West London &Suburban Property Investment Ltd.

80 Charlotte Street Amendment Scheme

Basement Impact Assessment

Issue | 8 December 2015

Job number 207329-00

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1 Introduction

Arup has been appointed by West London & Suburban Property Investments Ltd (WLSPIL) to provide structural and geotechnical engineering advice for the approved redevelopment of 80 Charlotte Street and 65 Whitfield Street (the Site), in Fitzrovia, Camden. Planning permission (reference 2010/6873/P) was granted for the redevelopment of the Site on 16 March 2012 and was subsequently implemented. This Basement Impact Assessment (BIA) supports an application for minor material amendments to the approved scheme, which includes further demolition of buildings and the deepening of the existing basement at the 80 Charlotte Street part of the Site only by approximately 2m. The approved scheme was not supported by a BIA as the approved scheme did not involve the excavation of the existing basement. This BIA supersedes the information on the basements previously submitted and approved, contained within the 'Façade Retention and Basement Proposals' document, dated December 2010.

The Site includes the entire block bounded by Charlotte Street, Howland Street, Whitfield St and Chitty Street, which covers an area of approximately 5700m2.

The majority of this block is to be re-developed into commercial office space as part of a mixed use scheme. The new building will be 9 storeys high, founded on piles. The core will be situated in the current courtyard area.

The objective of the BIA is to assess the potential impact of the development and basement construction on the structural stability of the neighbouring buildings and surrounding infrastructure and on the local groundwater and surface water environment.

As recommended by the Guidance for Subterranean Development (Arup, 2010) the BIA methodology comprises the following steps:

1. Initial **Screening** to identify whether there are matters of concern;
2. **Scoping** to further define the matters of concern identified in the screening stage and devise an approach to evaluate the potential impacts;
3. **Site investigation and study** to establish baseline conditions; and
4. **Assessment** of the information to determine the impact of the proposed basement on baseline conditions.

The information contained within this BIA has been produced to meet the requirements of a BIA as set out by Camden Planning Guidance – Basements and Lightwells (CPG4) including Camden Development Policies DP27 – Basements and Lightwells (London Borough of Camden, 2013) in order to assist LBC with their decision making process.

1.1 Summary of the report

This report includes assessment of the following:

- Surface flow and flooding;
- Groundwater flow; and
- Slope stability.

These are assessed in accordance with the Camden Planning Guidance for relevance to the amended development and potential impacts that the development may cause.

The site investigation information is reviewed to assess the existing ground conditions. Using this information the key basement construction impacts which were identified have been investigated. These include:

- Impacts to the groundwater flow caused by the proposed basement; and
- Ground movements due to the proposed basement.

The small changes in groundwater level predicted by the modelling are negligible and are expected to be within the normal range of seasonal fluctuation in the aquifer. The construction of the contiguous pile walls around the development are therefore expected to have no impact on adjacent structures or basements surrounding the Site.

The impact of ground movements has been assessed for the following surrounding infrastructure and structures:

- Public highways Charlotte Street, Howland Street, Whitfield St and Chitty Street
- TW mains and sewers beneath surrounding roads;
- Cast iron gas main beneath Howland St
- BT tunnel beneath Howland St
- 67-69 Whitifeld St; and
- Buildings across the roads from the site.

It is found that assessment of damage to buildings across the road from the site is within Category 1 “Very Slight”.

It is found that there is no unacceptable impact on surrounding services and infrastructure.

Assessment of damage for 67-69 Whitfield St is within Category 2 “Slight”. However this building is within the Site and will be redeveloped as part of the approved scheme. Therefore any small repair required is not an issue.

This BIA has been prepared by specialists with the following qualification:

Hilary Shields BA Engineering, MSc Soil Mechanics, DIC, CEng, MICE

Jon Leech MGeol, MSc hydrogeology, Chartered Geologist, Fellow Geological Society

Jonathan Gaunt CEng MCIBSE

2 Site context

2.1 Site location and existing structures

The Site is located in the London Borough of Camden, at approximately National Grid Reference TQ293818 as shown in Figure 1. The amendment application this BIA supports covers the area of the following buildings (Figure 2):

- 80 Charlotte St, including:
    - Cartwright Estate Block G (circa 1965, 6 storeys + basement + roof level);
    - Cartwright Estate Block H (circa 1965, 7 storeys + basement + roof level);
    - Cartwright Estate Block K (circa 1958, 7 storeys + basement + roof level);
    - Other buildings in the internal courtyard of 80 Charlotte St
- 71-81 Whitfield Street (Pre-1948, 5 storeys + basement + roof level);
- 10 & 15 Chitty Street (Pre-1948, 4 storeys + basement + roof level);

