investigation.

The risk of movement and damage to this development due to shrink and swell of the London Clay is negligible, considering the depth of the proposed foundations.

Ground movements caused by the excavation and construction of the proposed development have been demonstrated by assessment to be minimal, considering the adoption of best practice construction methodologies and stiff propping of the basement. Damage Impact to adjacent structures will be limited to a maximum of Very Slight (Category 1 in accordance with the Burland Scale). It is recommended that structural movement monitoring is undertaken and mitigation actions implemented if ground movement trends indicate structural movement tolerances could be exceeded.

Movements to the highway / utilities are considered to be very small, such that they would cause negligible impact. Consultation with relevant asset owners is recommended to ensure that appropriate design and mitigation measures can be provided for the development such that impacts to the highway and utilities are maintained within the agreed limits.

6.2 Hydrogeology and Groundwater Flooding

The Lynch Hill Gravel Member is designated as a Secondary (A) Aquifer; The London Clay is designated as Unproductive Strata. By adopting appropriate structural waterproofing there is a very low risk of groundwater flooding. It has been assessed that there is very low potential for impacting the wider hydrogeological environment or to cause impact through the cumulative effects of basement construction

The detailed Construction Method Statement will require appropriate propping and mitigation measures to be implemented, including the use groundwater control, which will be controlled by the Contractor and supervised by the Engineer. It is unlikely that there will be impacts to stability during construction or in the permanent case as a result of encountering groundwater assuming best practice is adopted.

6.3 Hydrology and Surface Water Flow

The site and the adjacent properties have not been impacted by flooding. There is a very low risk of flooding to the proposed development and the proposed development will not impact the wider hydrological environment. The proposed drainage strategy should provide betterment and reduce the risk of surface water flooding or sewer surcharging on site and in the immediate vicinity.

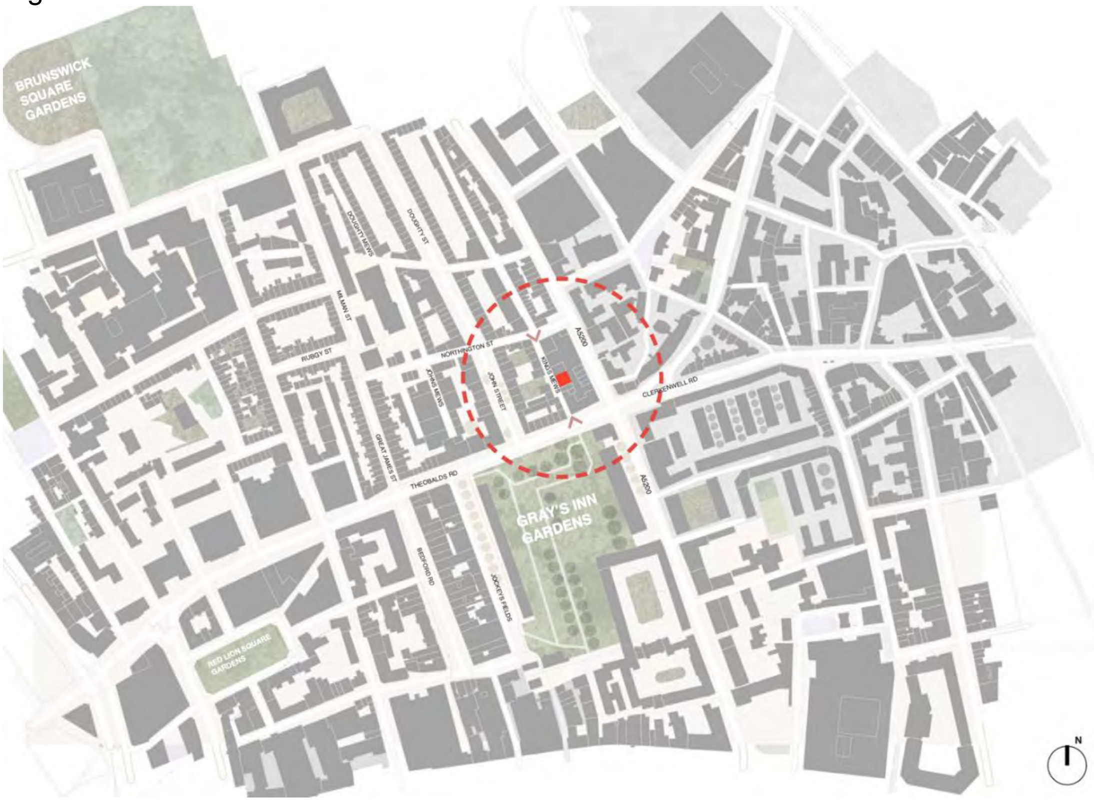
The SuDS proposals allow for a suitable attenuated drainage scheme with off-site discharge flow rates limited to the minimum practicable in accordance with best practice.

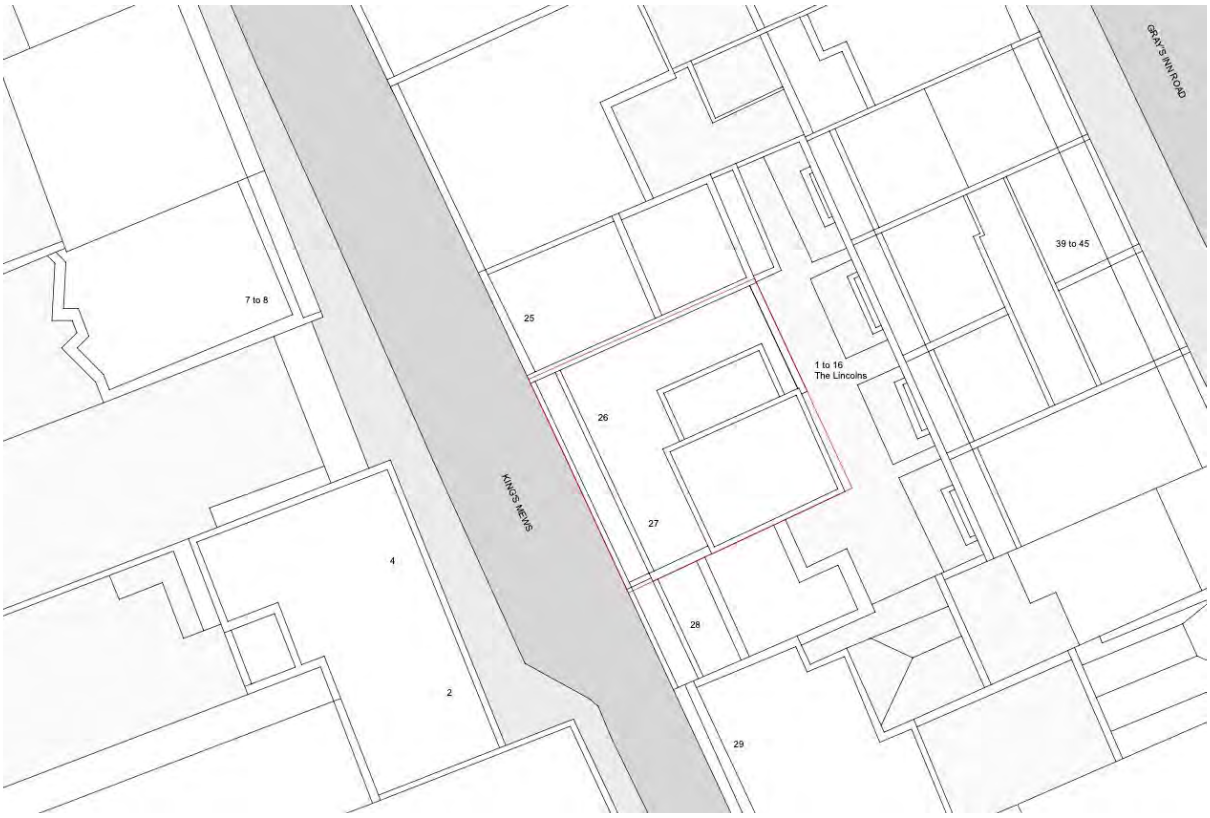
6.4 Residual Risks and Mitigation

As a contingency, and in accordance with best practice, a structural movement monitoring plan should be set out at design stage. Monitoring should include precise levelling, reflective survey targets or other appropriate instrumentation as determined by the Engineer being installed on adjacent structures and the highway. This should be agreed under the Party Wall Act and as part of any asset protection agreements required.

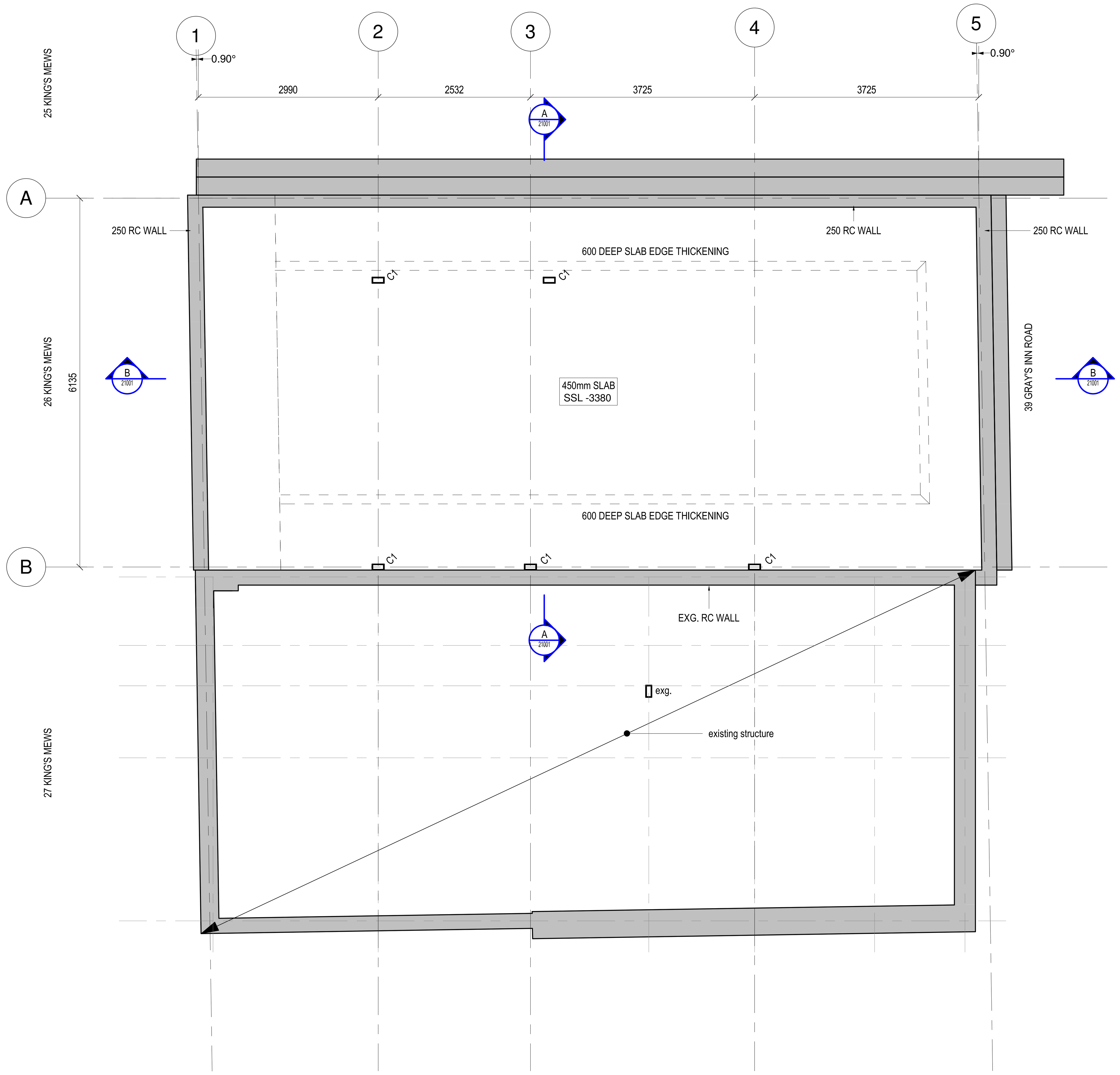
Appendix 1 Site Location

Figure 1 Site Location





Appendix 2 Proposed Structural Drawings



Basement
SCALE: 1 : 50

General Notes

1. All dimensions are in millimetres and levels in metres.
2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.
3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.
4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.
5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

1. All structural timber to be grade C24 and to be tanalised.
2. All structural steelwork to be grade S355 J0 to BS EN 10025 U.N.O.
3. All bolts to be 8.8 and to be zinc plated.
4. Concrete grade to be C30/37
5. Waterproofing by others

COLUMN SCHEDULE

REF.	DESCRIPTION
C1	RHS200x100x10
C2	SHS100x100x10

BEAM SCHEDULE

REF.	DESCRIPTION
B1	UC254x254x107
B2	UC254x254x73
B3	UB254x146x31
B4	L100x100x10
B5	UB203x102x23
B6	L200x100x10
VB1	Cross Bracing 2No. 100 x 8 Plate

LEGEND

REF.	DESCRIPTION
F1	COMFLOR 60 1.2 THICK DECKING WITH 150mm DEEP SLAB A193 MESH TOP LAYER AND H10 BARS IN TROUGH OF DECKING

Rev.	Date	By	Chk.	Details Of Revision
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Proposed Basement General Arrangement

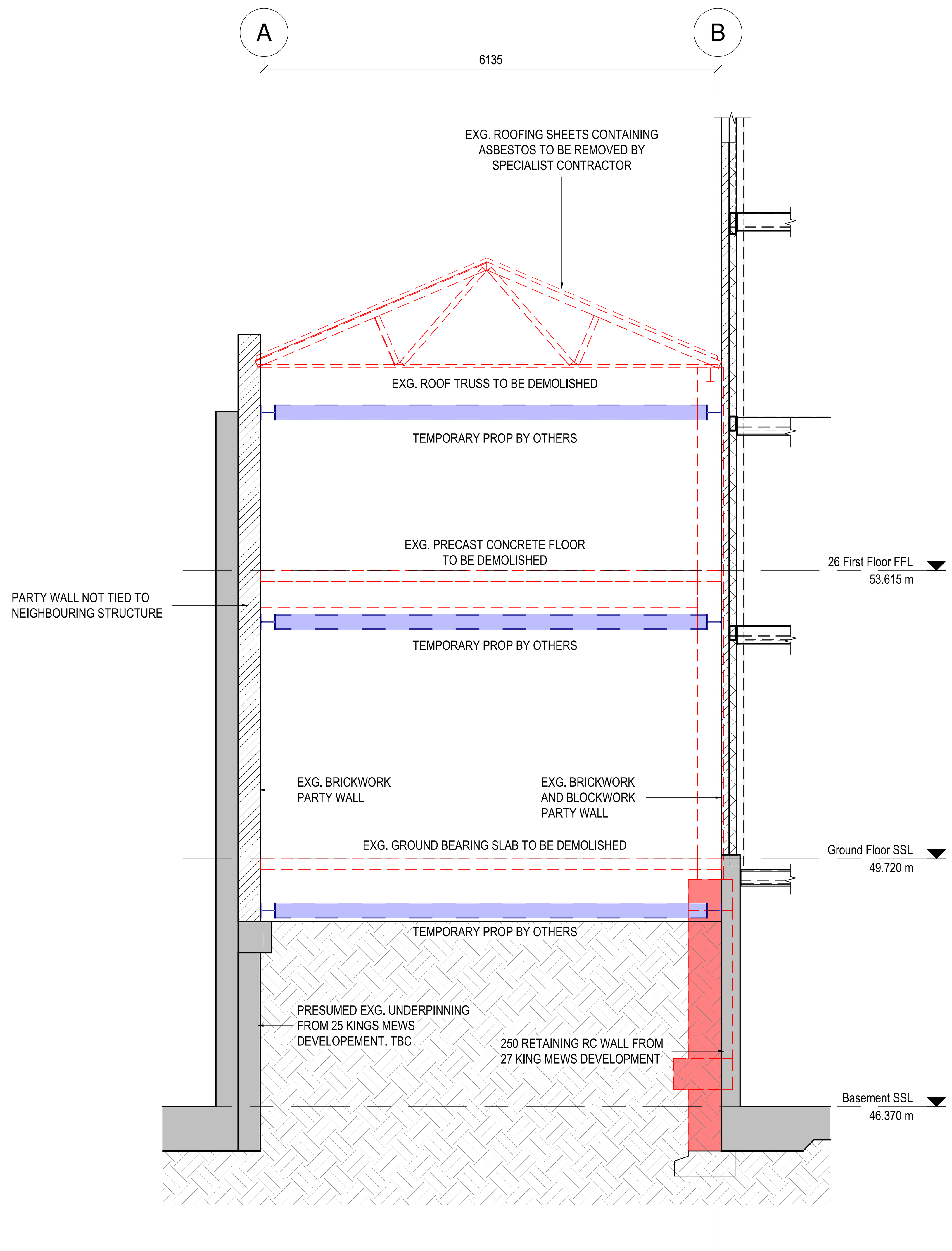
DCL
FORWARD THINKING ENGINEERING
6 Flitcroft Street, London, WC2H 8DJ Tel: 020 7998 5868
Email: admin@dcl.engineering Web: www.dcl.engineering

Status: INFORMATION

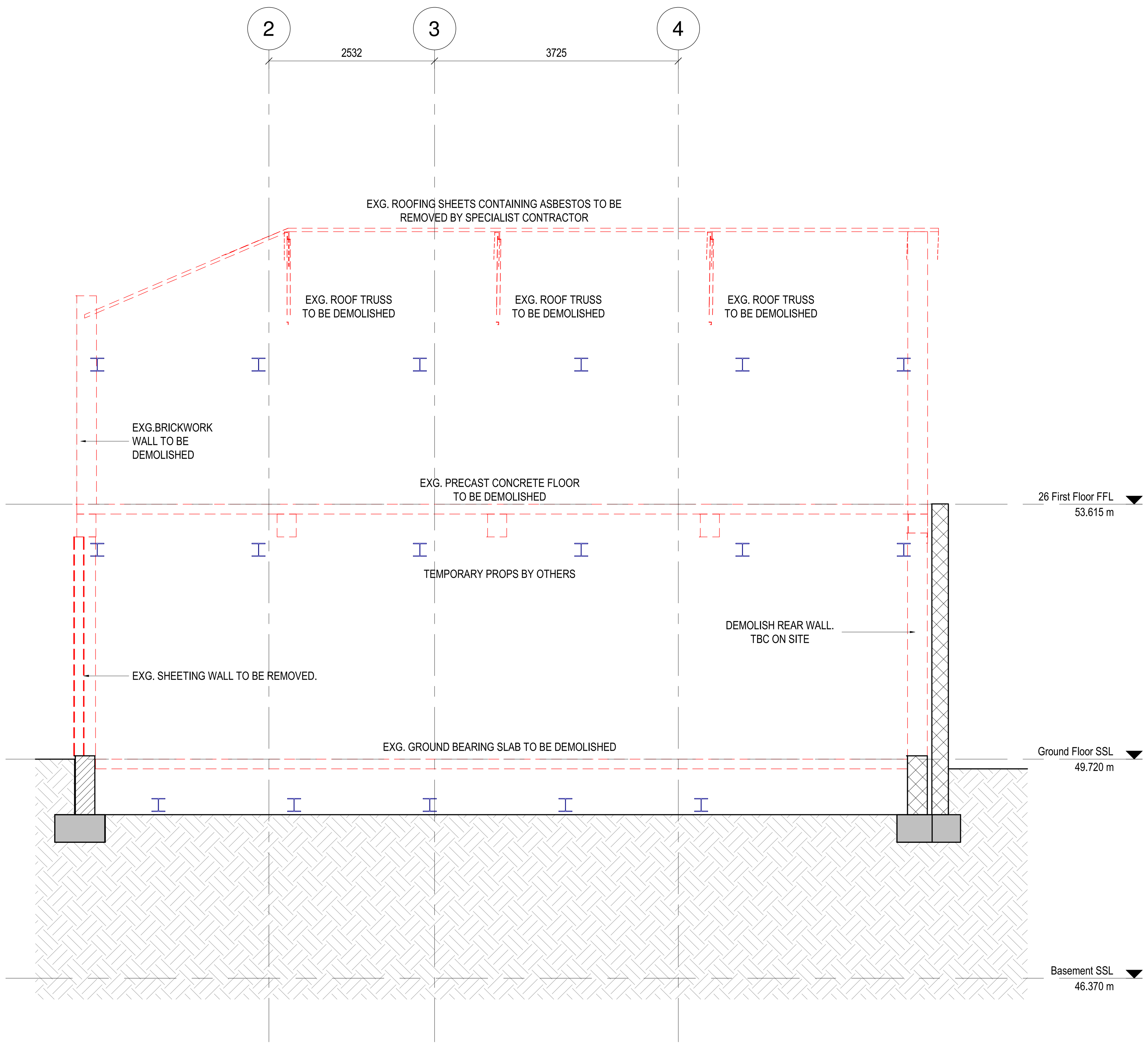
Drawn	A1 Scales		
NG	As indicated		
Date: Oct 2023	Eng: WW	Chk: SA	App'd: AO

ACTUAL DIMENSION = 80mm

Project No.	Drawing No.	Rev.
99737	20B01	P1



Phase 1 Section A-A
SCALE: 1 : 50



Phase 1 Section B-B
SCALE: 1 : 50

General Notes

1. All dimensions are in millimetres and levels in metres.

2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.

3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.

4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.