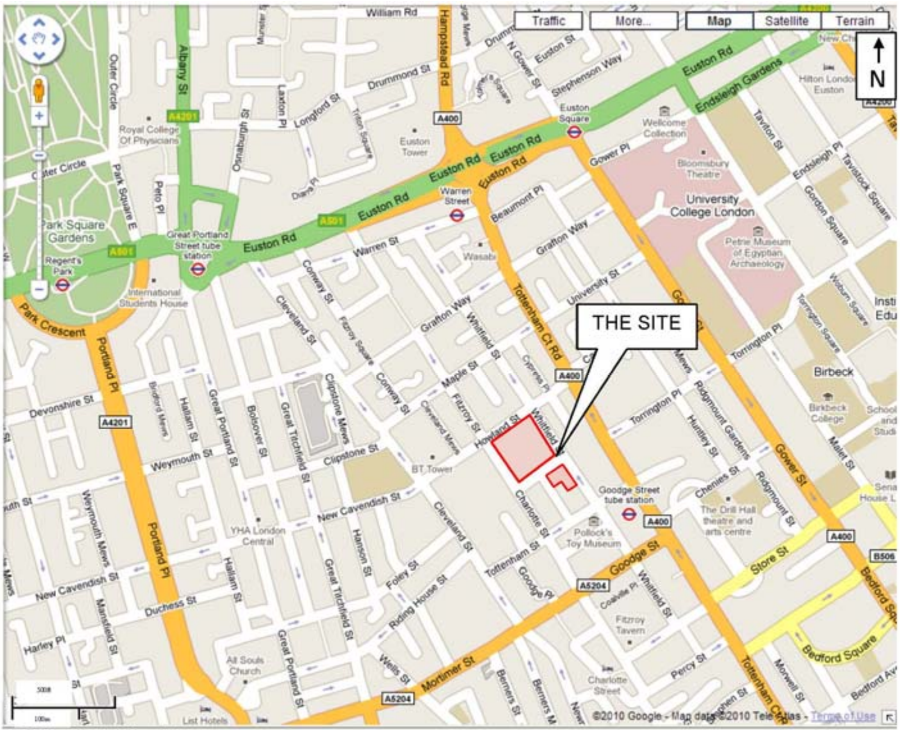
The above buildings make up almost the entire block bounded by Charlotte Street, Howland Street, Whitfield St and Chitty Street, which covers an area of approximately 5700m2. It should be noted that 67-69 Whitfield Street do not change under the amendment application this BIA supports.

The buildings are arranged around a courtyard space. Vehicular access to the courtyard space is provided at Block G on Howland Street and at 10 Chitty Street. The buildings are entered from the surrounding streets with steps up to ground floor level from external pavement level. They all have single lower ground floor (basement) levels.

A lightwell which is either open or has been covered extends around the outer edge of the site.

Structural drawings from archive searches show that Blocks G, H and K are of reinforced concrete construction apparently founded on piles while the remaining buildings are masonry and appear to be founded on shallow footings. The piles have been investigated in the two phases of site specific GI. At blocks G and H pile toe levels were found to vary between +12.4mOD and +15.6mOD. At Block K, pile toe levels were found to vary between +13.2mOD and +16.6mOD.

The Site is currently owned by WLSPIL and comprises vacant space. There is a building in the central courtyard area housing which was a licensed bar (not open to the public). Some of the basements are used for car parking.



@2010 Google-Map data

Figure 1 Site location (red line boundary includes 65 Whitfield St)

Figure 2 Existing buildings on site

2.2 Topography

The Site has no major topographical features. The land slopes very gently upwards to the north-west at a gradient of approximately 1:500. Figure 2 illustrates the existing site layout with representative photographs and levels of these existing basements. The street is at an elevation of approximately +28mOD. Existing survey data indicate the basement floor levels of the existing buildings range from an elevation of +23.1mOD (locally in the boiler room area) to +25.5mOD.

The courtyard area slopes from +26.5 in the east to +25.3mOD in the west.

2.3 Site history

A review of historical maps in the desk study (Arup, 2010) showed that the Site was undeveloped until 1746. Urban development at the site occurred between 1746 and 1813. The buildings are small and individual, possibly privately owned houses. The current Chitty Street was called North Street. The existing buildings at the centre of the Site are located around the North Street mews (later named North Court). The 1896 map from Ordnance Survey shows a change in the name of North Street to the current name, Chitty Street.

Goad fire insurance plans show the development of the Site from 1900 to 1966. The 1900 Goad plan shows the Site occupied by residential houses up to 4 storeys. No basements are identified. The 1948 Goad plan shows some of the buildings within the Site have been demolished, possibly due to World War II bomb damage. The buildings present have various uses, including metal works, welding facilities, rubber tyres storage, garages, electrical fittings, residential, and offices. 67-69 Whitfield Street has been built.

The 1957 Goad plan shows the Site was essentially commercial and industrial in nature. Some buildings were merged or refurbished by adding one or two floors and a basement level. 71-81 Whitfield Street has been built. The buildings located in the area currently occupied by Block K have been demolished. In 1963 Goad plan, block K appears to have been built and in 1966 Goad plan indicates that blocks H and G were built. According to Ordnance Survey maps, the site has essentially remained unchanged to the present.

2.4 Summary of geology and ground conditions

2.4.1 Published Geology

Records obtained from the British Geological Survey (Sheet 256 of the Geological Survey of Britain – Solid and Drift Edition) indicate the Site to be underlain by:

- River Terrace Deposits;
- London Clay;
- Lambeth Group;
- Thanet Sand; and

- Upper Chalk.