5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

LEGEND

REF.	DESCRIPTION
	DEMOLISHED
	TEMPORARY WORKS (BY OTHERS)
	NEXT WORK SEQUENCE

Rev.	Date	By	Chk.	Details Of Revision
P2	26.10.23	NG	WW	Coordinated Updates
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Work Sequence Phase 1

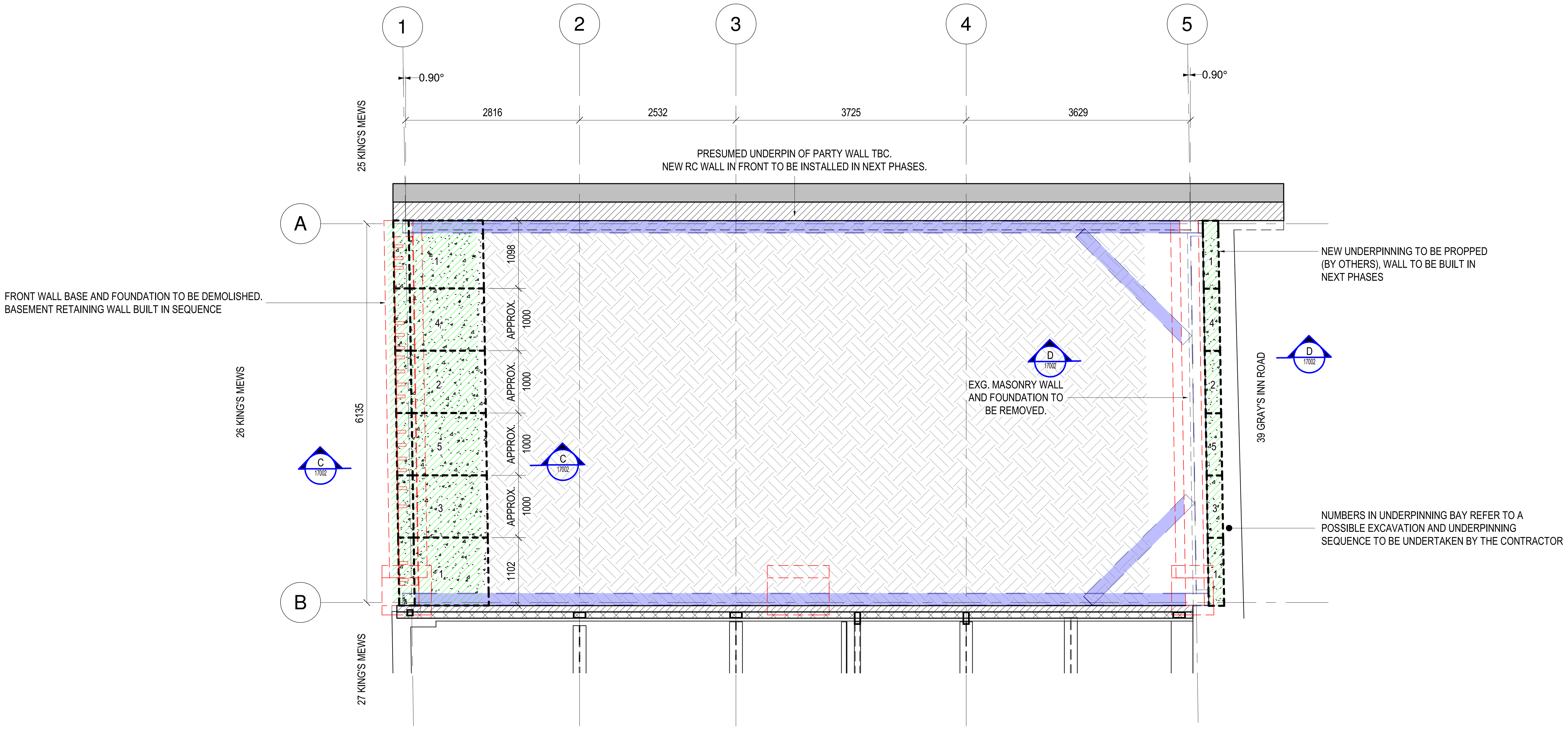
FORWARD THINKING ENGINEERING
6 Flitcroft Street, London, WC2H 8DJ Tel: 020 7998 5868
Email: admin@dcl.engineering Web: www.dcl.engineering

Status: INFORMATION

Drawn	NG	A1 Scales	As indicated		
Date	Oct 2023	Eng. WW	Chk. SA	App'd AO	

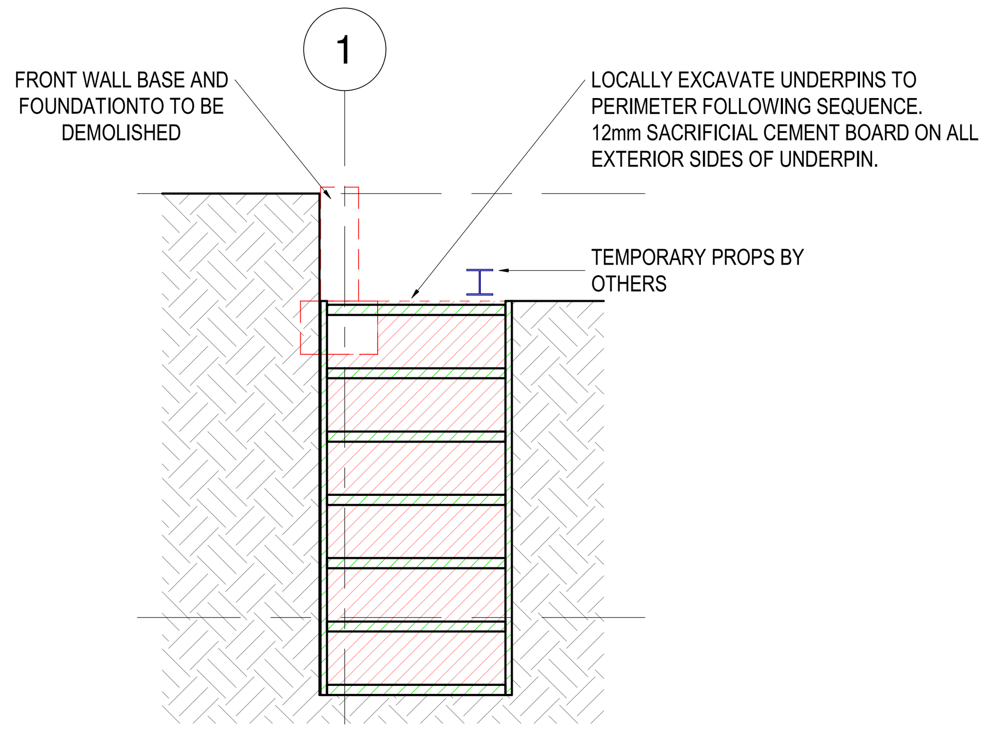
ACTUAL DIMENSION = 80mm

Project No.	Drawing No.	Rev.
99737	17001	P2



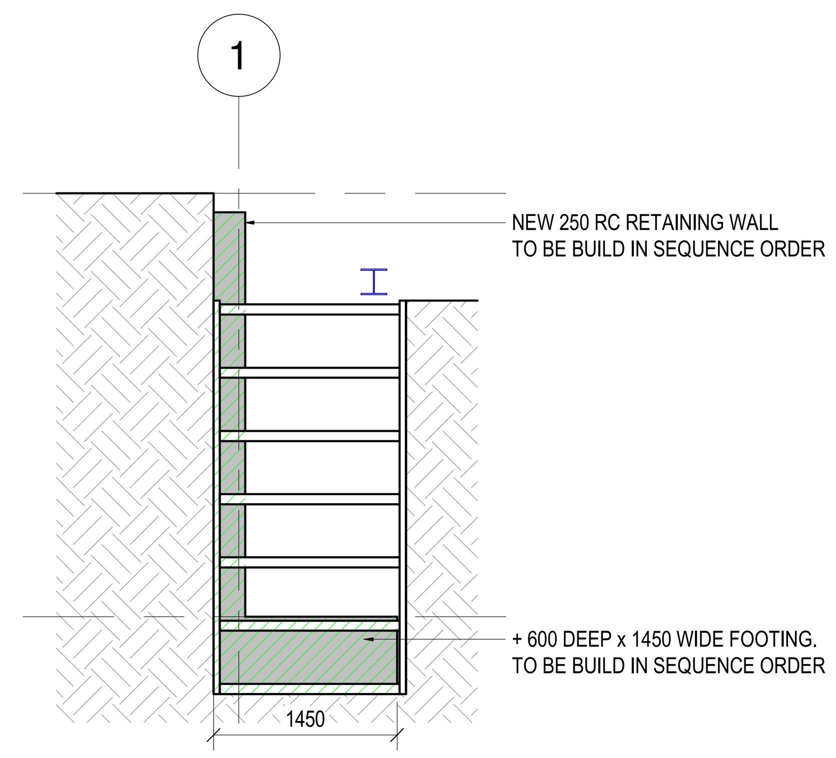
Phase 2 - Underpinning Sequence

SCALE: 1 : 50



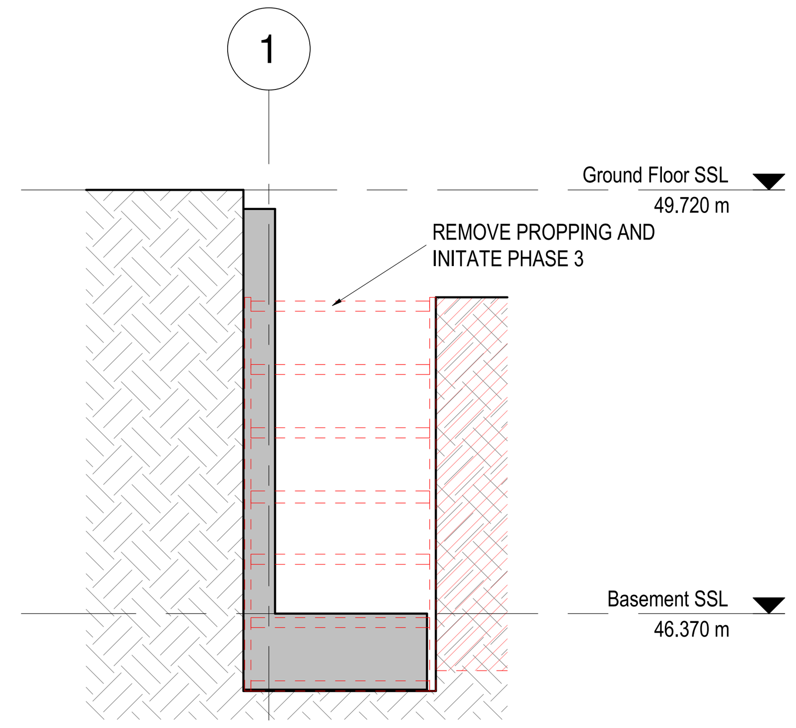
Phase 2.1 Section C-C

SCALE: 1 : 50



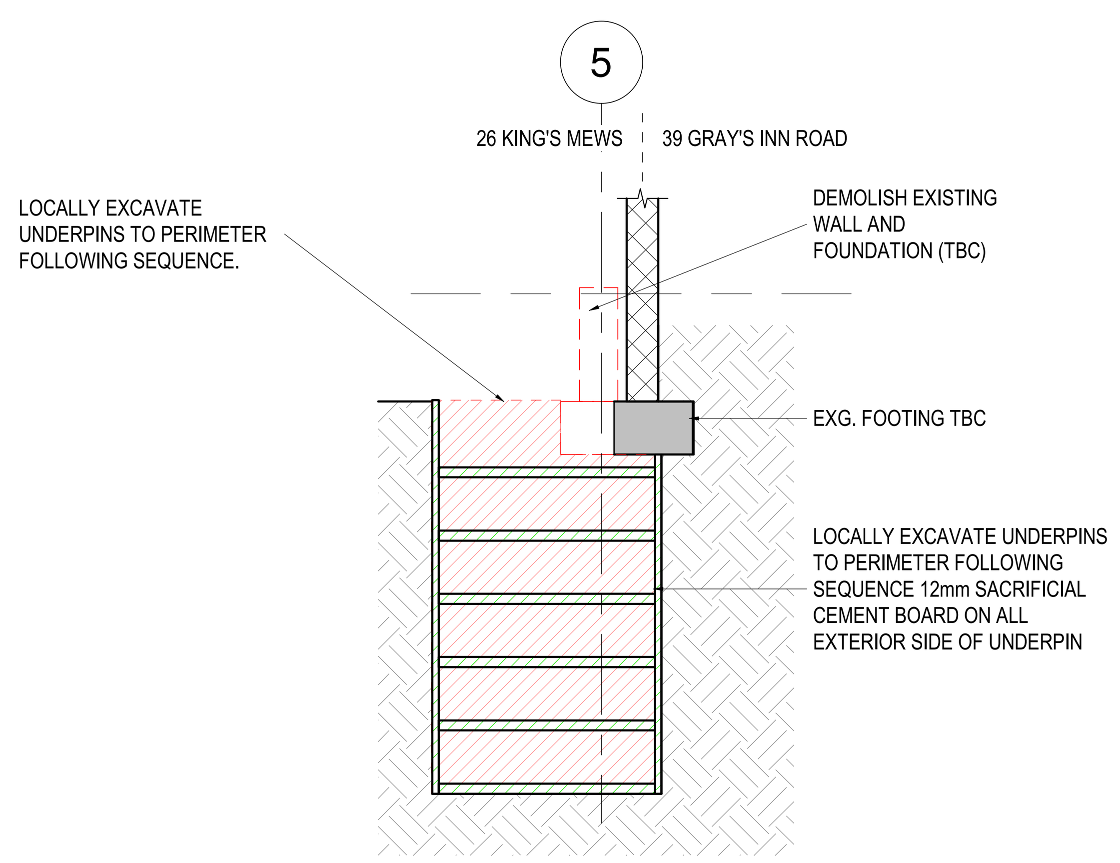
Phase 2.2 Section C-C

SCALE: 1 : 50



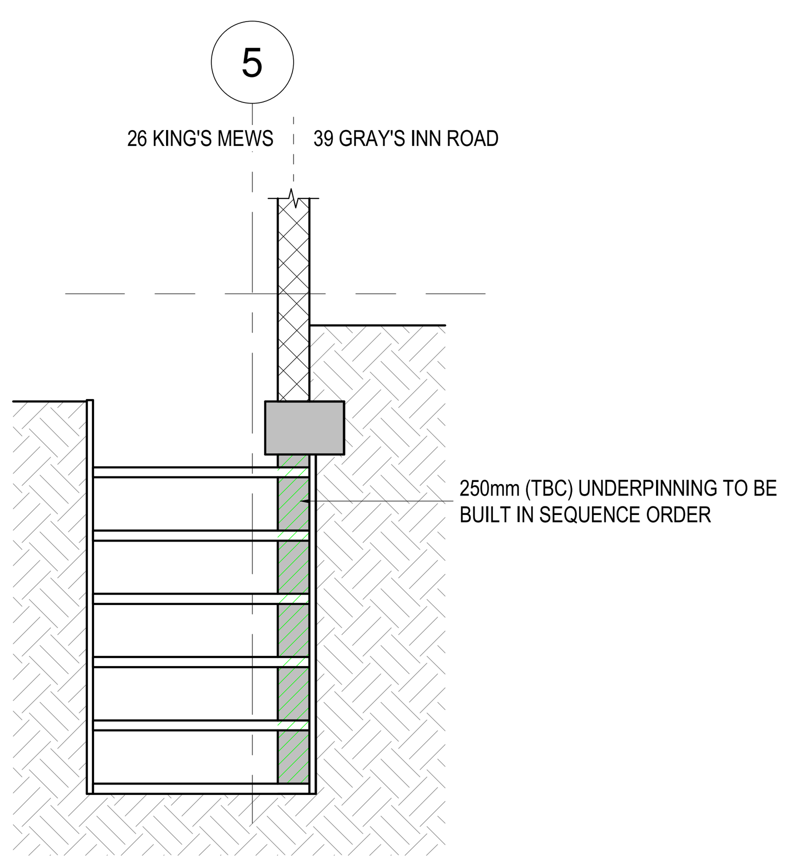
Phase 2.3 Section C-C

SCALE: 1 : 50



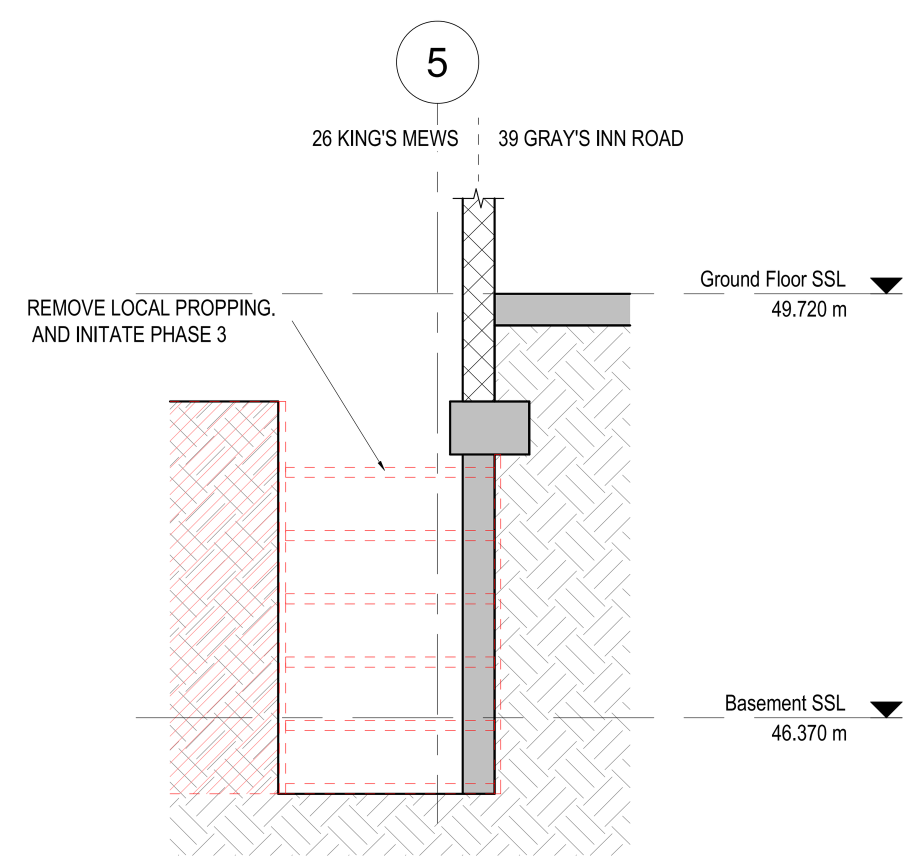
Phase 2.1 Section D-D

SCALE: 1 : 50



Phase 2.2 Section D-D

SCALE: 1 : 50



Phase 2.3 Section D-D

SCALE: 1 : 50

General Notes

1. All dimensions are in millimetres and levels in metres.

2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.

3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.

4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.

5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

LEGEND

REF.	DESCRIPTION
	DEMOLISHED
	TEMPORARY WORKS (BY OTHERS)
	NEXT WORK SEQUENCE

Rev.	Date	By	Chk.	Details Of Revision
P2	26.10.23	NG	WW	Coordinated Updates
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Work Sequence Phase 2

DCL
FORWARD THINKING ENGINEERING

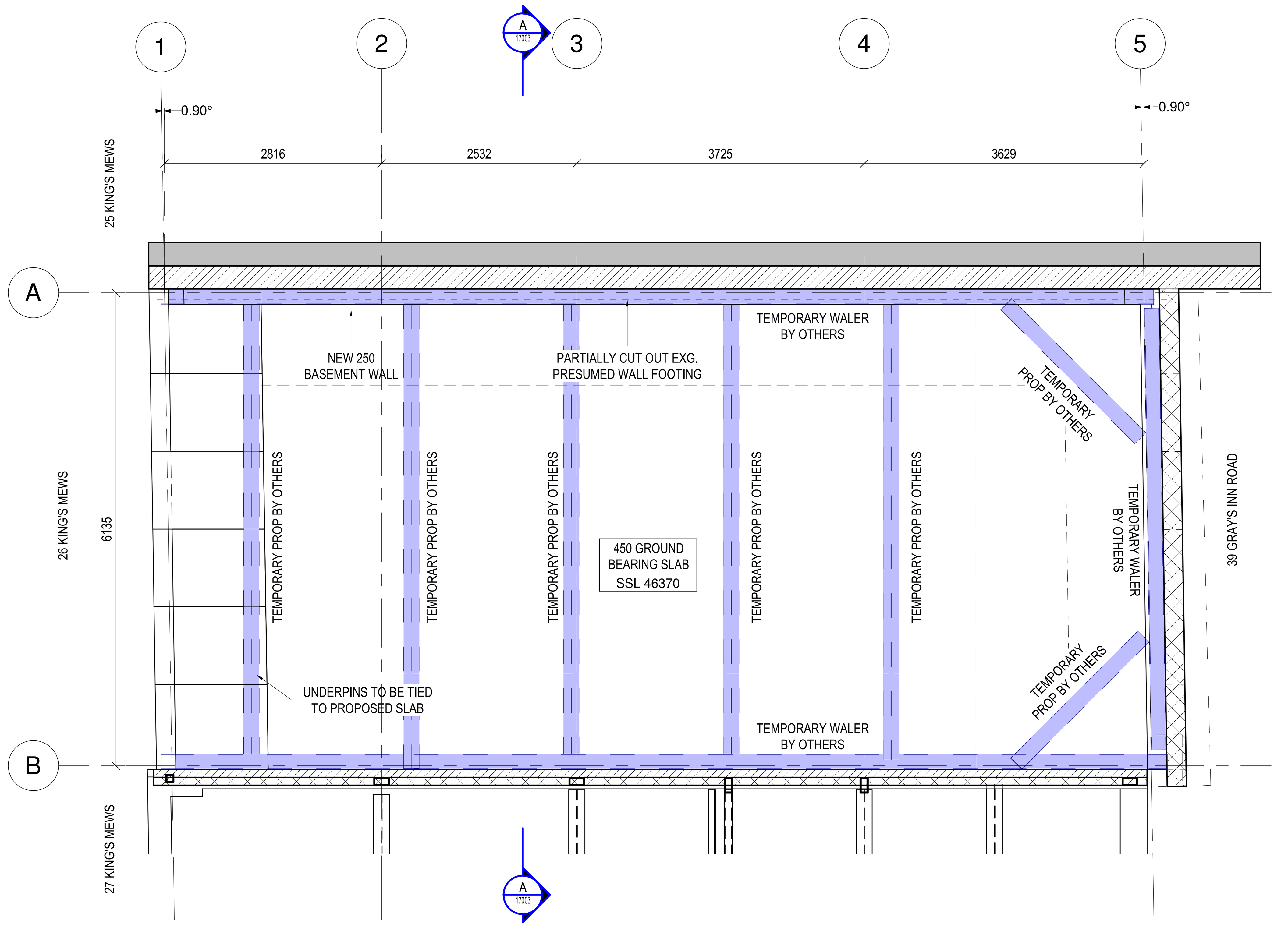
6 Flitcroft Street, London, WC2H 8DJ   Tel: 020 7998 5868
Email: admin@dcl.engineering   Web: www.dcl.engineering