2.4.2 Summary of Site stratigraphy based on GI

A full description of ground investigation information for the Site and surrounding area is given in Section 8.

A summary of the Site stratigraphy is given in Table 1.

Stratum	Thickness at location investigated (m)	Top surface at location investigated (mOD)
Made Ground	0.6 to 6.5	+25.2 to +27.5
River Terrace Deposits	1.3 to 5.0	+19 to +23.9
London Clay	15.6 to 16.0	+18.1 to +20.6
Lambeth Group & Thanet Sands	21.7	+2.8 to +3.0
Chalk	unproven	-18.7

Table 1 site stratigraphy

2.4.3 Summary of groundwater level measured in GI

A full description of groundwater measurements at the Site and surrounding area is given in Section 8.

Groundwater level measured at the Site is approximately +22mOD.

2.5 Neighbouring infrastructure and buildings and nearby tunnels

2.5.1 Utilities

2.5.1.1 Thames Water – Water main

A 600mm diameter Trunc main pipeline runs beneath Charlotte Street at 1.1m below ground level (see Figure 3). Distribution mains also run around the perimeter of the Site beneath the roads at 0.9m below ground level.

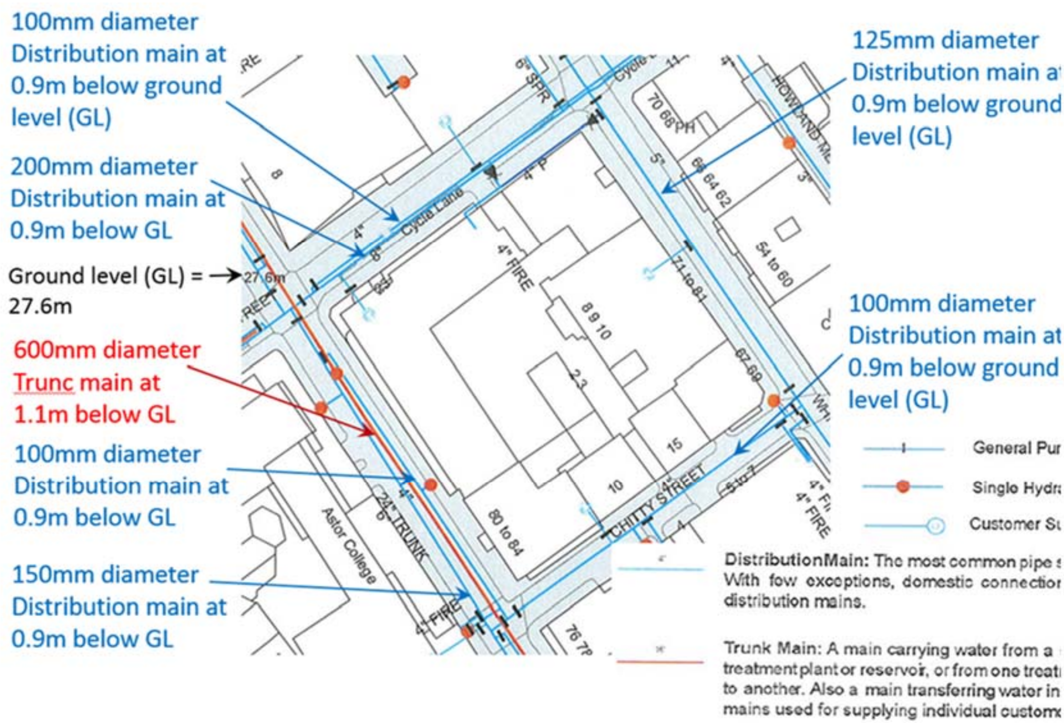


Figure 3 Location of Thames Water mains

2.5.1.2 Thames Water – Sewer

Thames Water sewers run beneath the surrounding roads as shown in Figure 4. The sewers are located at depths above the proposed formation level. The deepest invert level is +23.2mOD.

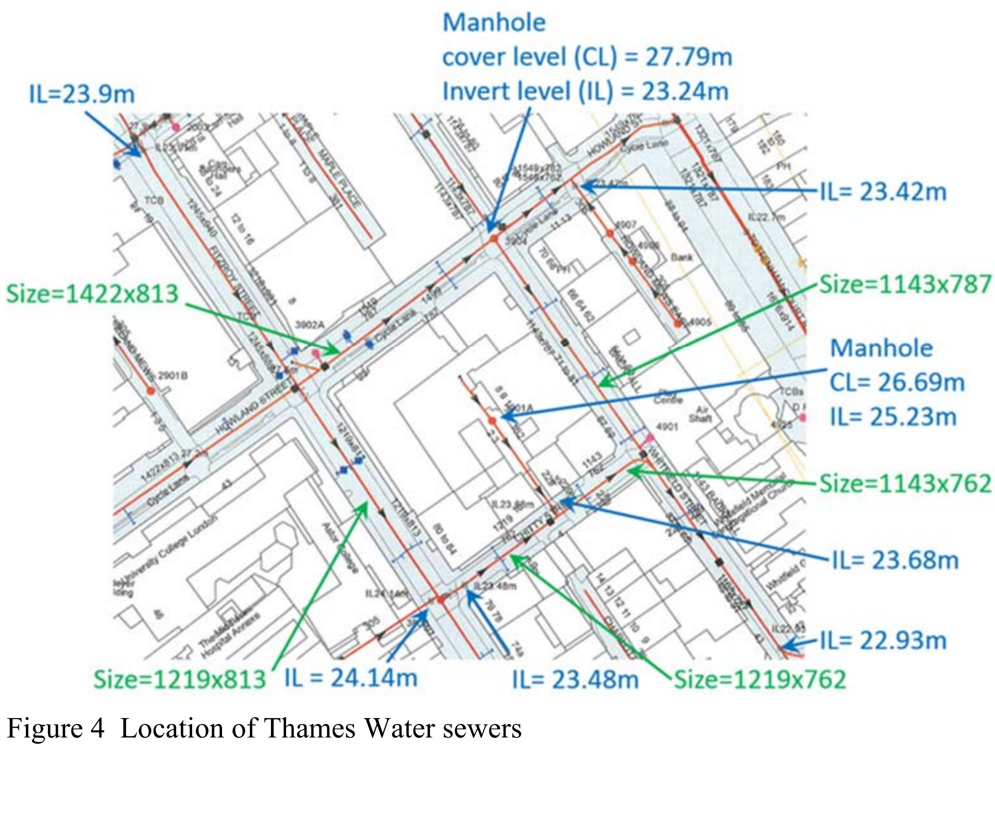


Figure 4 Location of Thames Water sewers

2.5.1.3 National grid gas

Gas distribution pipes run beneath the surrounding streets as shown in Figure 5. In particular a 300mm diameter Cast Iron gas pipe runs beneath Howland Street.

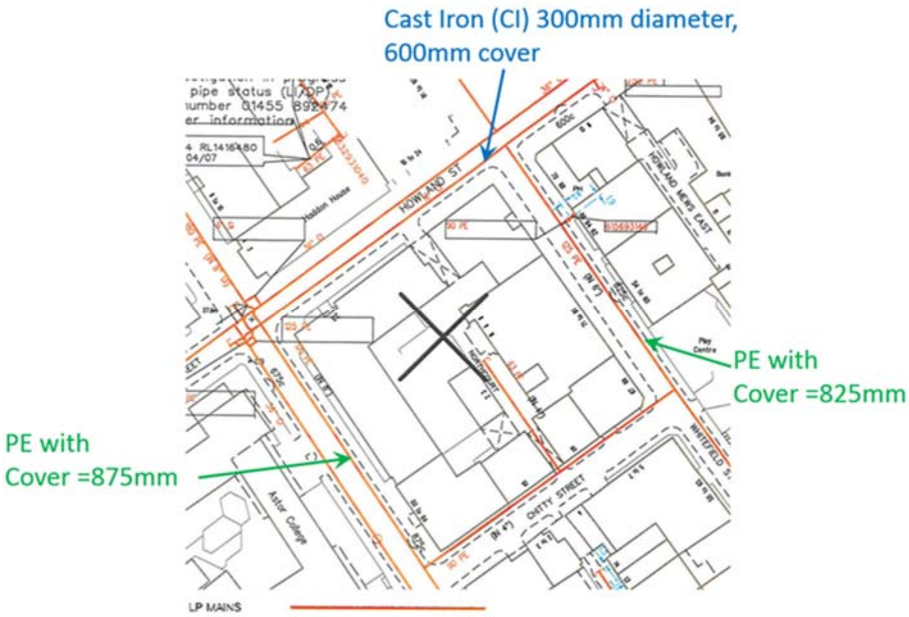


Figure 5 Location of cast iron gas main

2.5.2 BT tunnels

A BT tunnel runs adjacent to the Site location beneath Howland Street as shown in Figure 6 (based on the details received from a previous project). The crown levels of the tunnel at the junction of Charlotte St and Howland St is shown to be -11m OD Newlyn and hence it is located well below the proposed basement excavation level. The tunnel and the chamber internal diameters are shown to be 2.13m and 3.65m. The Site is located outside the minimum clearance zone of BT tunnels which is 2.0m. The guidelines suggest that pile and other construction works should not cause vibration on the tunnel in excess of 20mm/sec; and if there are piles within 3.0m of the BT tunnel, a position survey of the BT tunnel should be carried out.