Status: INFORMATION

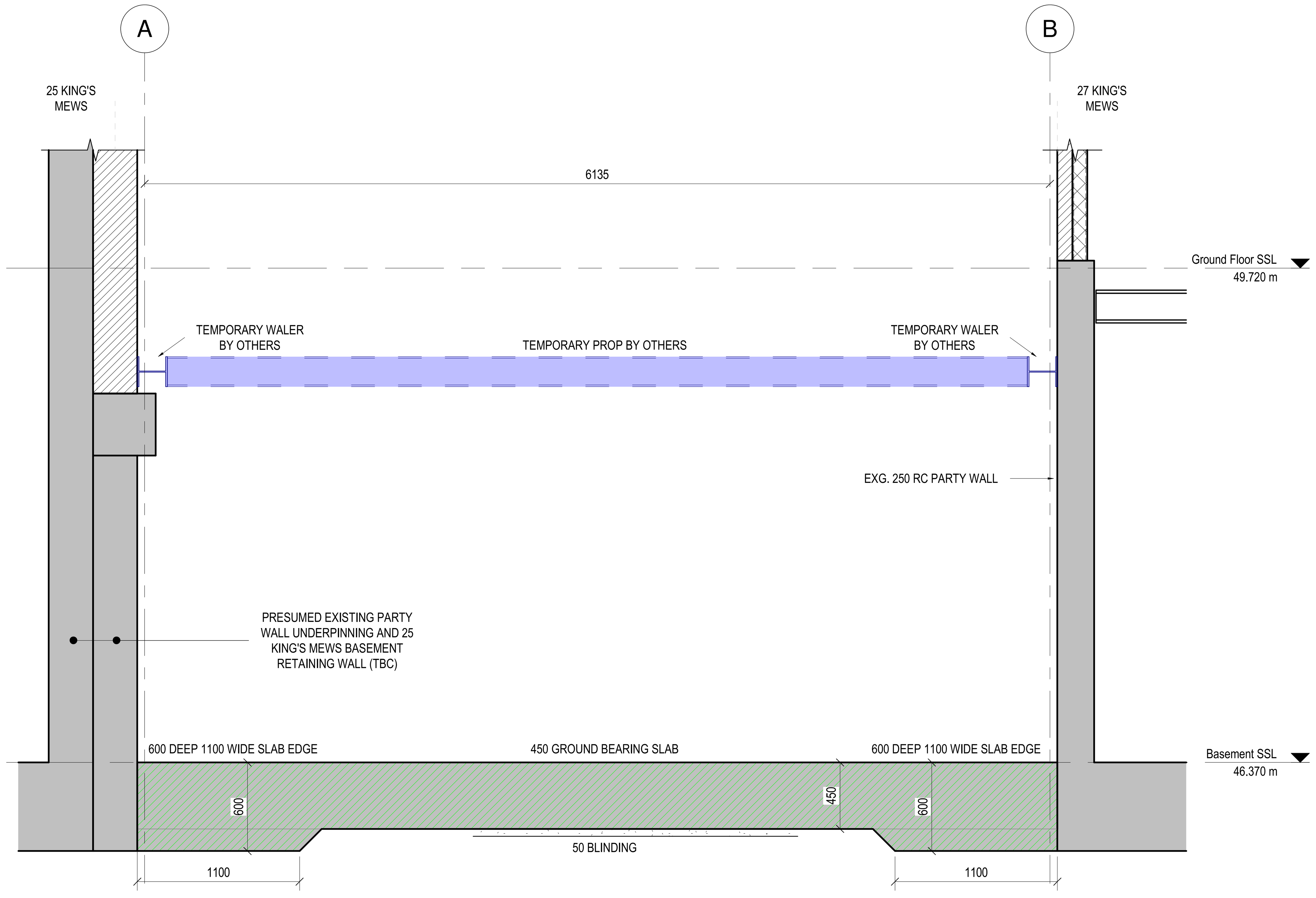
Drawn	A1 Scales		
NG	As indicated		
Date	Eng.	Chk.	App'd
Oct 2023	WW	SA	AO

ACTUAL DIMENSION = 80mm

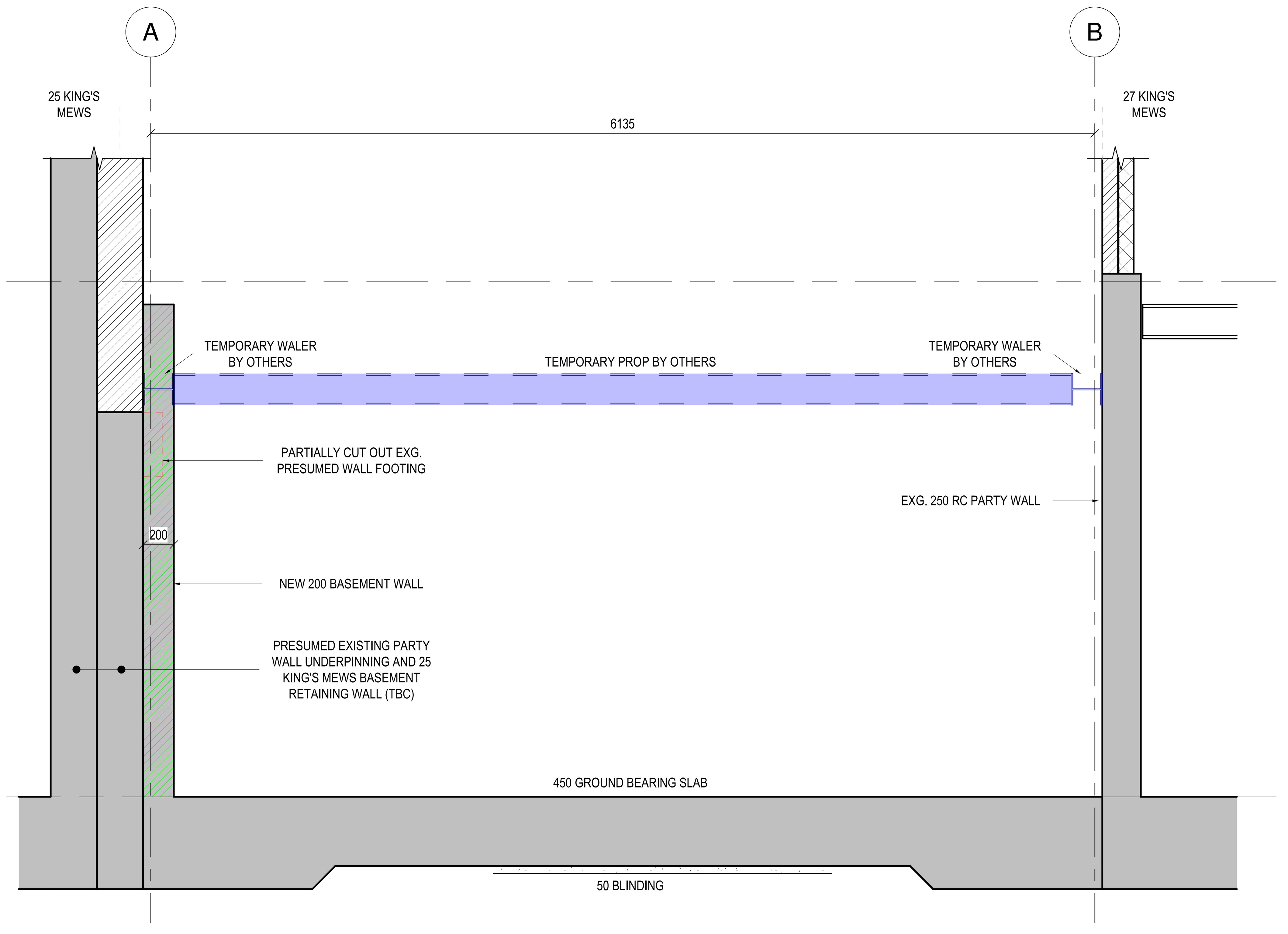
Project No.	Drawing No.	Rev.
99737	17002	P2



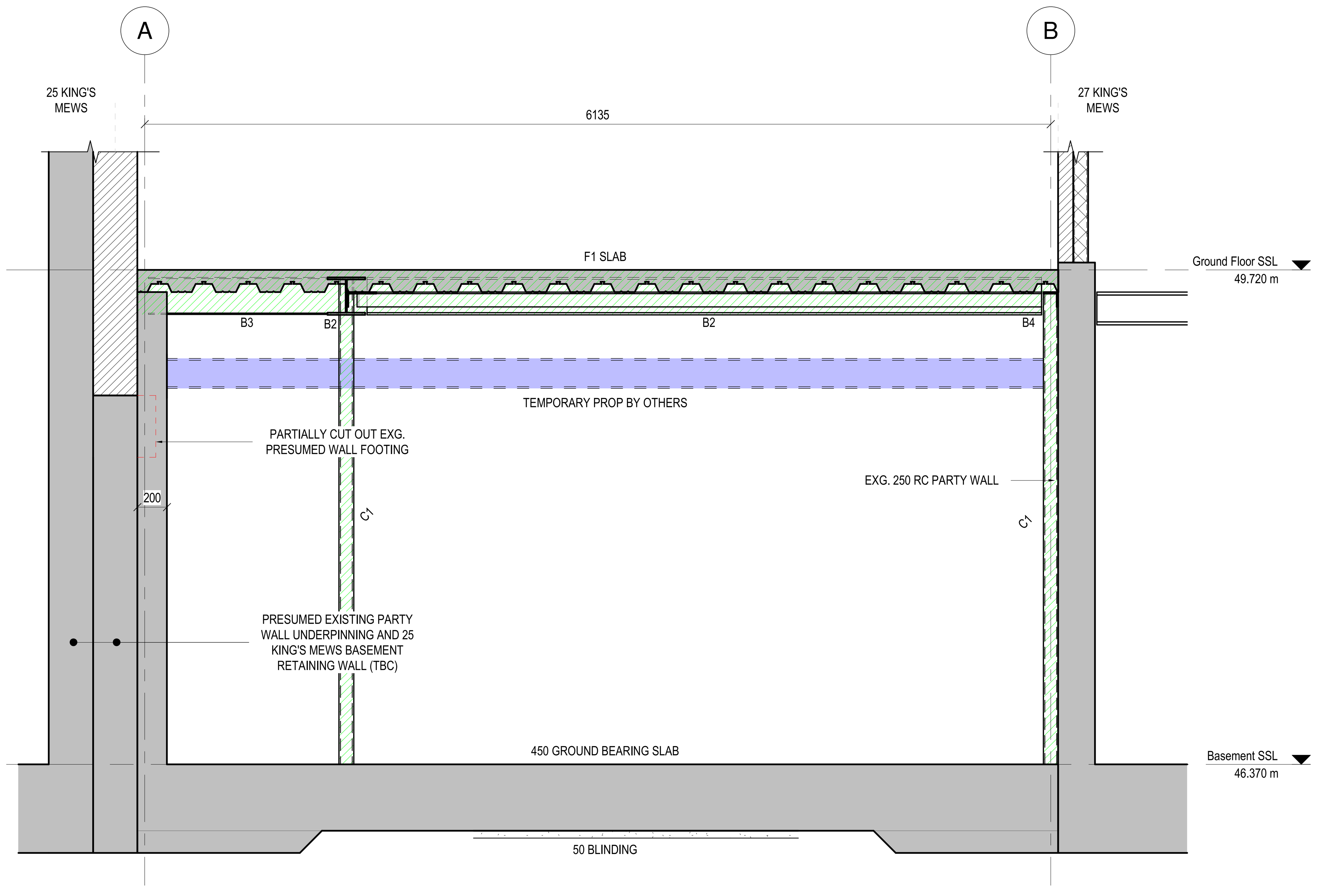
Phase 3 - Bulk Excavation and Forming Basement Raft
SCALE: 1 : 50



Phase 3.1 Section A-A
Form Basement Raft
SCALE: 1 : 25



Phase 3.2 Section A-A
Form Basement Wall
SCALE: 1 : 25



Phase 3.3 Section A-A
Install Steelwork and Ground Floor Slab
SCALE: 1 : 25

General Notes

1. All dimensions are in millimetres and levels in metres.

2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.

3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.

4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.

5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

LEGEND

REF.	DESCRIPTION
	DEMOLISHED
	TEMPORARY WORKS (BY OTHERS)
	NEXT WORK SEQUENCE

Rev.	Date	By	Chk.	Details Of Revision
P2	26.10.23	NG	WW	Coordinated Updates
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Work Sequence Phase 3

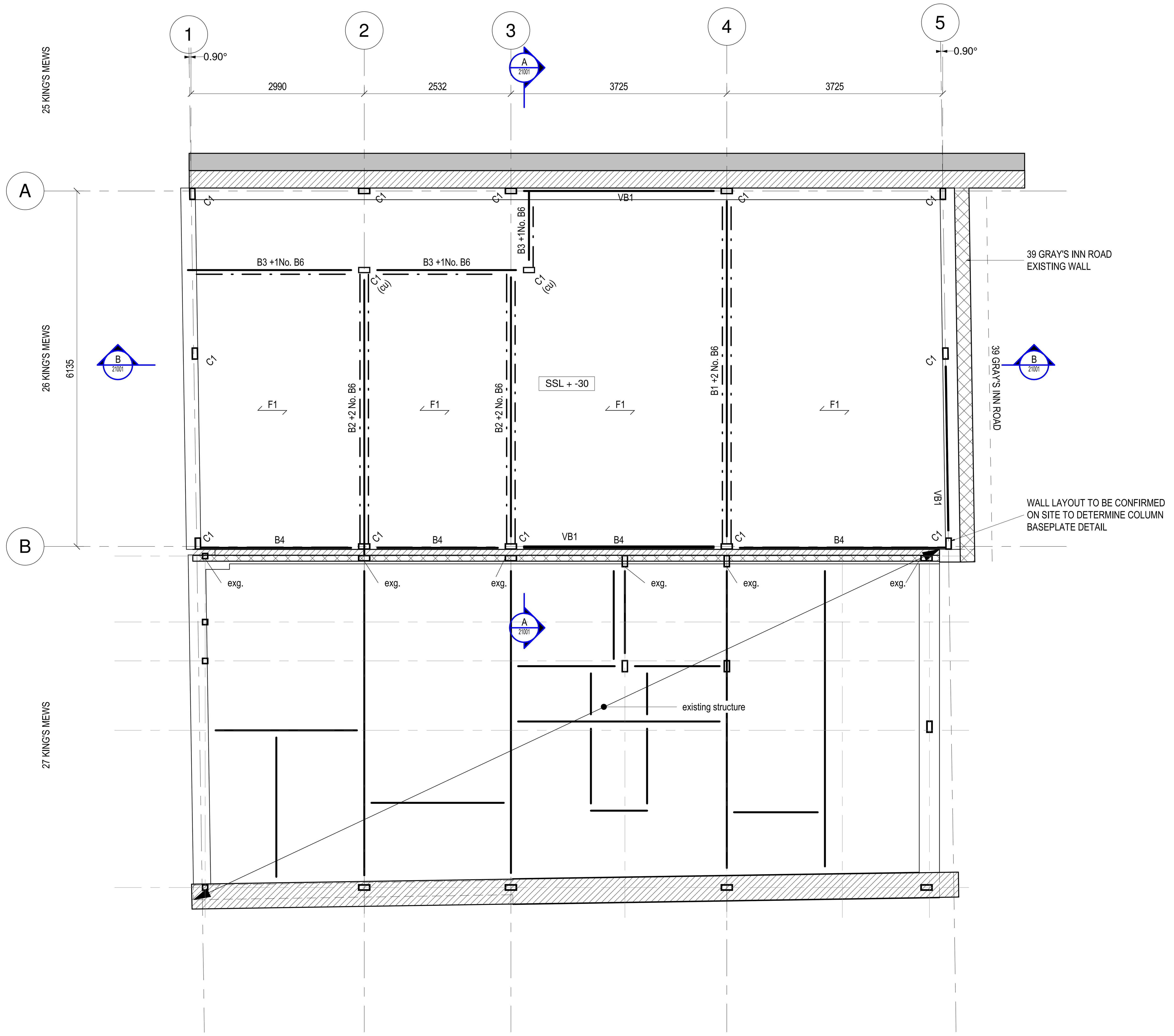
FORWARD THINKING ENGINEERING
6 Flitcroft Street, London, WC2H 8DJ Tel: 020 7998 5868
Email: admin@dcl.engineering Web: www.dcl.engineering

Status: **INFORMATION**

Drawn	NG	A1 Scales	As indicated		
Date	Oct 2023	Eng. WW	Chk. SA	App'd. AO	

ACTUAL DIMENSION = 80mm

Project No.	Drawing No.	Rev.
99737	17003	P2



Ground Floor
SCALE: 1 : 50

General Notes

1. All dimensions are in millimetres and levels in metres.

2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.

3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.

4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.

5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

1. All structural timber to be grade C24 and to be tanalised.

2. All structural steelwork to be grade S355 J0 to BS EN 10025 U.N.O.

3. All bolts to be 8.8 and to be zinc plated.

4. Concrete grade to be C30/37

5. Waterproofing by others

COLUMN SCHEDULE

REF.	DESCRIPTION
C1	RHS200x100x10
C2	SHS100x100x10

BEAM SCHEDULE

REF.	DESCRIPTION
B1	UC254x254x107
B2	UC254x254x73
B3	UB254x146x31
B4	L100x100x10
B5	UB203x102x23
B6	L200x100x10
VB1	Cross Bracing 2No. 100 x 8 Plate

LEGEND

REF.	DESCRIPTION
F1	COMFLOR 60 1.2 THICK DECKING WITH 150mm DEEP SLAB A193 MESH TOP LAYER AND H10 BARS IN TROUGH OF DECKING

Rev.	Date	By	Chk.	Details Of Revision
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Proposed Ground Floor General Arrangement

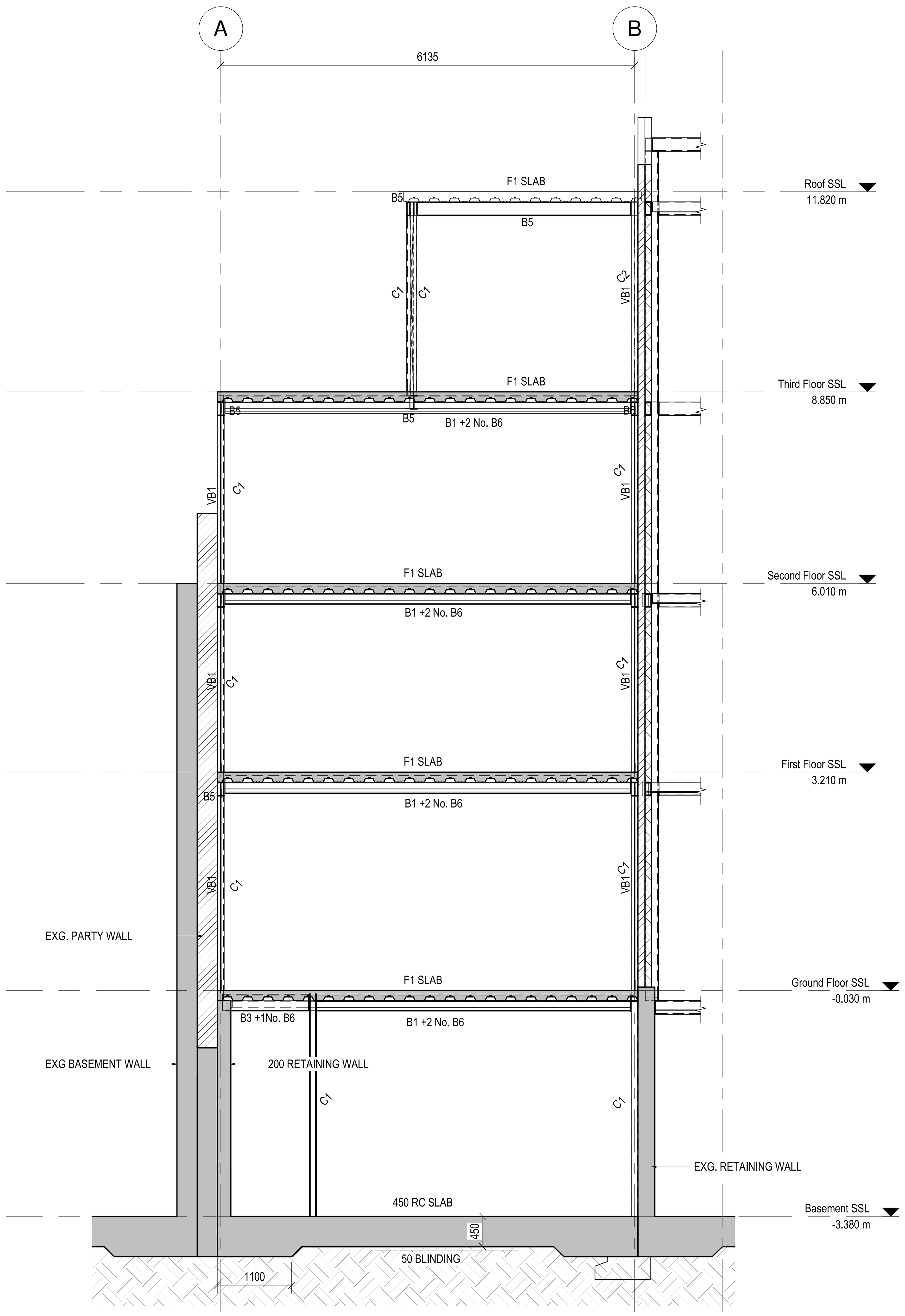
DCL
FORWARD THINKING ENGINEERING
6 Flitcroft Street, London, WC2H 8DJ Tel: 020 7998 5868
Email: admin@dcl.engineering Web: www.dcl.engineering

Status: INFORMATION

Drawn	NG	A1 Scales	As indicated		
Date	Oct 2023	Eng. WW	Chk. SA	App'd	AO

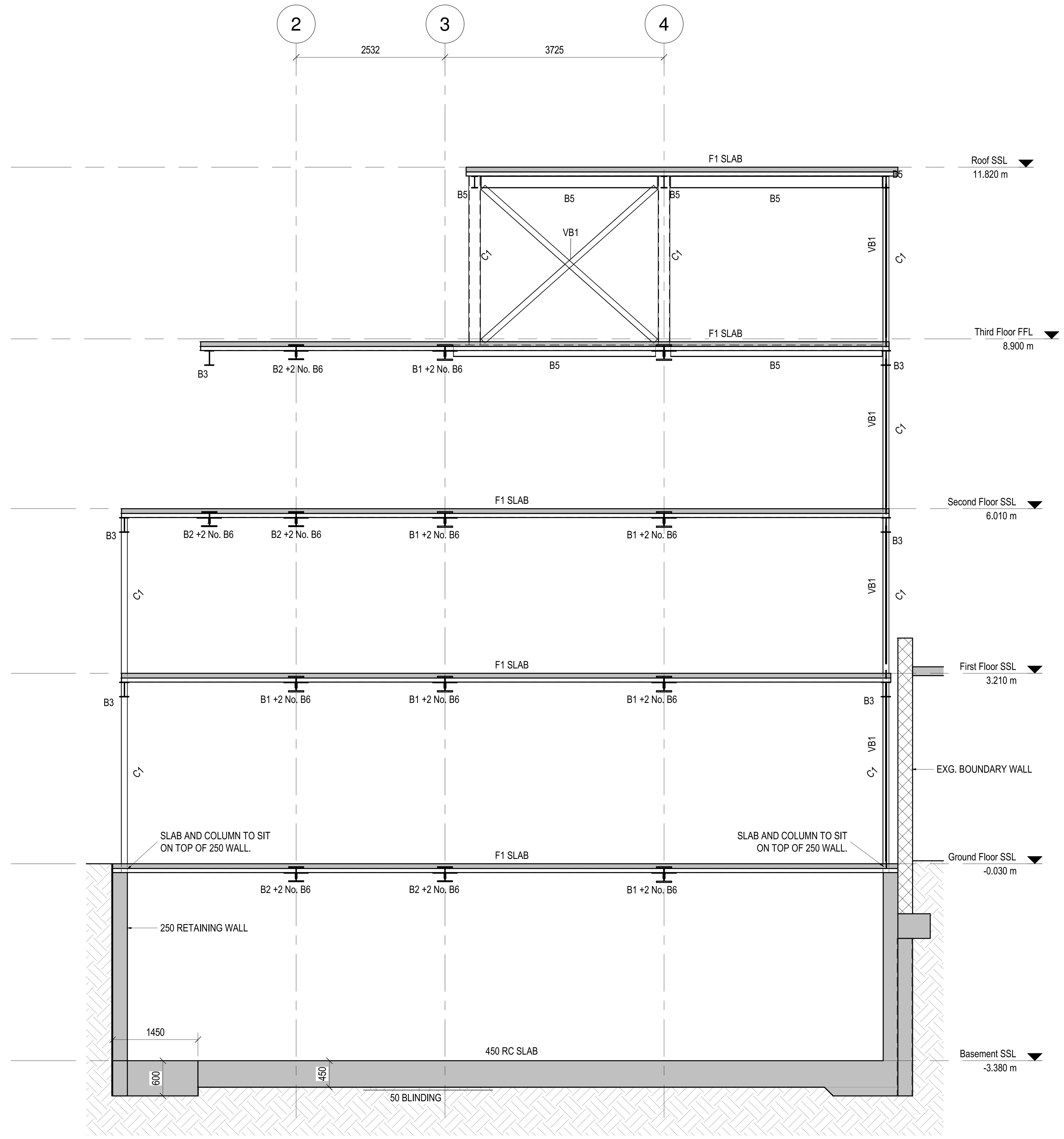
ACTUAL DIMENSION = 80mm

Project No.	Drawing No.	Rev.
99737	20001	P1



PROP. Section A-A

SCALE: 1 : 50



PROP. Section B-B

SCALE: 1 : 50

General Notes

1. All dimensions are in millimetres and levels in metres.
2. This drawing is to be read in conjunction with relevant Architect's and Engineer's drawings and specifications.
3. Do not scale from this drawing in either paper or digital form. Use written dimensions only.
4. The contractor is to verify all dimensions on site before commencing work. All error and omissions are to be reported to the Engineer.
5. All work is to be carried out in accordance with the latest Building Regulations and Codes of Practice.

Drawing Notes

1. All structural timber to be grade C24 and to be tanalised.
2. All structural steelwork to be grade S355 J0 to BS EN 10025 U.N.O.
3. All bolts to be 8.8 and to be zinc plated.
4. Concrete grade to be C30/37
5. Waterproofing by others

COLUMN SCHEDULE

REF.	DESCRIPTION
C1	RHS200x100x10
C2	SHS100x100x10

BEAM SCHEDULE

REF.	DESCRIPTION
B1	UC254x254x107
B2	UC254x254x73
B3	UB254x146x31
B4	L100x100x10
B5	UB203x102x23
B6	L200x100x10
VB1	Cross Bracing 2No. 100 x 8 Plate

LEGEND

REF.	DESCRIPTION
F1	COMFLOR 60 1.2 THICK DECKING WITH 150mm DEEP SLAB A193 MESH TOP LAYER AND H10 BARS IN TROUGH OF DECKING

Rev.	Date	By	Chk.	Details Of Revision
P1	13.10.23	NG	WW	Issued for Comments

Client: Hallam Management Limited

Project: 26 King Mews

Title: Proposed Sections

DCL
FORWARD THINKING ENGINEERING
6 Flitcroft Street, London, WC2H 8DJ Tel: 020 7998 5868
Email: admin@dcl.engineering Web: www.dcl.engineering

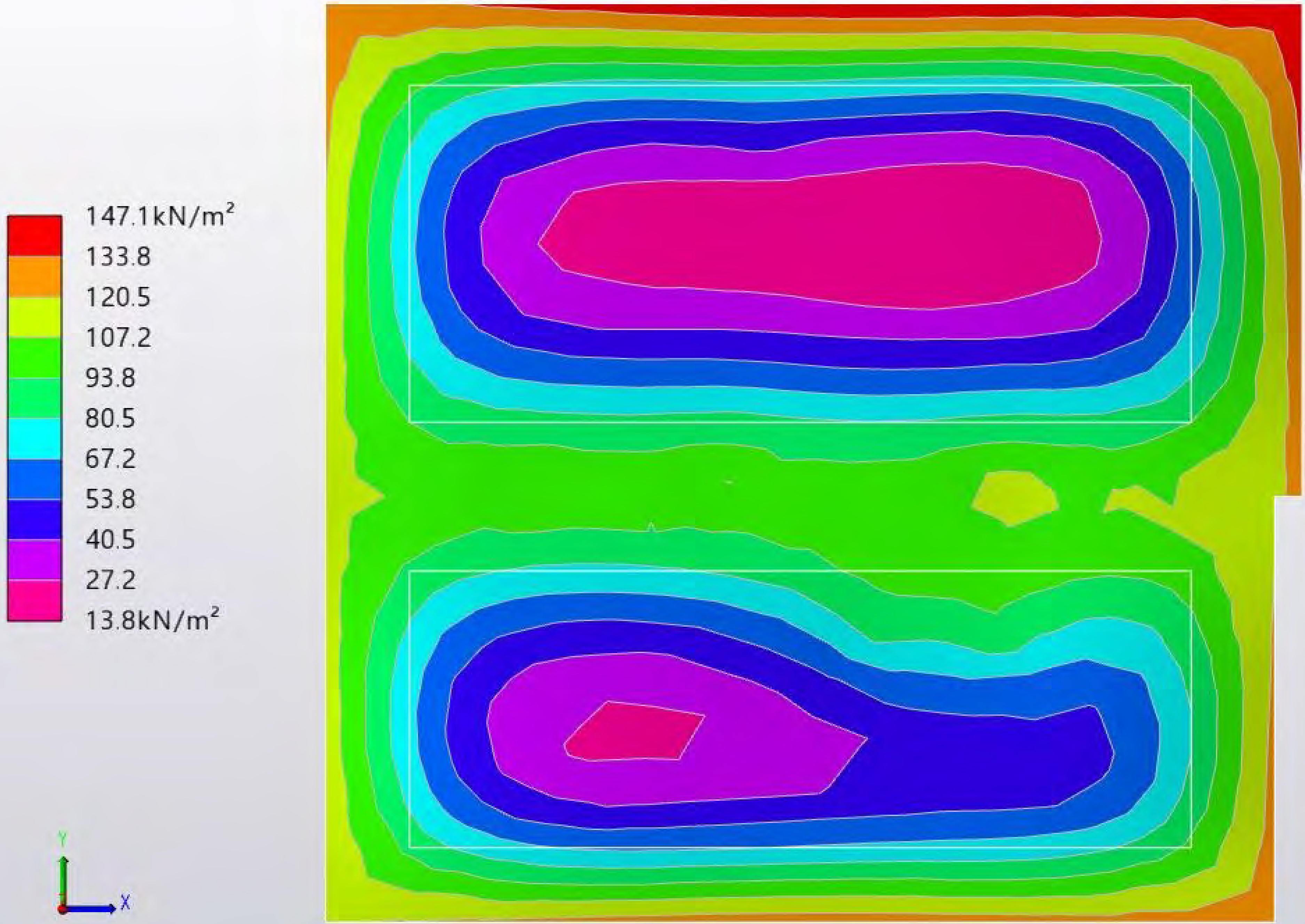
Status: INFORMATION

Drawn: NG | A1 Scales: As indicated

Date: Oct 2023 | Eng.: WW | Chk.: SA | App'd: AO

ACTUAL DIMENSION = 80mm

Project No.: 99737 | Drawing No.: 21001 | Rev.: P1


147.1kN/m²
133.8
120.5
107.2
93.8
80.5
67.2
53.8
40.5
27.2
13.8kN/m²
Y
X

Appendix 3 Site Investigation Report

GROUND ENGINEERING LIMITED
Newark Road
Peterborough PE1 5UA
Tel: 01733 566566

GROUND INVESTIGATION REPORT

26 & 27 KING'S MEWS

LONDON WC1

Report Reference No. C13870A

On behalf of:-

1156 Limited
27 King's Mews
London
WC1N 2JB

September 2023

1156 LIMITED

DCL CONSULTING ENGINEERS LIMITED

REPORT ON A GROUND INVESTIGATION

AT

26 & 27 KING'S MEWS

LONDON WC1

Report Reference No. C13870A **September 2023**

INTRODUCTION

1156 Limited, the client, intends to remodel the existing adjacent commercial buildings, Nos.26 & 27 King's Mews, London WC1. The proposed redevelopment will include the construction of a 3.50m deep basement beneath the footprint of the existing No. 26.

Ground Engineering Limited was instructed by the client to review ground investigations undertaken within and adjacent the site, by Ground Engineering Limited and within other reports in the public realm, and produce a ground investigation report under the direction of DCL Consulting Engineers Limited. This report provides comment on the nature and geotechnical properties of the underlying soils in relation to foundation/basement design and construction, and technical information to support the planning application/basement impact assessment (BIA) for the proposed basement beneath No.26, as required by the London Borough of Camden Planning Guidance 'Basements' document (2021).

This review was also informed by a desk study provided by the client (Ref. MES/2309/DCL004, September 2023).

LOCATION, TOPOGRAPHY, GEOLOGY AND HYDROGEOLOGY OF THE SITE

Location/Description

Nos.26 & 27 King's Mews are situated on the eastern side of the road, some 35m north of its junction with Theobald's Road, and 25m west of Gray's Inn Road, within the Bloomsbury district of the London Borough of Camden, London WC1. The site is centred at approximate National Grid Reference TQ 30940 81998.

The approximately 14m long and 12m wide rectangular site extends eastwards from its frontage on King's Mews roadway. At the time of the investigation the adjoining pair of two-storey (No.26) and four-storey with basement (No.27), brick buildings occupied the whole of the plot.

The plot was bounded to the north and south by Nos.25 & 28 King's Mews, respectively, and to the east by Nos.1 to 16 The Lincolns.

The site and immediate surrounding area was devoid of vegetation.

Topography

The site stands at an approximate elevation of 20.5mOD on locally gently northward and eastward falling ground, some 1.25km north of the eastward flowing River Thames.

Geology

The 1936 geological map for the area at 1:10,560 scale is based on the 1920 Ordnance Survey London Sheet V SW and shows the site to be covered by Taplow Gravel and underlain by the solid geology of the London Clay. This map also shows the culverted course of the River Fleet, flowing southwards, some 625m east of the site.

The 2006 geological map for the area at 1:50,000 scale, Sheet 256, also shows the site to be covered by the renamed superficial Lynch Hill Gravel Member and underlain by the solid geology of the London Clay Formation.

Well records on the 1936 geological map indicate that the surface cover of made ground and superficial deposits are together about 5m thick beneath this part of London.

Previous ground investigations adjacent the site in Nos.25 and 28 King's Mews, confirmed the presence of 3.60m to 4.00m of made ground, underlain by sand and gravel, and then the London Clay at 5.10m to 6.00m below ground level. The latter was found to at least 25m depth, and groundwater was recorded at about 4.00m below ground level.

<u>Hydrogeology</u>

The site is designated by the Environment Agency (EA) as being underlain by a Secondary (A) Aquifer, the Lynch Hill Gravel, which overlies the Unproductive stratum of the London Clay. Based on the local topography and geology of the site area, the direction of near surface groundwater and surface water flow would be expected to be from west to east, towards the culverted River Fleet.

Well records on the 1936 geological map indicate that the practically impervious Unproductive stratum of the London Clay Formation is 12m to 15m thick beneath this part of London and that the underlying Principal Aquifer of the White Chalk Subgroup lies about 40m below ground level, about -18mOD.

SITE WORK

A single borehole was undertaken at the position depicted on the site plan at the rear of this report. Services information was obtained and referenced in relation to the exploratory hole positions prior to boring/excavation.

The investigation was undertaken following the protocols detailed in British Standards (BS) 'Code of Practice for Site Investigations' (BS5930:2015) and 'Methods of test for soils for engineering purposes' (BS1377:1990).

Borehole

A single borehole (BH A) was undertaken by a restricted access, low headroom cable percussive rig within No. 27 King's Mews on 9th June 2016. The final borehole position was chosen following a scan using a cable avoidance tool (CAT). The concrete floor slab was cored using electrically powered diamond drilling equipment at 250mm diameter, and a starter pit was hand dug to 1.20m depth in order to confirm the absence of buried services.

The borehole was then advanced using weighted claycutter and shell tools, initially working within 150mm diameter casing. Water was added to enable drilling of coarse grained soils. Borehole BH A was completed at the intended depth of 15.00m below ground level.

Standard penetration tests were undertaken in the borehole within made ground and coarse grained soils in order to give an indication of the in-situ relative density/shear strength of the material. The test was made by driving a 50mm diameter solid cone (C) into the soil at the base of the borehole by means of an automatic trip hammer weighing 63.50kg falling freely through 750mm. The penetration resistance was usually determined as the number of blows required to drive the tool the final 300mm of a total penetration of 450mm into the soil ahead of the borehole. Where the full penetration was not achieved the actual penetration and the number of blows were recorded. The results have been tabulated and added to the borehole record.

Undisturbed samples (U) nominally 100mm in diameter were taken in clay. The ends of the samples were capped and sealed to maintain them in as representative condition as possible during transit to the laboratory.

Representative disturbed samples of soil were taken from the boring tools at regular intervals throughout the depth of the borehole and placed in polycarbonate pots/small plastic bags (D samples) and large plastic bags (B samples).

On completion of borehole BH A, a 50mm diameter standpipe was installed to 7.00m below ground level, with a gravel response zone up to 1.00m below ground level. Above the response zone to this installation, the borehole was backfilled with bentonite, whilst the hole beneath the installation was infilled with clean arisings. A protective stopcock cover was concreted into the ground flush with the surface over the top of the installation.

The borehole record gives the descriptions and depths of the various strata encountered, results of the in-situ tests, details of all samples taken, installation details and the groundwater conditions observed during boring, on completion and subsequently in the standpipe. Excess spoil was removed from site and disposed of at a licenced facility.

Gas and Groundwater Monitoring

A single return visit was made on 30th June 2016 in order to monitor methane, carbon dioxide and oxygen gas levels in the borehole standpipe. Ambient pressures and flow rates were recorded together with the depth to groundwater. The water level has been added to the borehole record, whilst the gas/groundwater results are presented following the exploratory hole record.

A sample of groundwater was recovered from the borehole standpipe during the monitoring visit, placed in a plastic bottle, and transported directly to the analysing laboratory.

LABORATORY TESTING

The samples were inspected in the laboratory and assessments of the soil characteristics have been taken into account during preparation of the exploratory hole records. The soil sample descriptions are in accordance with BS5930:2015.

The geotechnical tests were conducted to BS1377:1990 and other industry standards, and the results are presented following the exploratory hole records.

Geotechnical Testing

The particle size distribution of selected samples were obtained by sieve analysis. The results of these tests are given as particle size distribution curves at the end of this report.

Selected test specimens were prepared at full diameter from the undisturbed samples recovered from the borehole. An immediate undrained triaxial compression test was made on each sample at a single cell pressure approximately equivalent to the overburden pressure for that sample's depth. The results have been plotted against depth in Figure 1. The moisture content and bulk densities of these specimens were also determined.

Selected samples of soil were analysed to determine the concentration of soluble sulphates. The pH values were also determined using an electrometric method.

GROUND CONDITIONS

The ground conditions encountered in the borehole were as expected from the known history of the site and geological records with a significant thickness of made ground covering the Lynch Hill Gravel at 3.30m below ground level. This superficial deposit was underlain by the solid geology of the London Clay at 5.90m depth. The latter was found to at least 15.00m depth in the completed BH A. A standing water level was subsequently recorded in the borehole standpipe at 3.04m below ground floor level.

Made Ground

The concrete floor slab was 0.30m thick and was underlain by coarse grained made ground. The latter was generally a very loose, dark brown, slightly clayey, ashy sand and gravel with occasional brick cobbles. The gravel fraction consisted of brick, concrete, ash, flint, mortar, slate and fragments of bone, glass and pottery.

The base of the coarse grained fill was proved at 3.30m below ground level.

Lynch Hill Gravel

The superficial Lynch Hill Gravel was met beneath the made ground at 3.30m and was initially a medium dense, orange brown, silty sand and gravel, with a gravel fraction of angular to sub-rounded flint. Below 4.00m depth the Lynch Hill Gravel became very dense and slightly silty, and this stratum was proved to 5.90m depth in BH A, a recorded thickness of 2.60m, which was consistent with nearby well and borehole records.

London Clay

The solid geology of the London Clay was reached at 5.90m depth and was initially reworked to a stiff, brown, slightly gravelly, silty clay with a gravel fraction of sub-angular to rounded flint. This reworked horizon was 0.50m thick and was followed by a stiff, closely fissured, grey brown clay with occasional silt partings and rare gravel size pyrite nodules. The London Clay became silty below 10.00m depth, and then below 13.00m below ground level was a very stiff, grey brown, slightly sandy, silty clay with occasional silt and fine sand partings, and rare gravel size pyrite nodules. The London Clay was found to at least 15.00m below ground level where the borehole was completed.

Groundwater

The addition of water to enable boring of the Lynch Hill Gravel from 3.60m to 4.00m depth in BH A will have masked any initial water ingress within this stratum, but water was recorded by the driller as being met at 4.00m and rose to 3.60m in the fifteen minutes before drilling resumed. This water was sealed out of the borehole once the casing entered the underlying London Clay, and the 15.00m deep borehole was 'damp' on completion.

The water level recorded in the 7.00m deep standpipe three weeks after installation was 3.04m below ground level.

COMMENTS ON THE GROUND CONDITIONS IN RELATION TO FOUNDATION DESIGN AND CONSTRUCTION

The investigation found a significant thickness of made ground beneath the existing building (No.26), which is bounded to the north (No.25) and south (No.27) by similar existing basements. Foundations for the new 3.50m deep basement will need to penetrate this made ground to reach the top of the underlying medium dense becoming very dense Lynch Hill Gravel, which was met at 3.30m, a minimum of 2.50m above the interface with the underlying stiff solid geology London Clay. A standpipe water level was recorded at 3.04m below the ground level, above the proposed basement floor level. This water level is considered to reflect the depth of 'perched' groundwater within the superficial Lynch Hill Gravel.

Foundations for the envisaged basement will need to penetrate the made ground and be based within the medium dense becoming very dense Lynch Hill Gravel. Existing shallow foundations will need to be underpinned to the same level as the adjacent basements, which should be feasible using traditional underpinning techniques.

Foundation Depths

The exploratory hole encountered natural ground at 3.30m depth within this site although it may locally be expected to lie at slightly greater depths, as previously found beneath the adjacent sites where up to 4.00m of made ground was encountered.

The underlying Lynch Hill Gravel may be regarded as a non-shrinkable stratum. The top of the high volume change potential London Clay was recorded at 5.90m below street level and so will be well below the depth affected by tree root-induced desiccation.

Foundations will need to be taken down through the made ground and into the top of the medium dense, becoming very dense Lynch Hill Gravel, which was met at 3.30m below ground level within this small site. The 3.50m deep proposed basement floor level should therefore be within the top of this stratum.

Bearing Pressure/Capacity

The construction of a 3.50m deep basement on this site should remove most, if not all, of the made ground as its foundations will reach the underlying sand and gravel at 3.30m depth. With a minimum of 1.90m of sand and gravel remaining between the base of the made ground and the top of the London Clay, the superior bearing properties of the Lynch Hill Gravel can be utilised during the design of strip or pad foundations for the proposed basement walls.

The results of the in-situ standard penetration tests indicate that an allowable bearing pressure of $300kN/m^2$ could be applied on 1.00m wide foundations cast at a basement excavation level of 3.50m on the Lynch Hill Gravel. Such a pressure would not overstress the underlying stiff London Clay.

A bearing pressure of $300kN/m^2$ should be more than sufficient to support the likely foundation pressures for the new structure and for adjacent foundations underpinned to the same depth as the proposed and adjacent existing basements.

Basement

The construction of a 3.50m to 4.00m deep basement will remove the made ground. Foundations for the basement walls just below the new basement floor level would be within the very dense Lynch Hill Gravel and could be designed using the previously detailed bearing parameters.

Alternatively a basement raft foundation could be considered for this structure, although it's design would need to take into account the bearing properties of the underlying London Clay. A conservative net safe bearing capacity of $150kN/m^2$, which incorporates a factor of safety of 3.0, could be used for the design of a 6.00m wide raft foundation at 3.50m below existing ground level.

It is estimated that theoretical base heave at the centre of a 12m long and 6m wide, 3.50m deep unconfined basement excavation would be in the order of 15mm, based on the proposed basement dimensions and typical parameters for the underlying London Clay. However, with a minimum of 1.90m of Lynch Hill Gravel remaining below the proposed underside of the 3.50m to 4.00m deep basement floor slab, little, if any, base heave would be expected following the removal of about 65kN/m^2 of overburden pressure within the basement, as any heave would dissipate between inter-grain contacts within the Lynch Hill Gravel.

A likely basement raft loading is unknown but if it were the 65kN/m^2 of removed overburden pressure then no net heave/settlement would be expected. Raft loadings greater than 65kN/m^2 could result in net settlement, whilst conversely loads lower than 65kN/m^2 could result in net heave, although as detailed above this is considered unlikely. Net differential heave/settlement will need to be taken into account in the design of the basement floor. The advice of specialists should be sought in this regard.

<u>Excavations/Groundwater</u>

The excavation of the basement to between 3.50m and 4.00m below existing ground floor level will require the construction of close support to its sides, the control of groundwater, and the need to avoid undermining adjacent structures.

The use of mass concrete basement walls, constructed in alternate panels around the perimeter of the basement could provide support, a limited cut-off to 'perched' water and reduce the scale of any dewatering required within the basement excavation.

An alternative would be to use sheet, contiguous or secant piled walls around the perimeter of the basement, although this may well be problematical on this relatively small restricted access site, which already has adjacent basement walls on its northern and southern sides. Piling to a sufficient depth to mobilise adequate passive pressure below the basement level should be feasible on this site.

The excavation of a 3.50m deep basement could then be undertaken within the mass concrete or piled walls, although it should be noted that mass concrete, contiguous and sheet pile lined excavations may not be water tight.