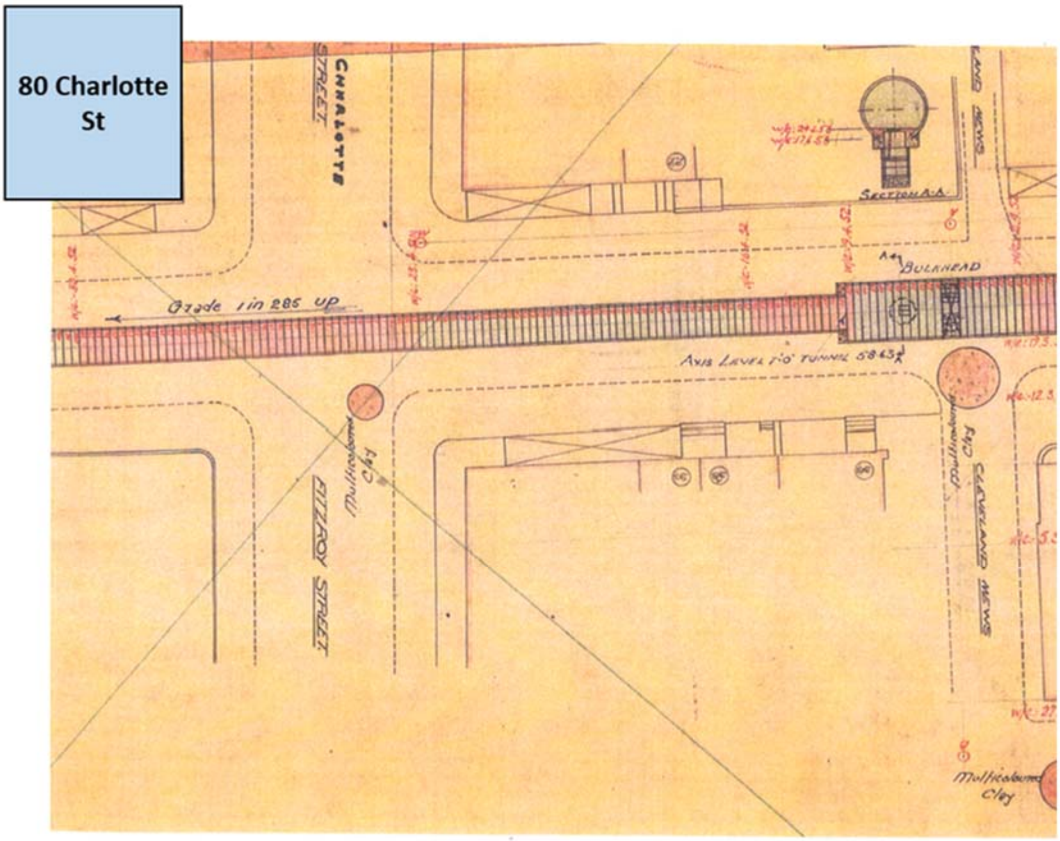


Figure 6 Plan of BT tunnel layout

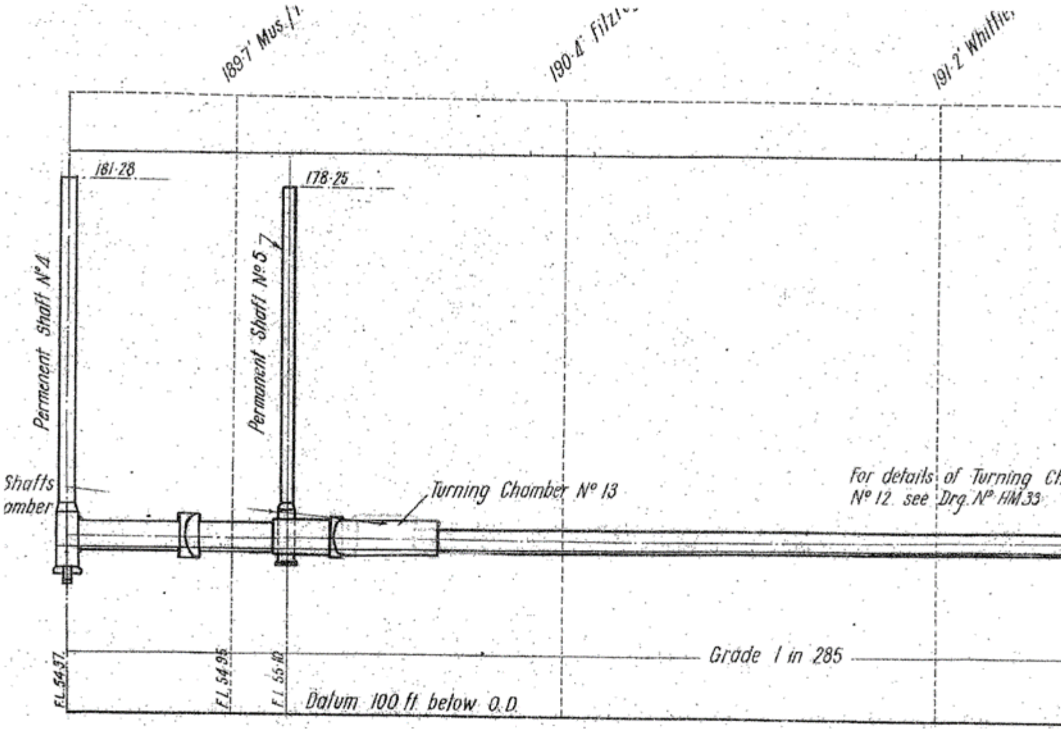


Figure 7 Section view of BT tunnel

2.5.3 Surrounding structures

67-69 Whitfield St is the only building directly adjacent to the site. The building is owned by the Client. The site GI has found that the building is founded on pads, founded on the Terrace Gravels at about +21.2mOD.