In order to construct the basement beneath this site it will be necessary to provide permanent support to the adjacent structures, some of which are based on deepened strip and underpinned foundations, whilst others are supported by basement walls. This support can either be provided by underpinning these structures to the same depth as the proposed basement prior to basement construction or by constructing piled walls to the excavation that are adequately propped during construction by temporary support and permanently by the basement and ground floors, to prevent movement at the top of the retaining walls.

Such lateral movement would otherwise be accompanied by settlement of the ground behind the basement walls. CIRIA report C760 'Guidance on Embedded Retaining Wall Design' (2017) indicates very small scale horizontal and vertical movements resulting from the construction of a secant piled wall, as does the use of high support stiffness (high propped walls and top down construction) to the basement excavation. Provided that such a very stiff bracing system is used to prevent deflection of the proposed basement walls, and that the neighbouring structures are of robust construction, the anticipated level of structural damage, if any, would fall within Category 1 'very slight' as described in Table 6.4 of the aforementioned CIRIA document.

The advice of specialist groundworks contractors with experience of constructing such basements should be sought, particularly in respect of other potential methods of providing support to the sides of the basement excavation on this small site.

The basement excavation should be inspected on completion to ensure that the condition of the soil complies with that assumed in design. Should pockets of inferior material be present, they should be removed and replaced with well graded hardcore or lean mix concrete. The excavated surface should be protected from deterioration and a blinding layer of concrete used where foundations are not completed without delay.

The recorded standpipe groundwater level of 3.04m would be just above the proposed floor level of 3.50m, and so potential flotation should not be a problem.

As the water level was recorded above the level of the proposed basement it will be necessary to waterproof the basement in order to prevent the ingress of 'perched' water and downward percolating surface water into the completed structure.

Safety precautions should not be neglected especially where personnel are to enter excavations, when close side support will be required in order to maintain excavation stability. All excavations should be undertaken in accordance with CIRIA Report 97 '*Trenching Practice*'. This is especially important on this site as excavations are unlikely to stand unsupported even in the short term.

Care should also be taken to ensure that the proposed retaining walls of the basement are not surcharged with plant and equipment or the stockpiling of materials and excavated soils outside of the basement excavation.

<u>Piled Foundations</u>

In the unlikely event that piled foundations are preferred due to practical or economic considerations related to the construction of the basement and underpinning foundations on this site, the ground conditions are considered suitable for bored or CFA, but not driven piles as the vibrations during installation of driven piles could damage the existing dwelling and adjacent structures. The advice of specialist piling contractors should be sought as to their preferred method of pile installation in these conditions on this restricted access site and their attention drawn to the very dense nature of the Lynch Hill Gravel, and the possible presence of concretionary limestone nodules within the London Clay beneath the site.

Preliminary working loads for a single bored pile may be estimated for design and cost purposes using pile bearing coefficients, which are based on the following assumptions.

1) The ultimate load on a pile would be the sum of the side friction/adhesion acting on the pile shaft together with the end bearing load.

2) The pile bearing properties within the depth of the proposed basement have been ignored.

3) The shaft friction of a pile within sand and gravel would be a function of the SPT 'N' values and the overburden pressure. The groundwater level was recorded at about 3.00m depth. End bearing within the very dense Lynch Hill Gravel should not be considered.

4) In the London Clay the shaft adhesion and end bearing would be a function of the lower bound average of the apparent cohesion values determined by triaxial compression strength tests (Figure 1).

5) A factor of safety of at least 2.0 would be used to assess pile working loads. If test loading of selected piles were not practical the factor of safety would be increased to at least 2.5.

Item	**Ultimate Pile Bearing Value kN/m^2**
Shaft adhesion/friction in ground to about 4m	Ignored
Average shaft adhesion in Lynch Hill Gravel	20
Average shaft adhesion in London Clay to 10m	50
Average shaft adhesion in London Clay, 10m to 15m	60
End bearing in London Clay above 10m	900
End bearing in London Clay at 10m	1125

Using these coefficients it is estimated that a single, 300mm diameter bored pile installed to 10m below ground level would have an anticipated working load of 125kN, with a factor of safety of 2.5. Different pile lengths, or diameters, from those detailed above would give different available working loads, which could be tailored to suit the working loads required.

The design of piled foundations on this site will also need to take into account potential tensile stresses in the piles during basement construction where the net change in load is to be reduced.

A piling specialist should undertake the final design of piles.

Retaining Walls

The walls of the proposed basement will act as retaining walls and will need to be designed accordingly. For a permanent retaining wall analysis effective stress parameters would be appropriate, however, in the absence of effective stress testing on samples from this site, published parameters, previous experience and in-situ test results could be used as a conservative approach.

The design of retaining walls around the basement area may be based on the following stress parameters:

Soil Type	Bulk Density (Mg/m^3) γ_B	Effective Shear Strength (kPa) c'	Angle of Shearing Resistance (degrees) ϕ'
Made Ground	1.80	0	28
Lynch Hill Gravel	2.10	0	36
London Clay	2.00	0-2	22

Buried Concrete

Sulphate analysis of the soil samples tested gave results in Design Sulphate Classes DS-1, DS-2 and DS-3 of the BRE Special Digest 1, Table C2 (2005) presented in Appendix 1. The pH results were between 6.7 and 7.4 and so slightly acidic to alkaline. The highest DS-3 results were obtained within the made ground.

The London Clay is listed in this publication as being a stratum that may contain sulphides, such as pyrite, hence oxidation due to disturbance during the excavation of foundations may increase the total potential sulphate content. Visual evidence of pyrite was recorded within the London Clay beneath this site. It should be noted that the use of piled foundations would minimise disturbance of the ground and consequently reduce the potential for the oxidation of any pyritic clay. Pile arisings should not be re-used and placed against foundations.

Using the sulphate and pH results an Aggressive Chemical Environment for Concrete (ACEC) Class of AC-3 would be considered appropriate for buried concrete beneath this site as detailed in the previously cited BRE document.

Slope Stability

The ground within which the level plot is located slopes down gently to the north/north-east and falls from 22.7mOD near the southern end of the parallel John Street to 19.9mOD near the junction of Gray’s Inn Road and Northington Street, 100m distant. This is a slope angle of less than 1 degree and hence this slope is not marked on Figure 16 of the London Borough of Camden 'Guidance for subterranean development' (2010), which indicates slopes of greater than 7 degrees.

There is no evidence of historical slope instability, nor would it be expected based on the topography of the immediate surrounding area.

On this site it is considered unlikely that the proposed basement development will induce slope instability.

Other Issues

The basement development beneath this site would only be considered likely to affect the drainage system of the site itself. However, drainage and sewerage records for the surrounding buildings will need to be referenced, if available, or perhaps surveyed to confirm that the site does not share a communal drainage system that runs beneath the site.

The flow of surface water within the surrounding area, from south to north/north-east, should not be changed by the proposed basement on this small site.

As previously described, 'perched' groundwater was recorded within the basal part of the made ground beneath this site at 3.04m below ground level. The proposed 3.50m to 4.00m basement excavation depth therefore does extend below the 'perched' groundwater level. However, little displacement of groundwater will take place by its exclusion from beneath the area of the proposed basement and footings, so little or no rise would be expected in the level at which groundwater currently stands adjacent to the site.

The orientation of the small proposed basement, when considered together with the adjacent existing basements to the immediate north and south of the site, would be across the likely direction of near surface groundwater flow from south to north-north-east on this very gently sloping ground. However, as the proposed 3.50m deep basement does not extend greatly below the recorded 'perched' groundwater level, it is considered that the drainage path will not be substantially increased.

SOIL GAS MONITORING RESULTS

A single return visit to site recorded concentrations of landfill type gasses (methane, carbon dioxide and oxygen), in the BH A standpipe installation. The results are presented to the rear of the exploratory hole record. The recorded concentration of methane was less than 0.1%. The carbon dioxide level was 3.1%. The recorded oxygen concentration within the standpipe was slightly depleted when compared to atmospheric conditions. The in-situ measurement confirmed a gas emission flow rate of <0.1l/hr.

Assuming a 'worst case' positive flow rate of 0.1l/hr, the carbon dioxide result gives a Gas Screening Value (GSV) of 0.0031l/hr. This GSV falls within Characteristic Situation 1 as defined by BS8485:2015+A1:2019 'Code of practice for the design of protective measures for methane and carbon dioxide ground gases for new buildings', and so no special precautions are required to protect the proposed redevelopment from ingress of soil gases.

GROUND ENGINEERING LIMITED

S. J. FLEMING	**J. E. M. DAVIES**
M.Sc., M.C.S.M.,	**B.Sc.(Hons.), M.Sc.,**
C.Geol., F.G.S.,	**C.Geol., F.G.S.,**
Director	**Associate Director**

Site Location Plan

Based on a proposed development plan provided by the Engineer

NOT TO SCALE

Project	**26 & 27 King's Mews, London WC1**	**GROUND ENGINEERING LIMITED**	**Project No.**
Client	**1156 Limited**	Peterborough Tel : 01733 566566	**C13870A**

Borehole Location Plan

Based on a plan provided by the Engineer

Key

Borehole

NOT TO SCALE

Project: 26 & 27 King's Mews, London WC1 **Client:** 1156 Limited	**GROUND ENGINEERING LIMITED** Peterborough Tel : 01733 566566	**Project No.** **C13870A**

GROUND ENGiNEERiNG LIMITED
Tel: 01733-566566
www.groundengineering.co.uk

Site: 26 & 27 KING'S MEWS, LONDON WC1

BOREHOLE BHA

Date: 09/06/16

Hole Size: 150mm dia to 15.00m

Ground Level:

Depth m	Type	Blows	(Date) Casing	Inst.	Description of Strata	Legend	Depth m	O.D. Level m
					MADE GROUND - CONCRETE.		0.30	
0.30-1.00	B1				MADE GROUND - Very loose, dark brown, slightly clayey SAND AND GRAVEL with occasional brick cobbles. Gravel of brick, concrete, ash, flint, quartzite, clinker and porcelain.			
1.20-1.70	B2							
1.35-1.65	C	N1						
2.00-2.50	B3							
2.15-2.45	C	N3	1.50					
3.00-3.30	B4							
3.04	W1							
3.15-3.45	C	N28	3.00				3.30	
3.30-3.60	B5				Medium dense, orange brown, silty SAND AND GRAVEL. Gravel of angular to sub-rounded flint. (LYNCH HILL GRAVEL MEMBER)		4.00	
4.20-4.70	B6				Very dense, orange brown, slightly silty SAND AND GRAVEL. Gravel of angular to rounded flint.			
4.35-4.54	C	50*	4.20					
5.00-5.50	B7				(LYNCH HILL GRAVEL MEMBER)			
5.15-5.41	C	50*	5.00				5.90	
5.90-6.20	B8				Stiff, brown, slightly gravelly, silty CLAY. Gravel of sub-angular to rounded flint. (REWORKED LONDON CLAY)			
6.20-6.60	U1	60	6.10				6.40	
6.60	D1				Stiff, closely fissured, grey brown CLAY with occasional silt partings and rare gravel size pyrite nodules.			
7.00-7.45	U2	55	6.10	BENEATH INSTALLATION				
7.45	D2							
8.00	D3				(LONDON CLAY FORMATION)			
8.50-8.95	U3	50	6.10					
8.95	D4							
9.50	D5							
10.00-10.45	U4	70	6.10				10.00	

REMARKS
1. Floor slab cored using diamond drilling equipment
2. Excavating a pit from 0.00m to 1.20m for 1.50 hours
3. Water added from 3.60m to 4.00m
4. Borehole cased to 6.10m depth
5. Gas monitoring standpipe installed to 7.00m depth

Project No 13870A

Scale 1:50

Page 1/2

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

Groundwater Strikes						Groundwater Observations			
No	Struck (Depth m)	Rose to (Depth m)	Rate	Cased	Sealed	Date	Hole (Depth m)	Casing (Depth m)	Water
1	4.00	3.60	medium	4.00	6.10	09/06/16	15.00	6.10	damp
						30/06/16	7.00	1.00	3.04

GROUND ENGiNEERiNG L I M I T E D
Tel: 01733-566566
www.groundengineering.co.uk

Site: 26 & 27 KING'S MEWS, LONDON WC1

BOREHOLE BHA

Date: 09/06/16

Hole Size: 150mm dia to 15.00m

Ground Level:

Depth m	Type	Blows	(Date) Casing	Inst.	Description of Strata	Legend	Depth m	O.D. Level m
				BENEATH INSTALLATION	Stiff, closely fissured, grey brown, silty CLAY with rare gravel size pyrite nodules.		10.00	
10.45	D6			BENEATH INSTALLATION				
11.00	D7			BENEATH INSTALLATION				
11.50-11.95	U5	70	6.10	BENEATH INSTALLATION				
					(LONDON CLAY FORMATION)			
11.95	D8			BENEATH INSTALLATION				
12.50	D9			BENEATH INSTALLATION				
13.00-13.40	U6	80	6.10	BENEATH INSTALLATION			13.00	
				BENEATH INSTALLATION	Very stiff, grey brown, slightly sandy, silty CLAY with occasional silt and fine sand partings, and rare gravel size pyrite nodules.			
13.40	D10			BENEATH INSTALLATION				
14.00	D11			BENEATH INSTALLATION				
					(LONDON CLAY FORMATION)			
14.50-14.95	U7	75	6.10	BENEATH INSTALLATION				
15.00	D12			BENEATH INSTALLATION			15.00	
					Borehole completed at 15.00m depth			

REMARKS

Project No 13870A

Scale 1:50

Page 2/2

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

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

GROUND ENGINEERING LIMITED

Results of Standard/Cone Penetration Tests

C13870A - 26 & 27 King's Mews, London WC1

BH.No.	Depth (m)	Casing Depth (m)	Depth to Water (m)	Type of Test *	Seating Drive Blows /Penetration (mm)	Test Drive: 300mm. Blows for each successive 75mm penetration				N Value	Extra-polated N Value
BHA	1.20-1.65			C	0 / 150	1	0	0	0	1	
	2.00-2.45	1.50		C	1 / 150	1	0	1	1	3	
	3.00-3.45	3.00		C	12 / 150	10	7	6	5	28	
	4.20-4.54	4.20	3.60	C	16 / 150	15	20	15/35			
	5.00-5.41	5.00	4.30	C	10 / 150	12	12	18	8/30		

*** C denotes test using a solid cone**
S denotes test using a split barrel sampler

Groundwater/Gas Monitoring Record

GROUND ENGINEERING LIMITED

Site: 26 & 27 King's Mews, London WC1

Report Ref: C13870A

Date	Borehole	Methane (% v/v)		Carbon Dioxide (% v/v)		Oxygen (% v/v)		Flow Rate (l/hr)	Atmosph. Pressure (mb)	Depth of Well (m)	Depth to Groundwater (m)
		Peak	Steady	Peak	Steady	Min.	Max.				
30/06/16	BH A	<0.1	<0.1	3.1	3.1	16.9	16.9	<0.1	1002	7.00	3.04#

- Water samples recovered.

LABORATORY TEST RESULTS

CONTRACT 26 & 27 KING'S MEWS, LONDON WC1

Bore-hole	Sample	Depth m	Classification				Density		Triaxial Compression					Sulphates (SO_4)				Remarks
			Liquid Limit %	Plastic Limit %	Plasticity Index %	Moisture Content %	Bulk Mg/m^3	Dry Mg/m^3	Type	Principal Stress Difference kPa	Cell Pressure kPa	Shear Strength kPa	Angle of Shear Resistance degrees	Soil Total % Dry Wt.	Soil Aqueous Extract mg/l	Water mg/l	pH	
BHA	B2	1.20 - 1.70													1539		6.7	
	B4	3.00 - 3.30													2478		7.4	
	B8	5.90 - 6.20													337		7.4	
	U1	6.20 - 6.60				31	1.96	1.49	Q	183	130	91	0					
	U2	7.00 - 7.45				25	2.05	1.64	Q	228	150	114	0					
	U3	8.50 - 8.95				31	1.97	1.51	Q	178	180	89	0		710		7.4	
	U4	10.00 - 10.45				27	2.01	1.58	Q	262	210	131	0					
	U5	11.50 - 11.95				23	2.04	1.66	Q	241	240	121	0					
	U6	13.00 - 13.40				25	2.08	1.66	Q	398	270	199	0		856		7.1	
	U7	14.50 - 14.95				23	2.08	1.70	Q	346	300	173	0					

U - UNDISTURBED SAMPLE
D - DISTURBED SAMPLE
B - BULK SAMPLE
W - WATER SAMPLE

C.U. - CONSOLIDATED UNDRAINED
C.D. - CONSOLIDATED DRAINED
Q. - IMMEDIATE UNDRAINED
Q.M. - IMMEDIATE UNDRAINED MULTISTAGE

Aqueous Extract 2:1 Water:Soil

13870

GROUND ENGiNEERiNG LIMITED
Tel: 01733-566566
www.groundengineering.co.uk

8180

GROUND ENGINEERING LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

TEST CERTIFICATE

Determination of Particle Size Distribution

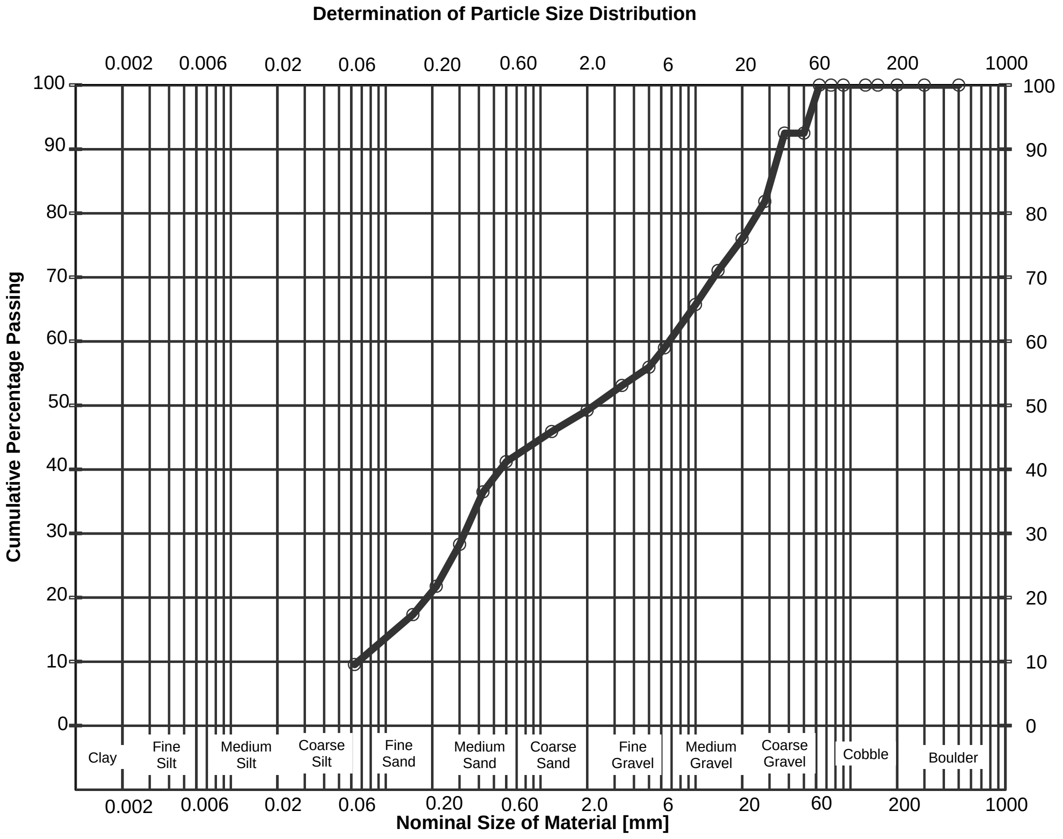
Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/1/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

TEST RESULTS **Laboratory Reference:** PL5462-1/1 **Pre-treatment for organic material:** N/A
Client Reference: B2

Sample Description: Dark brown slightly clayey SAND and GRAVEL

Material Specification: Not Required **Depth Top:** 1.20m
Location: BHA **Depth Base:** 1.70m
Source: **Supplier:**



Sieve Analysis

Sieve mm	%Passing
125	100
90	100
75	100
63	100
50	93
37.5	93
28	82
20	76
14	71
10	66
6.3	59
5.0	56
3.35	53
2.00	49
1.18	46
0.600	41
0.425	37
0.300	28
0.212	22
0.150	17
0.063	10
0.020	
0.006	
0.002	

Comments:

Approved Signatory: M. Hartnup - Laboratory Manager Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016 Page 1 of 1
Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA

GROUND ENGiNEERiNG LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

UKAS TESTING 8180

TEST CERTIFICATE

Determination of Particle Size Distribution

Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/2/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

TEST RESULTS **Laboratory Reference:** PL5462-1/2 **Pre-treatment for organic material:** N/A

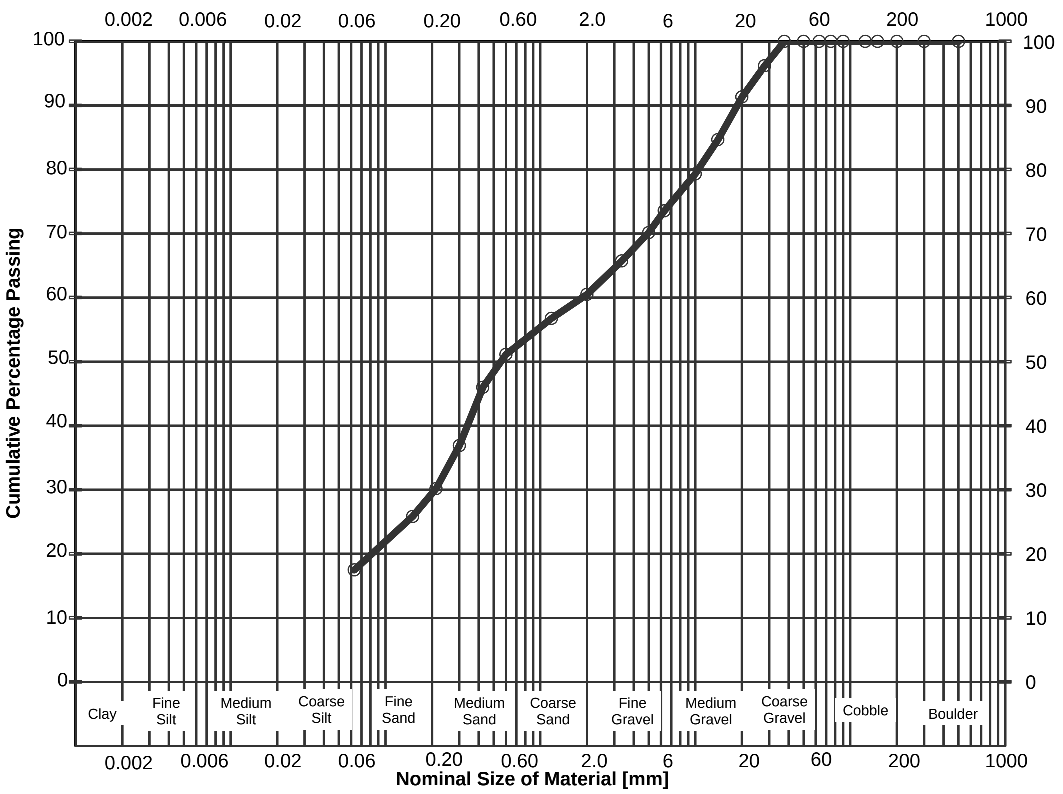
Client Reference: B3

Sample Description: Brown clayey very gravelly SAND

Material Specification: Not Required **Depth Top:** 2.00m

Location: BHA **Depth Base:** 2.50m

Source: **Supplier:**



Sieve Analysis

Sieve mm	%Passing
125	100
90	100
75	100
63	100
50	100
37.5	100
28	96
20	91
14	85
10	79
6.3	74
5.0	70
3.35	66
2.00	61
1.18	57
0.600	51
0.425	46
0.300	37
0.212	30
0.150	26
0.063	17
0.020	
0.006	
0.002	

Comments:

Approved Signatory: M. Hartnup - Laboratory Manager Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016 Page 1 of 1
Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA

UKAS TESTING
8180

GROUND ENGINEERING LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

TEST CERTIFICATE

Determination of Particle Size Distribution

Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/3/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

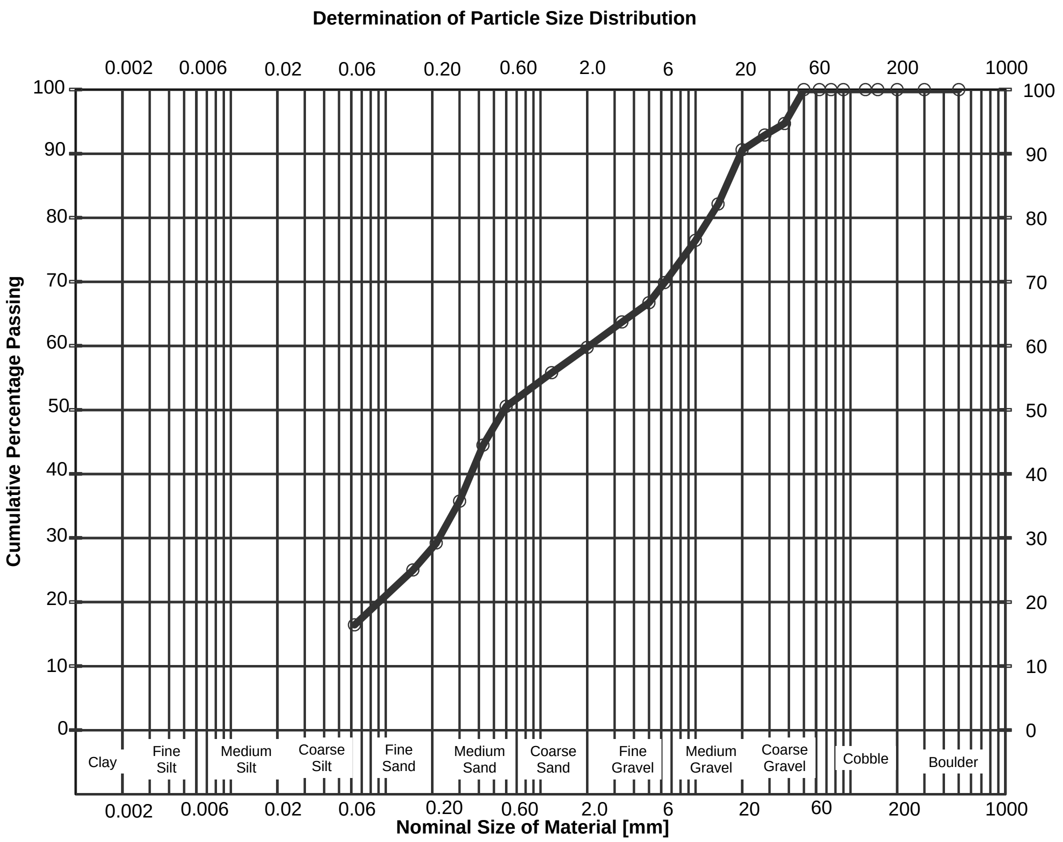
TEST RESULTS

Laboratory Reference: PL5462-1/3
Client Reference: B4
Pre-treatment for organic material: N/A

Sample Description: Brown clayey very gravelly SAND

Material Specification: Not Required
Location: BHA
Source:

Depth Top: 3.00m
Depth Base: 3.30m
Supplier:



Sieve Analysis

Sieve mm	**%Passing**
125	100
90	100
75	100
63	100
50	100
37.5	95
28	93
20	91
14	82
10	76
6.3	70
5.0	67
3.35	64
2.00	60
1.18	56
0.600	51
0.425	44
0.300	36
0.212	29
0.150	25
0.063	16
0.020	
0.006	
0.002	

Comments:

Approved Signatory: M. Hartnup - Laboratory Manager

Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016

Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA

GROUND ENGiNEERiNG LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

UKAS TESTING 8180

TEST CERTIFICATE

Determination of Particle Size Distribution

Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

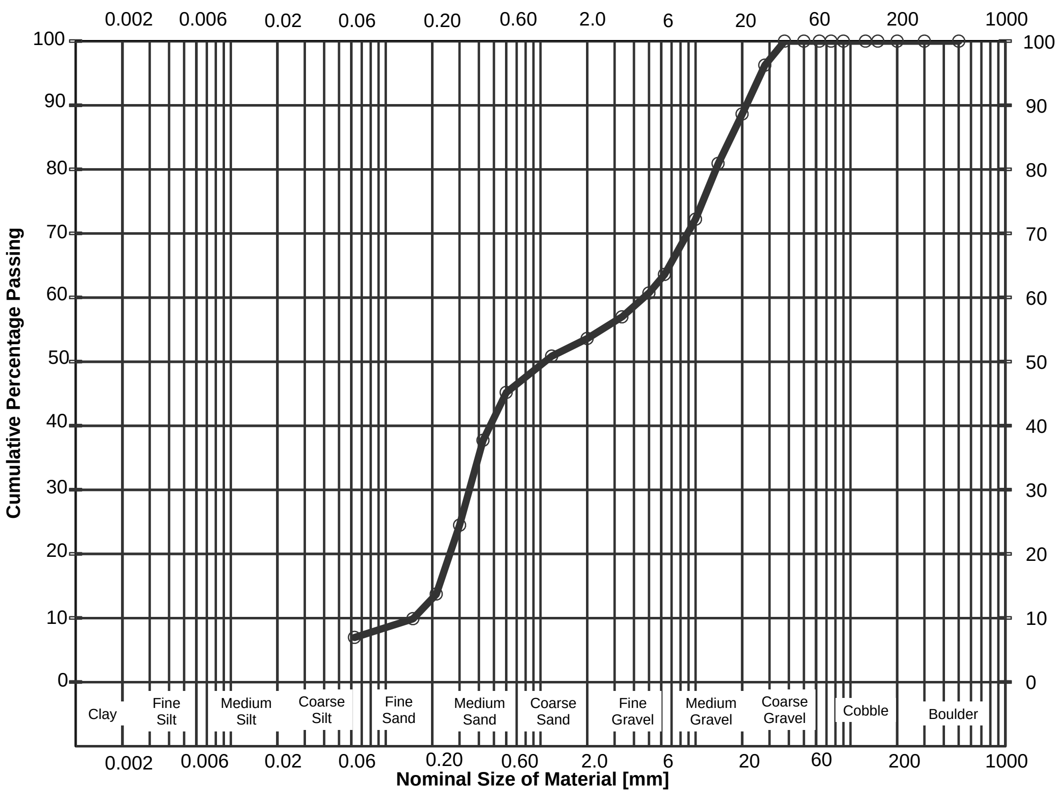
Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/4/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

TEST RESULTS **Laboratory Reference:** PL5462-1/4 **Pre-treatment for organic material:** N/A
Client Reference: B5

Sample Description: Dark brown slightly clayey SAND and GRAVEL

Material Specification: Not Required **Depth Top:** 3.30m
Location: BHA **Depth Base:** 3.60m
Source: **Supplier:**

Determination of Particle Size Distribution



Sieve Analysis

Sieve mm	%Passing
125	100
90	100
75	100
63	100
50	100
37.5	100
28	96
20	89
14	81
10	72
6.3	64
5.0	61
3.35	57
2.00	54
1.18	51
0.600	45
0.425	38
0.300	24
0.212	14
0.150	10
0.063	7
0.020	
0.006	
0.002	

Comments:

Approved Signatory: M. Hartnup - Laboratory Manager Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016 Page 1 of 1
Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA

GROUND ENGiNEERiNG
LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

UKAS TESTING
8180

TEST CERTIFICATE

Determination of Particle Size Distribution

Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

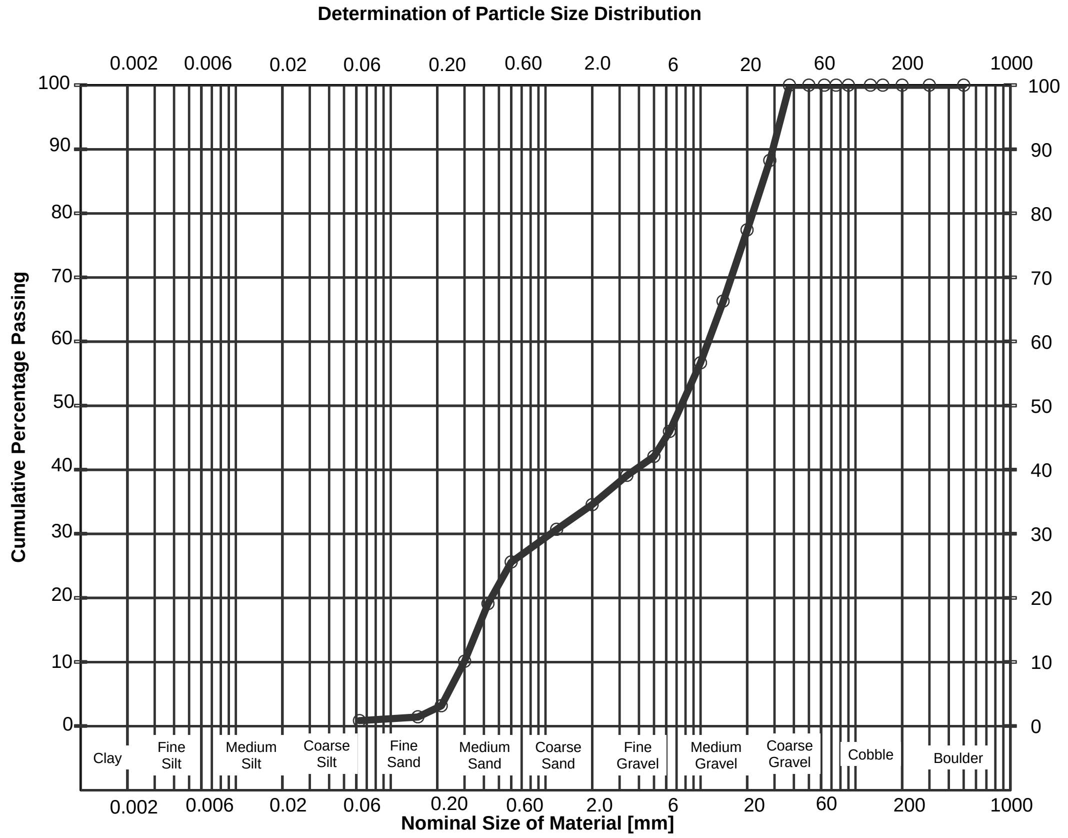
Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/5/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

TEST RESULTS

Laboratory Reference: PL5462-1/5
Client Reference: B6
Pre-treatment for organic material: N/A

Sample Description: Brown sandy GRAVEL

Material Specification: Not Required
Location: BHA
Source:
Depth Top: 4.20m
Depth Base: 4.70m
Supplier:



Sieve Analysis

Sieve mm	%Passing
125	100
90	100
75	100
63	100
50	100
37.5	100
28	88
20	77
14	66
10	57
6.3	46
5.0	42
3.35	39
2.00	35
1.18	31
0.600	26
0.425	19
0.300	10
0.212	3
0.150	1
0.063	1
0.020	
0.006	
0.002	

Comments:

Approved Signatory: M. Hartnup - Laboratory Manager
Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016 Page 1 of 1
Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA

GROUND ENGINEERING LIMITED
Newark Road Peterborough
t: 01733 566566 f: 01733 315280
e: admin@groundengineering.co.uk

8180

TEST CERTIFICATE

Determination of Particle Size Distribution

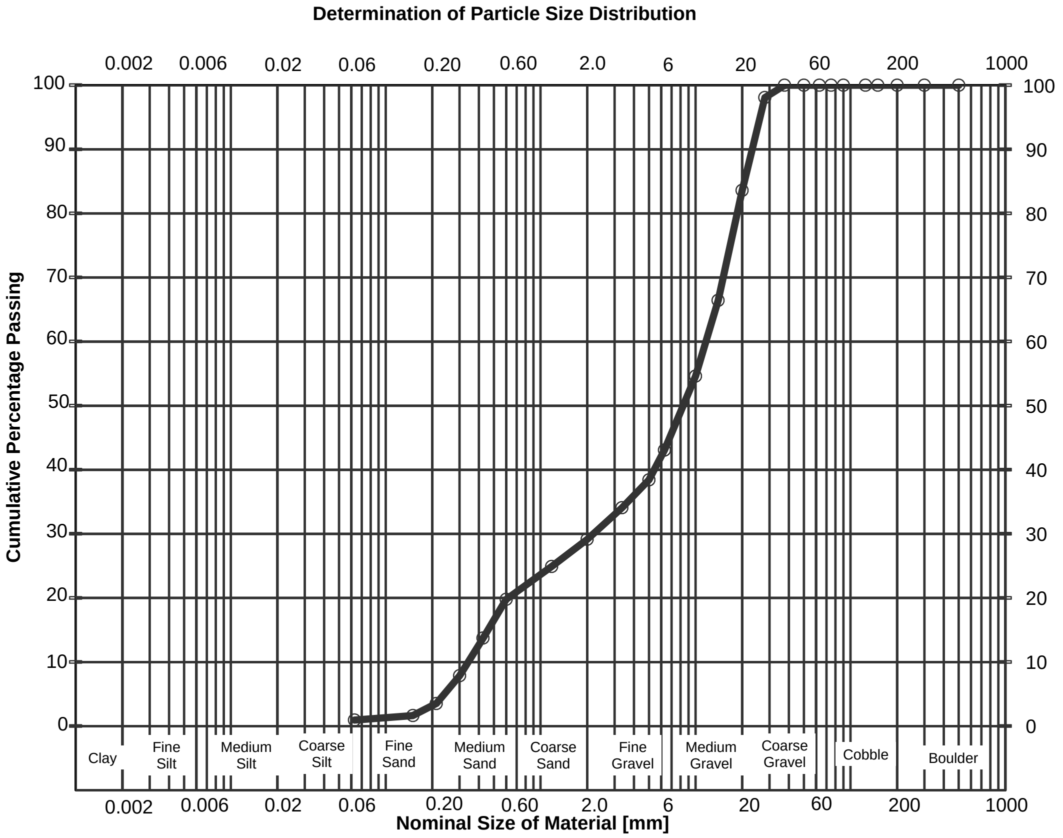
Tested in Accordance with BS 1377-2: 1990: Clause 9.2
Sieved Grading

Client:	Ground Engineering Ltd	Certificate Number:	PL5462-1/6/710-2
Client Address:	Newark Road	Client Reference:	C13870
	Peterborough	Lab Job Number:	PL5462-1
	PE1 5UA	Date Sampled:	Unknown
		Date Received:	16.06.2016
Contact:	Steve Fleming	Date Tested:	06.07.2016
		Certificate of Sampling:	N/A
Site Name:	27 Kings Mews	Sampling Certificate No.:	N/A
Site Address:	London WC1	Sampled By:	Client

TEST RESULTS **Laboratory Reference:** PL5462-1/6 **Pre-treatment for organic material:** N/A
Client Reference: B7

Sample Description: Brown sandy GRAVEL

Material Specification: Not Required **Depth Top:** 5.00m
Location: BHA **Depth Base:** 5.50m
Source: **Supplier:**



Sieve Analysis

Sieve mm	%Passing
125	100
90	100
75	100
63	100
50	100
37.5	100
28	98
20	84
14	66
10	55
6.3	43
5.0	38
3.35	34
2.00	29
1.18	25
0.600	20
0.425	14
0.300	8
0.212	4
0.150	2
0.063	1
0.020	
0.006	
0.002	

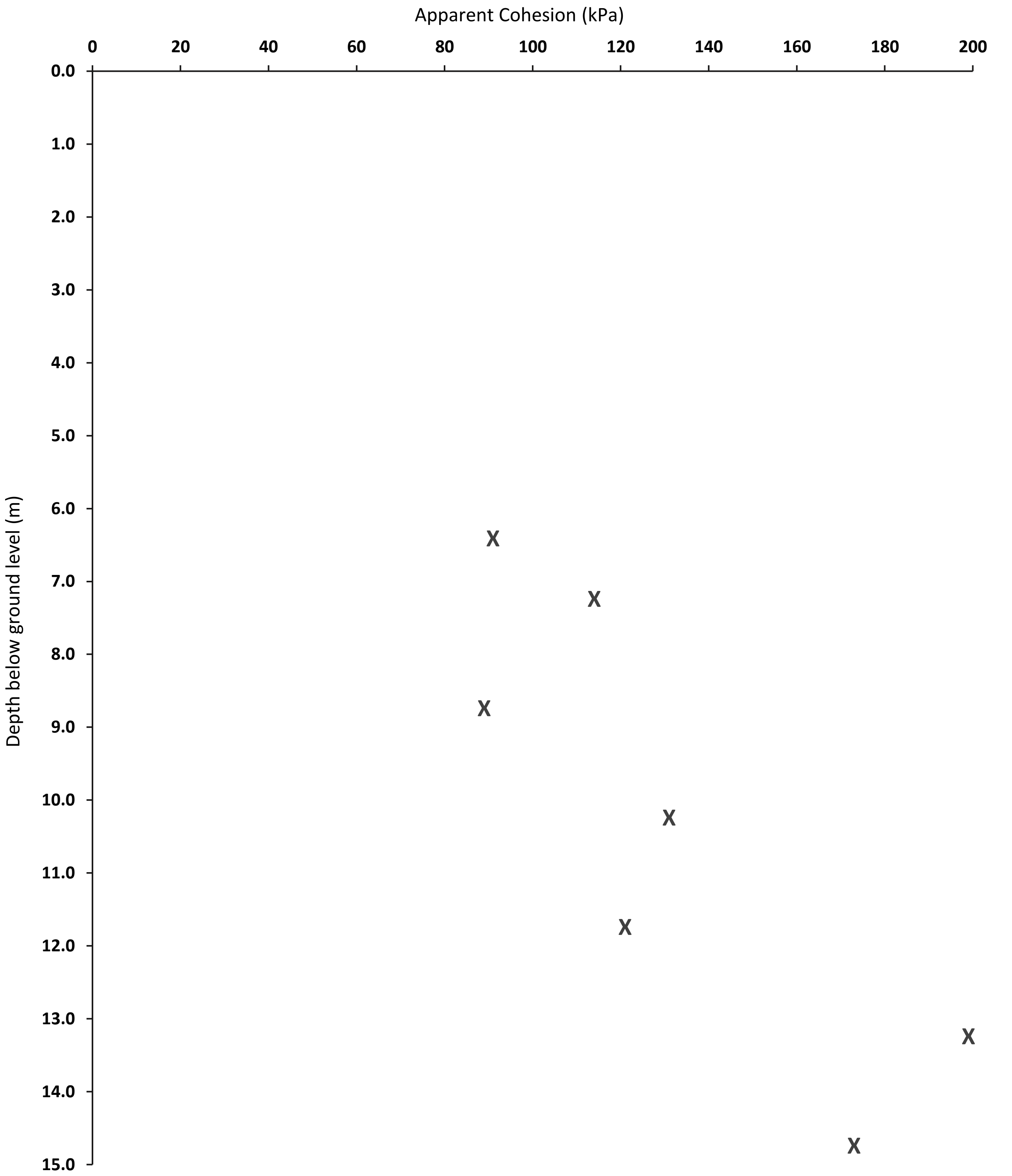
Comments:

Approved Signatory: M. Hartnup - Laboratory Manager Signed:

for and on behalf of Ground Engineering Ltd

Date Reported: 11.07.2016 Page 1 of 1
Form Number: GELab/C/709-2 Version 46

Registered in England & Wales
Registration Number: 6929574
Reg Office: Ground Engineering Ltd
Newark Rd, Peterborough PE1 5UA



Key: BHA = X

Apparent Cohesion (kPa) v Depth below ground level (m)

SITE:	**26 & 27 King's Mews, London WC1**
CLIENT:	**1156 Limited**
GROUND ENGINEERING LIMITED	Project No. C13870A
Newark Road, Peterborough	Figure 1

APPENDIX 1

CLASSIFICATION OF AGGRESSIVE CHEMICAL ENVIRONMENT FOR BURIED CONCRETE

TABLE C2 – AGGRESSIVE CHEMICAL ENVIRONMENT FOR CONCRETE (ACEC) CLASSIFICATION FOR BROWNFIELD LOCATIONS[a]

Table C2 Aggressive Chemical Environment for Concrete (ACEC) classification for brownfield locations[a]

Sulfate and magnesium						Groundwater		ACEC
Design Sulfate Class for location	2:1 water/soil extract[b]		Groundwater		Total potential sulfate[c]	Static water	Mobile water	Class for location
1	2 (SO_4 mg/l)	3 (Mg mg/l)	4 (SO_4 mg/l)	5 (Mg mg/l)	6 (SO_4 %)	7 (pH)[d]	8 (pH)[d]	9
DS-1	< 500		< 400		< 0.24	≥ 2.5		AC-1s
							> 6.5[d]	AC-1
							5.5–6.5	AC-2z
							4.5–5.5	AC-3z
							2.5–4.5	AC-4z
DS-2	500–1500		400–1400		0.24–0.6	> 5.5		AC-1s
							> 6.5	AC-2
						2.5–5.5		AC-2s
							5.5–6.5	AC-3z
							4.5–5.5	AC-4z
							2.5–5.5	AC-5z
DS-3	1600–3000		1500–3000		0.7–1.2	> 5.5		AC-2s
							> 6.5	AC-3
						2.5–5.5		AC-3s
							5.5–6.5	AC-4
							2.5–5.5	AC-5
DS-4	3100–6000	≤ 1200	3100–6000	≤ 1000	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4
						2.5–5.5		AC-4s
							2.5–6.5	AC-5
DS-4m	3100–6000	> 1200[e]	3100–6000	> 1000[e]	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4m
						2.5–5.5		AC-4ms
							2.5–6.5	AC-5m
DS-5	> 6000	≤ 1200	> 6000	≤ 1000	> 2.4	> 5.5		AC-4s
						2.5–5.5	≥ 2.5	AC-5
DS-5m	> 6000	> 1200[e]	> 6000	> 1000[e]	> 2.4	> 5.5		AC-4ms
						2.5–5.5	≥ 2.5	AC-5m

Notes

a Brownfield locations are those sites, or parts of sites, that might contain chemical residues produced by or associated with industrial production (Section C5.1.3).
b The limits of Design Sulfate Classes based on 2:1 water/soil extracts have been lowered from previous Digests (Box C7).
c Applies only to locations where concrete will be exposed to sulfate ions (SO_4), which may result from the oxidation of sulfides such as pyrite, following ground disturbance (Appendix A1 and Box C8).
d An additional account is taken of hydrochloric and nitric acids by adjustment to sulfate content (Section C5.1.3).
e The limit on water-soluble magnesium does not apply to brackish groundwater (chloride content between 12 000 mg/l and 17 000 mg/l). This allows 'm' to be omitted from the relevant ACEC classification. Seawater (chloride content about 18 000 mg/l) and stronger brines are not covered by this table.