An extract from the Large Scale National Grid Data 1993-1995 showing the Site and surrounding buildings is shown in Figure 8. Whilst some of the surrounding buildings have been redeveloped since, they remain approximately within the footprints shown in the figure.

There is a minimum distance of 11m from the edge of the lightwell to the face of structures across the surrounding roads.

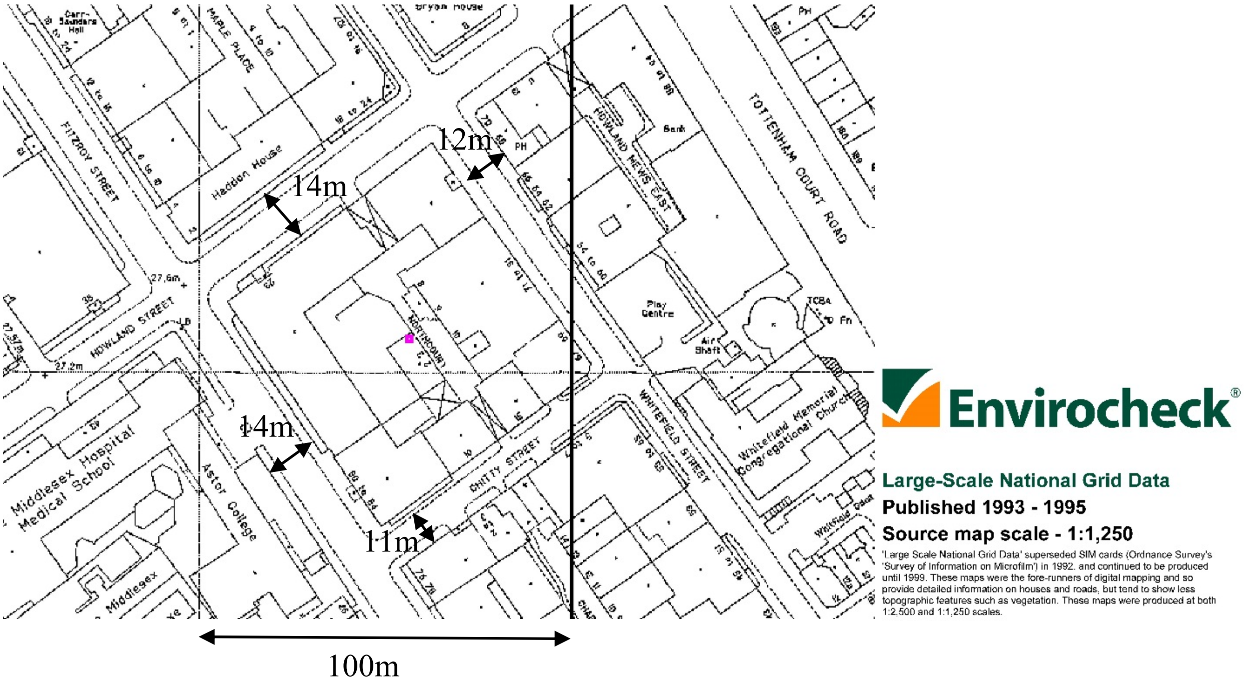


Figure 8 Extract from the Large Scale National Grid Data 1993-1995 Showing the Site and surrounding buildings

2.5.4 LUL Northern Line

The LUL northern line tunnels run at depth between Tottenham Court Rd and Whitfield St as shown in Figure 9. Based on information from Crossrail 1 at Tottenham Court Rd Station, the tunnel crown is expected to be at about 3mOD and the diameter of the running tunnels is 3.8m. The tunnels are more than 40m in plan from the edge of the site and are therefore not considered further in this impact assessment.