Explanation of suffix symbols to ACEC Class

- Suffix 's' indicates that the water has been classified as static.
- Concrete placed in ACEC Classes that include the suffix 'z' have primarily to resist acid conditions and may be made with any of the cements in Table D2 on page 42.
- Suffix 'm' relates to the higher levels of magnesium in Design Sulfate Classes 4 and 5.

Appendix 4 Drainage Calculations

Design Settings

Rainfall Methodology	FEH-22
Return Period (years)	2
Additional Flow (%)	0
CV	0.750
Time of Entry (mins)	5.00
Maximum Time of Concentration (mins)	30.00
Maximum Rainfall (mm/hr)	50.0
Minimum Velocity (m/s)	1.00
Connection Type	Level Soffits
Minimum Backdrop Height (m)	0.200
Preferred Cover Depth (m)	1.200
Include Intermediate Ground	✓
Enforce best practice design rules	✓

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Easting (m)	Northing (m)	Depth (m)
Roof Storage	0.017	5.00	100.200		-17.252	94.363	0.200
Outfall			100.200	300	-3.577	94.259	0.369

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
1.000	Roof Storage	Outfall	10.000	0.600	100.000	99.831	0.169	59.0	100	5.17	50.0

Name	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
1.000	1.004	7.9	2.3	0.100	0.269	0.017	0.0	37	0.873

Pipeline Schedule

Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.000	10.000	59.0	100	Circular	100.200	100.000	0.100	100.200	99.831	0.269

Link	US Node	Node Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	Roof Storage	Junction	Outfall	300	Manhole	Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Link	IL (m)	Dia (mm)
Roof Storage	-17.252	94.363	100.200	0.200				
						1.000	100.000	100
Outfall	-3.577	94.259	100.200	0.369	300	1.000	99.831	100

Simulation Settings

Rainfall Methodology	FEH-22	Drain Down Time (mins)	240	30 year (l/s)	4.9
Summer CV	0.750	Additional Storage (m³/ha)	20.0	100 year (l/s)	6.7
Winter CV	0.840	Check Discharge Rate(s)	✓	Check Discharge Volume	x
Analysis Speed	Normal	1 year (l/s)	1.3		
Skip Steady State	x	2 year (l/s)	1.6		

Storm Durations

15	30	60	120	180	240	360	480	600	720	960	1440

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	0	0	0
30	20	0	0
100	25	0	0

Pre-development Discharge Rate

Site Makeup	Greenfield	Betterment (%)	0
Greenfield Method	ReFH2	Q 1 year (l/s)	1.3
Region	England, Wales, NI	Q 2 year (l/s)	1.6
Include Baseflow	x	Q 30 year (l/s)	4.9
Positively Drained Area (ha)	1.000	Q 100 year (l/s)	6.7

Node Roof Storage Online Orifice Control

Flap Valve	x	Invert Level (m)	100.000	Diameter (m)	0.050
Downstream Link	1.000	Design Depth (m)	0.200	Discharge Coefficient	0.600
Replaces Downstream Link	✓	Design Flow (l/s)	2.0		

Node Roof Storage Carpark Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Invert Level (m)	100.000	Slope (1:X)	80.0
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Depth (m)	
Safety Factor	2.0	Width (m)	5.000	Inf Depth (m)	
Porosity	1.00	Length (m)	12.000		

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)	Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
2 year 15 minute summer	107.507	30.421	2 year 480 minute summer	13.148	3.475
2 year 15 minute winter	75.444	30.421	2 year 480 minute winter	8.735	3.475
2 year 30 minute summer	67.853	19.200	2 year 600 minute summer	10.662	2.916
2 year 30 minute winter	47.616	19.200	2 year 600 minute winter	7.285	2.916
2 year 60 minute summer	43.928	11.609	2 year 720 minute summer	9.403	2.520
2 year 60 minute winter	29.185	11.609	2 year 720 minute winter	6.319	2.520
2 year 120 minute summer	32.774	8.661	2 year 960 minute summer	7.564	1.992
2 year 120 minute winter	21.774	8.661	2 year 960 minute winter	5.011	1.992
2 year 180 minute summer	26.844	6.908	2 year 1440 minute summer	5.356	1.435
2 year 180 minute winter	17.449	6.908	2 year 1440 minute winter	3.599	1.435
2 year 240 minute summer	21.784	5.757	30 year +20% CC 15 minute summer	394.618	111.663
2 year 240 minute winter	14.473	5.757	30 year +20% CC 15 minute winter	276.925	111.663
2 year 360 minute summer	16.796	4.322	30 year +20% CC 30 minute summer	251.924	71.286
2 year 360 minute winter	10.918	4.322	30 year +20% CC 30 minute winter	176.789	71.286

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)	Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
30 year +20% CC 60 minute summer	164.173	43.386	100 year +25% CC 30 minute summer	345.257	97.696
30 year +20% CC 60 minute winter	109.072	43.386	100 year +25% CC 30 minute winter	242.286	97.696
30 year +20% CC 120 minute summer	104.943	27.733	100 year +25% CC 60 minute summer	226.010	59.728
30 year +20% CC 120 minute winter	69.722	27.733	100 year +25% CC 60 minute winter	150.156	59.728
30 year +20% CC 180 minute summer	80.693	20.765	100 year +25% CC 120 minute summer	144.018	38.060
30 year +20% CC 180 minute winter	52.452	20.765	100 year +25% CC 120 minute winter	95.682	38.060
30 year +20% CC 240 minute summer	63.202	16.702	100 year +25% CC 180 minute summer	111.455	28.681
30 year +20% CC 240 minute winter	41.990	16.702	100 year +25% CC 180 minute winter	72.448	28.681
30 year +20% CC 360 minute summer	46.909	12.071	100 year +25% CC 240 minute summer	87.913	23.233
30 year +20% CC 360 minute winter	30.492	12.071	100 year +25% CC 240 minute winter	58.408	23.233
30 year +20% CC 480 minute summer	35.918	9.492	100 year +25% CC 360 minute summer	66.036	16.993
30 year +20% CC 480 minute winter	23.863	9.492	100 year +25% CC 360 minute winter	42.925	16.993
30 year +20% CC 600 minute summer	28.690	7.847	100 year +25% CC 480 minute summer	50.957	13.466
30 year +20% CC 600 minute winter	19.603	7.847	100 year +25% CC 480 minute winter	33.854	13.466
30 year +20% CC 720 minute summer	25.014	6.704	100 year +25% CC 600 minute summer	40.878	11.181
30 year +20% CC 720 minute winter	16.811	6.704	100 year +25% CC 600 minute winter	27.930	11.181
30 year +20% CC 960 minute summer	19.799	5.214	100 year +25% CC 720 minute summer	35.725	9.575
30 year +20% CC 960 minute winter	13.115	5.214	100 year +25% CC 720 minute winter	24.009	9.575
30 year +20% CC 1440 minute summer	13.662	3.662	100 year +25% CC 960 minute summer	28.299	7.452
30 year +20% CC 1440 minute winter	9.182	3.662	100 year +25% CC 960 minute winter	18.746	7.452
100 year +25% CC 15 minute summer	538.091	152.261	100 year +25% CC 1440 minute summer	19.459	5.215
100 year +25% CC 15 minute winter	377.608	152.261	100 year +25% CC 1440 minute winter	13.078	5.215

Results for 2 year Critical Storm Duration. Lowest mass balance: 100.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	Roof Storage	23	100.053	0.053	1.8	0.6584	0.0000	OK
15 minute summer	Outfall	1	99.831	0.000	0.8	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
30 minute winter	Roof Storage	Orifice	Outfall	0.9	1.4

Results for 30 year +20% CC Critical Storm Duration. Lowest mass balance: 100.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	Roof Storage	26	100.124	0.124	6.5	3.2736	0.0000	FLOOD RISK
15 minute summer	Outfall	1	99.831	0.000	1.5	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
30 minute winter	Roof Storage	Orifice	Outfall	1.6	5.1

Results for 100 year +25% CC Critical Storm Duration. Lowest mass balance: 100.00%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	Roof Storage	28	100.150	0.150	8.9	4.7779	0.0000	FLOOD RISK
15 minute summer	Outfall	1	99.831	0.000	1.7	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Discharge Vol (m³)
30 minute winter	Roof Storage	Orifice	Outfall	1.8	7.0

BauderBLUE STORMvoid System

Simplest blue roof solution beneath hard landscaping on a pedestal support system

Courtyard podiums or terraces are ideal locations for this blue roof solution with a completely paved finish above the void space created by the pedestal system.

This blue roof solution incorporates open-jointed paving on a Bauder pedestal support system that covers the height of the H-Max. The weight loading of the paving must exceed any buoyancy forces that will be exerted on the pedestals. The STORMvoid system is likely to require additional ballast to prevent floatation if used on inverted blue roofs.

The Bauder pedestal range is used in the STORMvoid system with hard landscaping. Selection will depend on the performance required.
Options include:

Bauder Adjustable Pedestal System
Simple, high strength, low-cost pedestal units that achieve depths from 18mm to 955mm. The pedestals feature a 197mm diameter base to negate the need for additional load spreader.

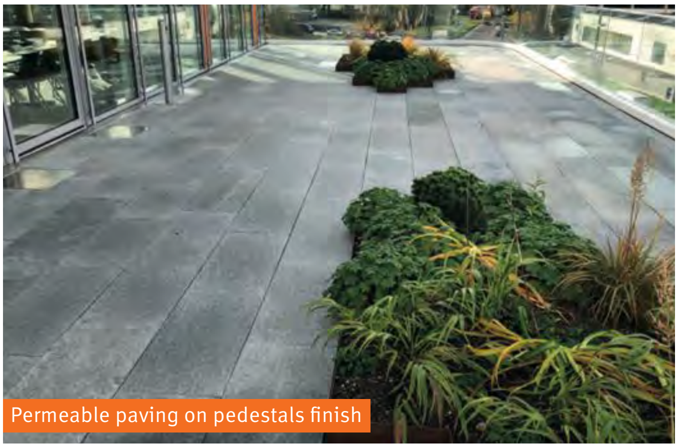
Bauder Non-Combustible Pedestal System
An all metal, non-combustible pedestal with a 170mm diameter base plate to spread load across the roof surface. The pedestal system can achieve a variety of heights from 42mm to 282mm.

Plus points

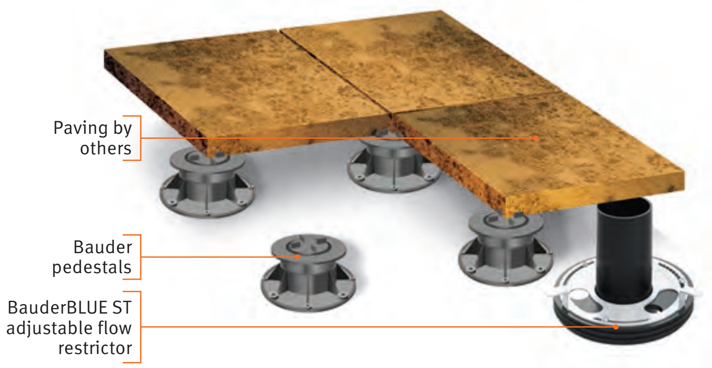
- Accommodates high volumes of water.
- Hard landscape finish.
- Often an ideal finish for simple roof areas.
- Ideal as part of a comprehensive BREEAM solution.
- Comprehensive range of guarantee packages to fulfil cover requirements for the project (dependant on system/product selection). For more information contact our technical dept for a sample guarantee outlining cover level, terms and conditions.

Roof finish options

- Paving.
- Metal decking.

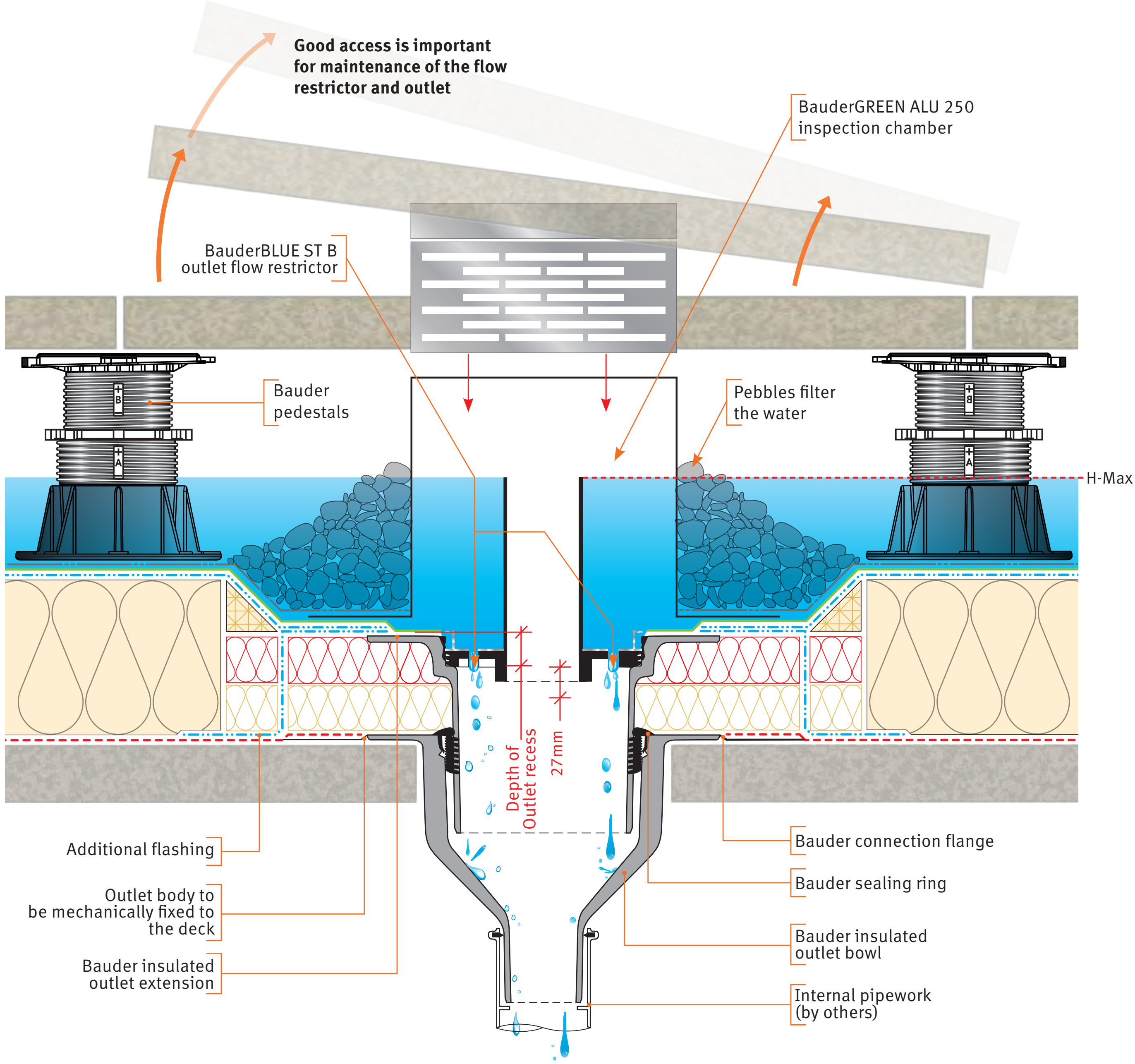

Permeable paving on pedestals finish


Bauder pedestals



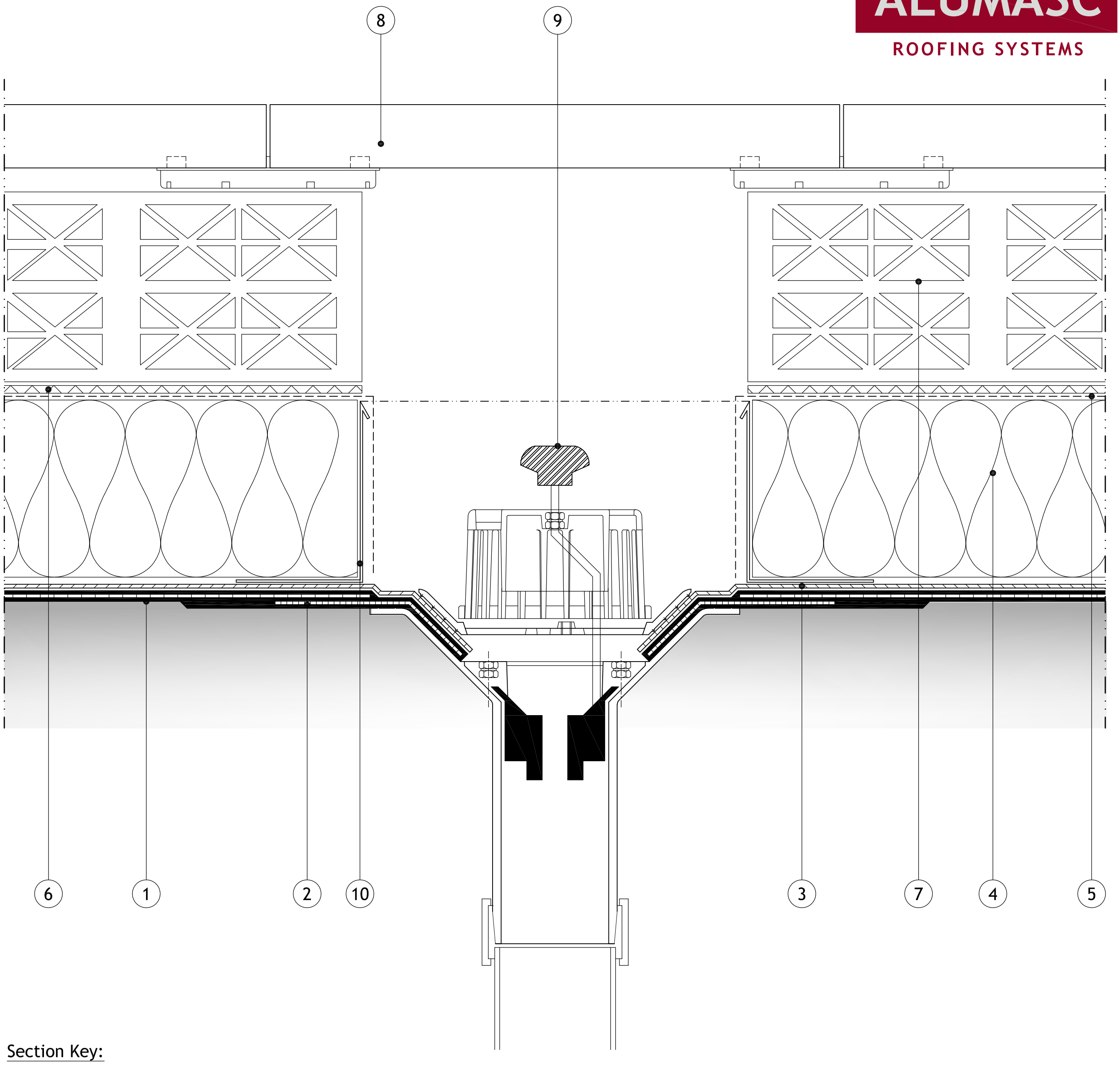
BauderBLUE STORMvoid System

The STORMvoid system uses the Bauder range of pedestals to form the blue roof void. Rainwater landing on the decking or paving drains through the open joints between them into the void below. Here the water is held via the BauderBLUE ST adjustable blue roof flow restrictor and discharged at the required rate for the roof. The system is ideal for simple hard landscaped blue roofs.



The discharge flow rate is controlled by the adjustable flow Restrictor. The pebble margin filters the rainwater to prevent debris being washed into the restrictor.	Sumping of the outlet increases efficiency of the flow restrictor.

ALUMASC
ROOFING SYSTEMS



Section Key:

1. Alumasc BluRoof Hot Melt Waterproofing.
2. FlexFlash UN uncured neoprene rubber reinforcement.
3. BluRoof protection sheet.
4. Alumasc Extruded Polystyrene thermal insulation, thickness determined to meet the U-value and dew point of the structure
5. Alumasc Polyethylene Separator Sheet.
6. Hydrodrain FC6 drainage layer.
7. BluRoof Void Former.
8. Precast concrete paving slabs, by others, seated on Harmer Uni-Ring flat roof paving supports.
9. Harmer AV outlet with Harmer BluRoof Insert, incorporating a clamping ring and domical grate.
10. Alumasc BluRoof Aluminium Chamber.

Notes:
1. This detail was prepared to serve as a guideline representing typical detailing conditions and illustrate the correct application of Alumasc products only. It may be necessary to modify this detail in whole or in part in accordance with specific project conditions, design or requirements.
2. Refer to the Alumasc project specification for product description and method of application.
3. Where applicable, any venting layer is not shown for clarity reasons.
4. Product data and safety data documents are available for download from http://www.alumascroofing.co.uk for all relevant Alumasc products.

alumasc
EXTERIOR BUILDING PRODUCTS

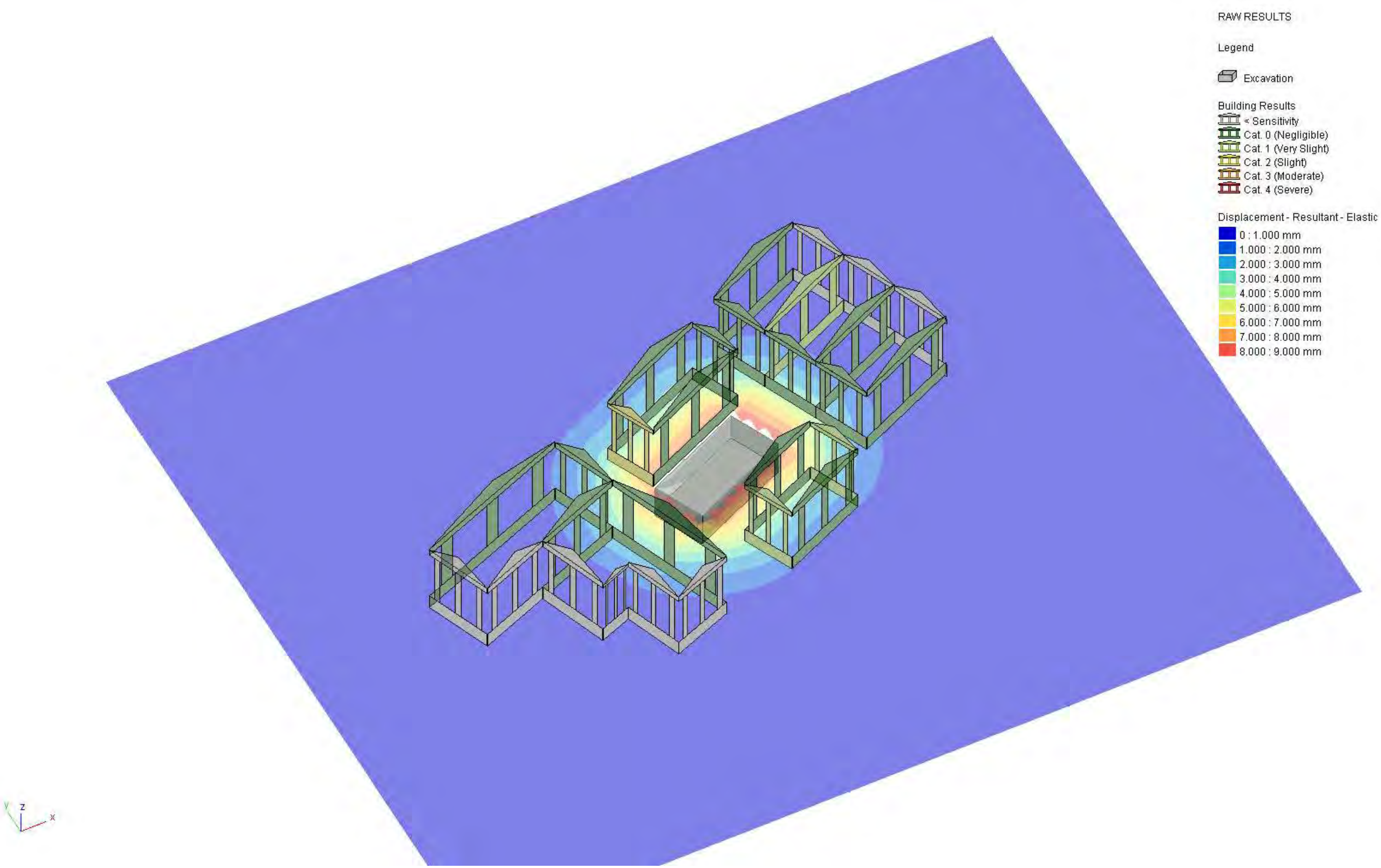
ALUMASC EXTERIOR BUILDING PRODUCTS LTD
White House Works, Bold Road, Sutton
St Helens, Merseyside, WA9 4JG, UK
Telephone: +44 (0)1744 648400
Facsimile: +44 (0)1744 648401
Website: www.alumascroofing.co.uk
E-mail: roofing@alumasc-exteriors.co.uk

Title:
BluRoof - Inverted Roof
Rainwater outlet with BluRoof Insert

Drawing No:	Scale:
HBR 03 04	Not to Scale
Revision:	Date:
Rev 1	Sept 2014

Appendix 6 GMA Outputs

Oasys	MILVUM ENGINEERINGSERVICES LTD	Job No. 26&27 Kings Mews	Sheet No.	Rev.
		Drg. Ref.		
		Made by CC	Date 06-Oct-2023	Checked / Date

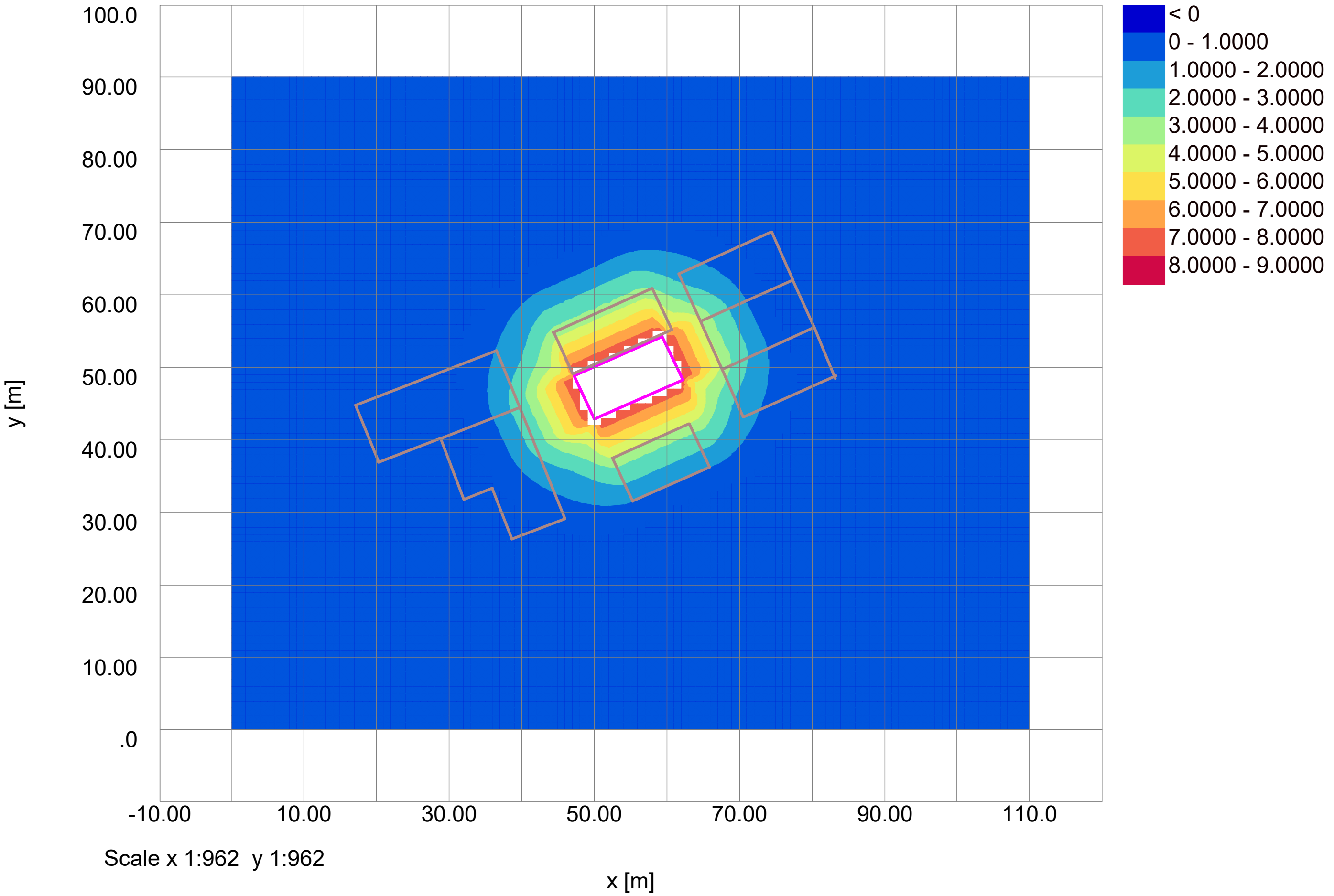


MILVUM
ENGINEERINGSERVICES LTD

Job No.	Sheet No.	Rev.
26&27 Kings Mews		
Drg. Ref.		

Made by	Date	Checked	Date
CC	06-Oct-2023		

Horizontal Displacement Contours: Grid 1 (level 100.000m) Interval 1mm

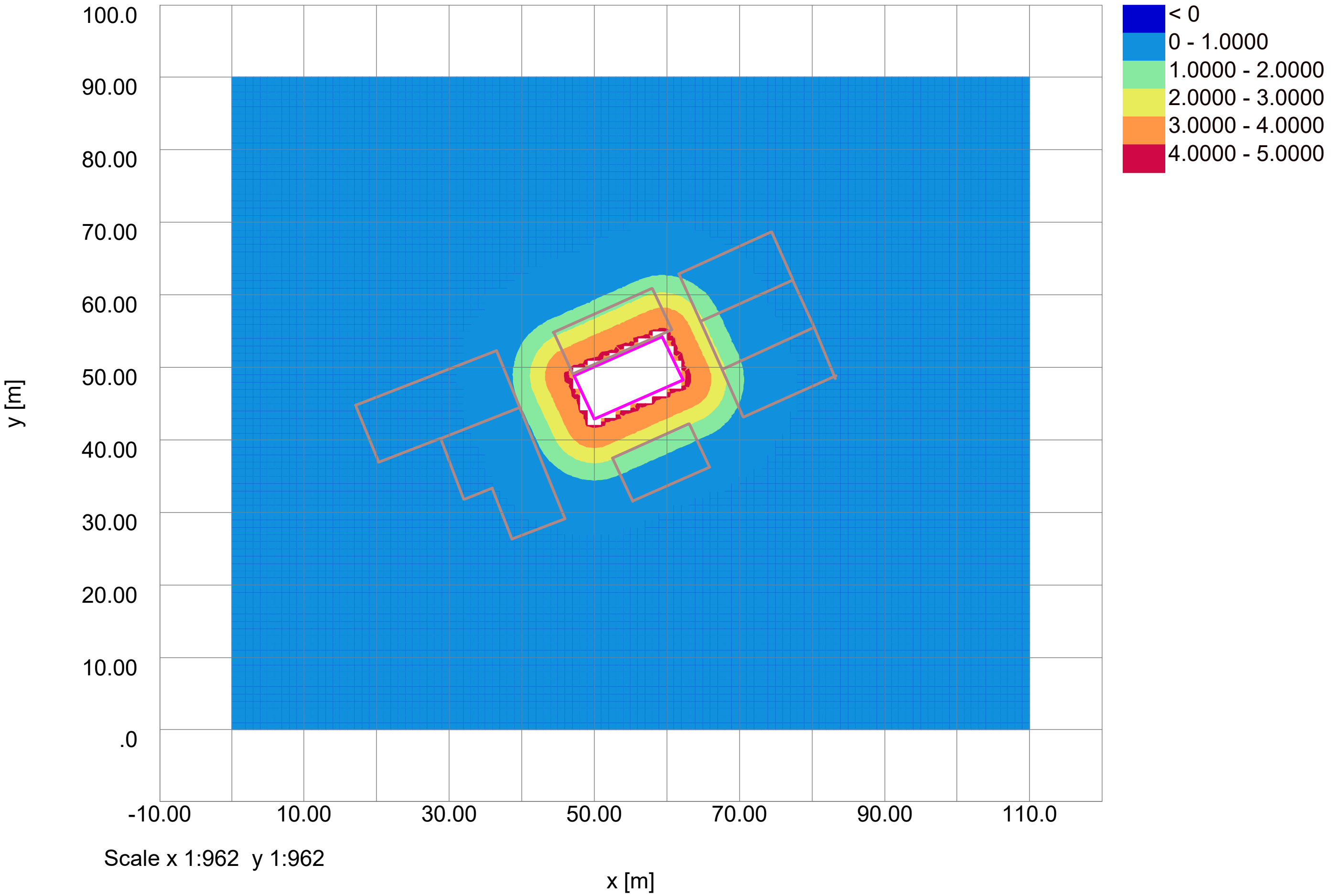


Scale x 1:962 y 1:962

MILVUM
ENGINEERINGSERVICES LTD

Job No.	Sheet No.	Rev.
26&27 Kings Mews		
Drg. Ref.		
Made by CC	Date 06-Oct-2023	Checked	Date

Vertical Settlement Contours: Grid 1 (level 100.000m) (Interval 1mm)



Scale x 1:962 y 1:962

Specific Building Damage Results - Critical Segments within Each Building

Stage: Ref.	Stage: Name	Specific Building: Ref.	Specific Building: Name	Parameter	Critical Sub-Building	Critical Segment	Start [m]	End [m]	Curvature	Max Slope	Max Settlement [mm]	Max Tensile Strain [%]	Min Radius of Curvature (Hogging) [m]	Min Radius of Curvature (Sagging) [m]	Damage Category
0	Base Model	0	2-4-A	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	2-4-B	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	2-4-C	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	2-4-D	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	2-4-E	Max Slope	Sub 5	1	8.1895	11.600	None	**300.25E-6**	0.89331	0.038774	-	-	0 (Negligible)
				Max Settlement	Sub 5	1	8.1895	11.600	None	300.25E-6	**0.89331**	0.038774	-	-	0 (Negligible)
				Max Tensile Strain	Sub 5	1	8.1895	11.600	None	300.25E-6	0.89331	**0.038774**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	2-4-F	Max Slope	Sub 6	2	1.9889	10.323	None	**106.88E-6**	0.88589	0.0014196	-	-	0 (Negligible)
				Max Settlement	Sub 6	1	0.0	1.9889	None	18.772E-6	**0.92312**	117.34E-6	-	-	0 (Negligible)
				Max Tensile Strain	Sub 6	3	10.323	14.956	None	106.88E-6	0.46733	**0.0050048**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	5-A	Max Slope	Sub 7	1	0.0	6.1997	None	**125.14E-6**	0.93897	0.0028305	-	-	0 (Negligible)
				Max Settlement	Sub 7	1	0.0	6.1997	None	125.14E-6	**0.93897**	0.0028305	-	-	0 (Negligible)
				Max Tensile Strain	Sub 7	1	0.0	6.1997	None	125.14E-6	0.93897	**0.0028305**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	5-B	Max Slope	Sub 8	1	0.0	2.0856	None	**78.966E-6**	0.27804	0.013833	-	-	0 (Negligible)
				Max Settlement	Sub 8	1	0.0	2.0856	None	78.966E-6	**0.27804**	0.013833	-	-	0 (Negligible)
				Max Tensile Strain	Sub 8	1	0.0	2.0856	None	78.966E-6	0.27804	**0.013833**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	5-C	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	5-D	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	25-A	Max Slope	Sub 11	1	0.0	2.0949	None	**469.15E-6**	2.6058	0.043918	-	-	0 (Negligible)
				Max Settlement	Sub 11	2	2.0949	6.3000	None	469.15E-6	**3.9690**	0.014543	-	-	0 (Negligible)
				Max Tensile Strain	Sub 11	1	0.0	2.0949	None	469.15E-6	2.6058	**0.043918**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	25-B	Max Slope	Sub 12	1	0.0	4.3199	None	**6.1186E-6**	3.9787	0.028754	-	-	0 (Negligible)
				Max Settlement	Sub 12	4	10.752	14.800	None	160.42E-9	**3.9799**	326.69E-6	-	-	0 (Negligible)
				Max Tensile Strain	Sub 12	1	0.0	4.3199	None	6.1186E-6	3.9787	**0.028754**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	
		0	25-C	Max Slope	Sub 13	1	0.0	4.0835	None	**499.87E-6**	4.0451	0.075827	-	-	
				Max Settlement	Sub 13	1	0.0	4.0835	None	499.87E-6	**4.0451**	0.075827	-	-	
				Max Tensile Strain	Sub 13	1	0.0	4.0835	None	499.87E-6	4.0451	**0.075827**	-	-	
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	25-D	Max Slope	Sub 14	3	10.425	14.800	None	**45.887E-6**	1.7223	0.0020151	-	-	0 (Negligible)

Stage: Ref.	Stage: Name	Specific Building: Ref.	Specific Building: Name	Parameter	Critical Sub-Building	Critical Segment	Start	End	Curvature	Max Slope	Max Settlement	Max Tensile Strain	Min Radius of Curvature (Hogging)	Min Radius of Curvature (Sagging)	Damage Category
				Max Settlement	Sub 14	3	10.425	14.800	None	45.887E-6	**1.7223**	0.0020151	-	-	0 (Negligible)
				Max Tensile Strain	Sub 14	3	10.425	14.800	None	45.887E-6	1.7223	**0.0020151**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	28-A	Max Slope	Sub 15	1	0.0	0.25490	None	**459.58E-6**	2.0454	0.070806	-	-	1 (Very Slight)
				Max Settlement	Sub 15	1	0.0	0.25490	None	459.58E-6	**2.0454**	0.070806	-	-	1 (Very Slight)
				Max Tensile Strain	Sub 15	1	0.0	0.25490	None	459.58E-6	2.0454	**0.070806**	-	-	1 (Very Slight)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	28-B	Max Slope	Sub 16	1	0.0	11.500	None	**8.5085E-9**	0.13704	35.763E-9	-	-	0 (Negligible)
				Max Settlement	Sub 16	1	0.0	11.500	None	8.5085E-9	**0.13704**	35.763E-9	-	-	0 (Negligible)
				Max Tensile Strain	Sub 16	1	0.0	11.500	None	8.5085E-9	0.13704	**35.763E-9**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	28-C	Max Slope	Sub 17	1	0.0	6.3466	None	**459.60E-6**	1.9262	0.047651	-	-	0 (Negligible)
				Max Settlement	Sub 17	2	6.3466	6.6000	None	459.60E-6	**2.0427**	0.070806	-	-	1 (Very Slight)
				Max Tensile Strain	Sub 17	2	6.3466	6.6000	None	459.60E-6	2.0427	**0.070806**	-	-	1 (Very Slight)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	28-D	Max Slope	Sub 18	1	0.0	11.500	None	**55.483E-9**	2.0461	35.763E-9	-	-	0 (Negligible)
				Max Settlement	Sub 18	1	0.0	11.500	None	55.483E-9	**2.0461**	35.763E-9	-	-	0 (Negligible)
				Max Tensile Strain	Sub 18	1	0.0	11.500	None	55.483E-9	2.0461	**35.763E-9**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	41-A	Max Slope	Sub 19	1	9.7791	13.900	None	**137.08E-6**	0.55765	0.0063969	-	-	0 (Negligible)
				Max Settlement	Sub 19	1	9.7791	13.900	None	137.08E-6	**0.55765**	0.0063969	-	-	0 (Negligible)
				Max Tensile Strain	Sub 19	1	9.7791	13.900	None	137.08E-6	0.55765	**0.0063969**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	41-B	Max Slope	Sub 20	2	3.0419	7.2000	None	**272.90E-6**	2.1925	0.0048563	-	-	0 (Negligible)
				Max Settlement	Sub 20	2	3.0419	7.2000	None	272.90E-6	**2.1925**	0.0048563	-	-	0 (Negligible)
				Max Tensile Strain	Sub 20	2	3.0419	7.2000	None	272.90E-6	2.1925	**0.0048563**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	41-C	Max Slope	Sub 21	1	0.0	6.9826	None	**456.55E-6**	2.1945	0.044533	-	-	0 (Negligible)
				Max Settlement	Sub 21	1	0.0	6.9826	None	456.55E-6	**2.1945**	0.044533	-	-	0 (Negligible)
				Max Tensile Strain	Sub 21	1	0.0	6.9826	None	456.55E-6	2.1945	**0.044533**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	41-D	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	43-A	Max Slope	Sub 23	1	0.0	2.8849	None	**43.376E-6**	2.2258	0.0032127	-	-	0 (Negligible)
				Max Settlement	Sub 23	1	0.0	2.8849	None	43.376E-6	**2.2258**	0.0032127	-	-	0 (Negligible)
				Max Tensile Strain	Sub 23	1	0.0	2.8849	None	43.376E-6	2.2258	**0.0032127**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	43-B	Max Slope	Sub 24	1	0.0	6.9826	None	**460.46E-6**	2.1773	0.050321	-	-	1 (Very Slight)
				Max Settlement	Sub 24	1	0.0	6.9826	None	460.46E-6	**2.1773**	0.050321	-	-	1 (Very Slight)
				Max Tensile Strain	Sub 24	1	0.0	6.9826	None	460.46E-6	2.1773	**0.050321**	-	-	1 (Very Slight)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	43-C	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
		0	45-A	Max Slope	Sub 26	1	0.0	5.3096	None	**269.15E-6**	2.1773	0.0052379	-	-	0 (Negligible)
				Max Settlement	Sub 26	1	0.0	5.3096	None	269.15E-6	**2.1773**	0.0052379	-	-	0 (Negligible)
				Max Tensile Strain	Sub 26	1	0.0	5.3096	None	269.15E-6	2.1773	**0.0052379**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	45-B	Max Slope	Sub 27	1	0.0	4.1963	None	**190.80E-6**	0.80680	0.0080340	-	-	0 (Negligible)
				Max Settlement	Sub 27	1	0.0	4.1963	None	190.80E-6	**0.80680**	0.0080340	-	-	0 (Negligible)
				Max Tensile Strain	Sub 27	1	0.0	4.1963	None	190.80E-6	0.80680	**0.0080340**	-	-	0 (Negligible)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-
		0	45-C	All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											
				All vertical displacements are less than the limit sensitivity.											

Appendix 7 Disclaimer

This report has been prepared by Milvum Engineer Services in its professional capacity as soil and groundwater specialists, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client, and is provided by Milvum Engineering Services solely for the use of its client (1156 Limited) and for reference by the London Borough of Camden.

The advice and opinions in this report should be read and relied on only in the context of the report as a whole, taking account of the terms of reference agreed with the client. The findings are based on the information made available to Milvum Engineering Services at the date of the report (and will have been assumed to be correct) and on current UK standards, codes, technology and practices as at that time. They do not purport to include any manner of legal advice or opinion. New information or changes in conditions and regulatory requirements may occur in future, which will change the conclusions presented here.

This report is confidential to the client. The client may submit the report to regulatory bodies, where appropriate. Should the client wish to release this report to any other third party for that party's reliance, Milvum Engineering Services may, by prior written agreement, agree to such release, provided that it is acknowledged that Milvum Engineering Services accepts no responsibility of any nature to any third party to whom this report or any part thereof is made known. Milvum Engineering Services accepts no responsibility for any loss or damage incurred as a result, and the third party does not acquire any rights whatsoever, contractual or otherwise, against Milvum Engineering Services except as expressly agreed with Milvum Engineering Services in writing.