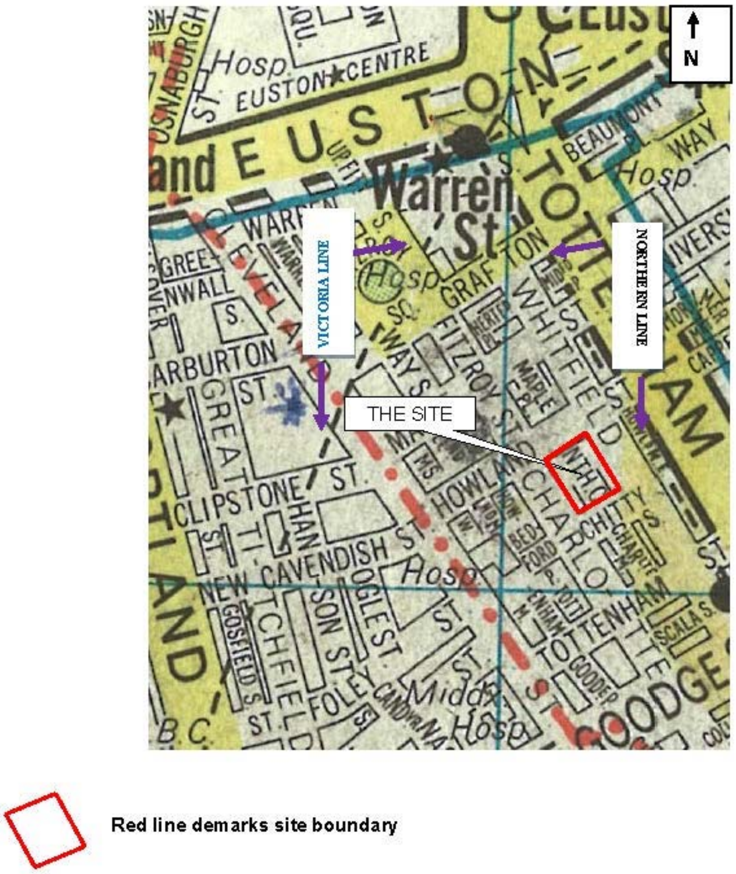


Figure 9 Location of LUL Northern Line

2.5.5 Crossrail 2 Safeguarding Zone

The Site sits outside the current proposed Crossrail 2 alignment (see Figure 10) and partially within the Crossrail 2 Safeguarding Zone (Figure 11).

The information has been obtained from the Crossrail 2 interactive website:

https://mm-evt.maps.arcgis.com/apps/webappviewer/index.html?id=ed525d6702d14122a7c1a3733d5b7ffd

The safeguarding map is located at:

http://crossrail2.co.uk/areas-safeguarded/

The level of the tunnel is not given on the Crossrail 2 website. However, the invert level of the Crossrail 1 platform tunnel at Tottenham Court Rd Station is 25m below ground level. The Crossrail 2 platform tunnel will go beneath the Crossrail 1 platforms (not far beneath). Platform diameter is 11m, so that puts invert of Crossrail 2 at least 36m below ground level. Crossrail running tunnel diameter is 7m so crown level is at most 29m below ground level. It would be reasonable to assume about 30m below ground level to the crown of Crossrail 2.

Since ground level at the site is about +28mOD this puts crown of crossrail about -2mOD in the vicinity of the site. This is about 7m below the deepest proposed pile toe level of +5mOD at the site.

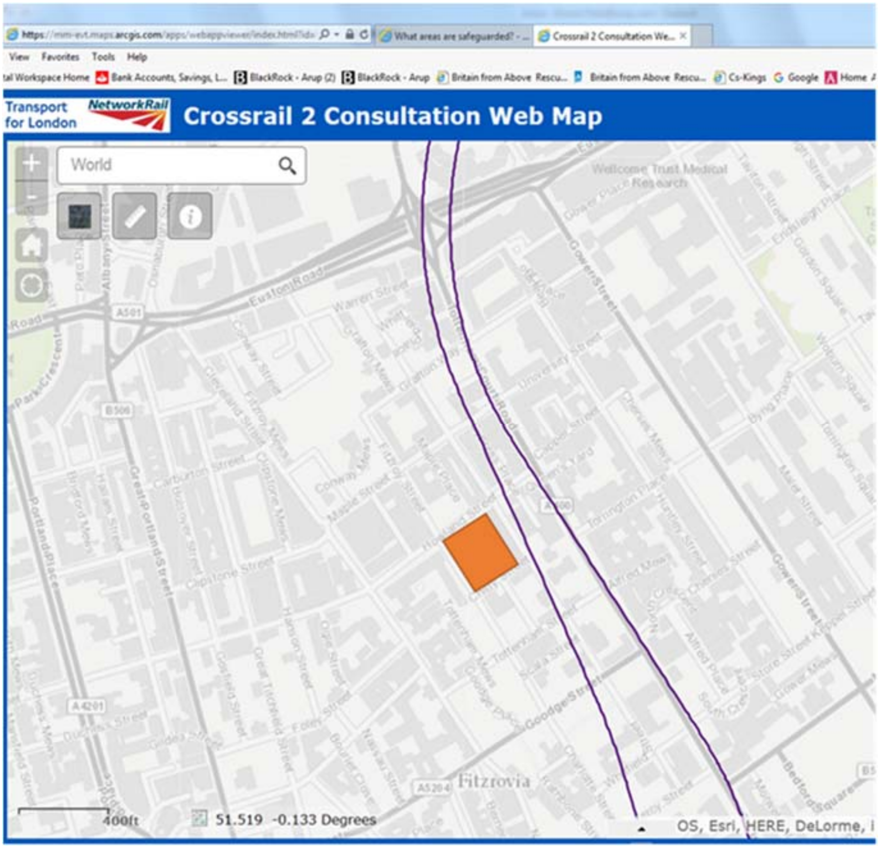


Figure 10 Crossrail 2 tunnel alignment as at time of writing

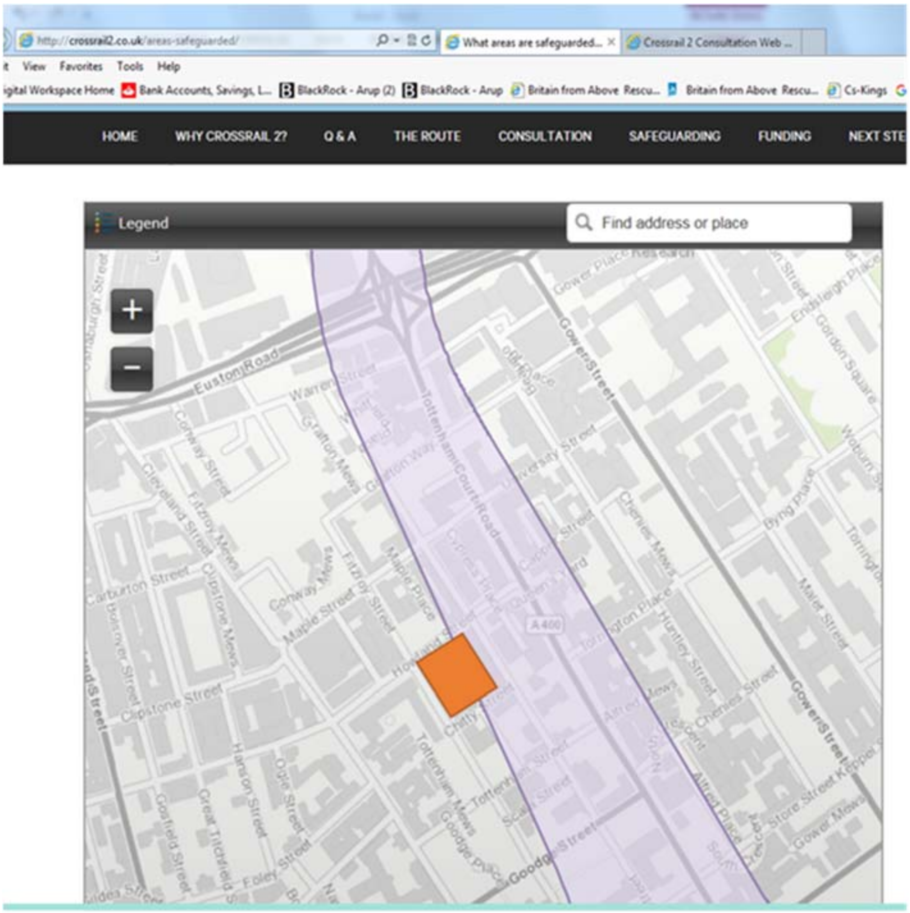


Figure 11 Crossrail 2 Extent of Safeguarded Zone

2.6 Hydrology

2.6.1 Rainfall and runoff

Rainfall in the area averages about 610 mm (Mayes, 1997), significantly less than the national annual average of about 900 mm. Rainfall in London is split almost equally over the seasons, with the winter months experiencing only marginally higher rainfall than summer months. However, the rainfall in summer months will often occur in a smaller number of intense rainfall events leading to peaks which can lead to flash flooding and overloading of sewer systems. Climate change predictions indicate that future winters may be wetter and summers drier, but that rainfall patterns may become more intense and the summer storms will become more frequent. Over time the standard of protection of existing sewers is likely to reduce leading to an increase in localised flooding incidents.

Evapotranspiration is typically about 450 mm/yr resulting in about 160 mm per year as “hydrologically effective” rainfall which is available to infiltrate into the ground or runoff as surface water flow.

The Site lies within the catchment of the River Fleet which shapes the eastern boundary of LB Camden.

The area around the Site, in central London, is highly developed with more than 80% of the surface covered with hard standing. Most of the rainfall in the area will runoff hard surface areas and be collected by the local sewer network.

2.6.2 Drainage

Surface drainage from the site currently drains into the surrounding sewer network shown in Figure 4.

Due to the lowered basement in the amended scheme the existing surface water drainage from the Site will be connected to the existing sewers in Whitfield Street and Chitty Street via new gravity connections. The use of anti-flood valves will protect against possible surcharging of the sewers.

2.6.3 Flood risk

Although Camden missed the serious national floods of 2007 & 2012 it is known that Camden is at risk of flooding because of the significant floods in 1975 and 2002 (Halcrow, 2011).

The lead local flood authority (LLFA) and local planning authority is the London Borough of Camden (LBC). The recommendations from the LBC Preliminary Flood Risk Assessment (PFRA) have been reviewed in undertaking this assessment. The LBC Local Flood risk Management Strategy (LFRMS) was approved in June 2013 (London Borough of Camden, 2013). LBC has also produced a strategic flood risk assessment (SFRA) in conjunction with a number of surrounding local planning authorities (Mouchel, 2008).

Review of these documents show that potential flooding risks in LBC are primarily from surface water flooding, when the intensity of rainfall can overwhelm sewers and drainage systems. There is also a small risk of groundwater flooding (which occurs when the water table rises to ground level); from inundation due to reservoir failure (e.g. Hampstead Ponds); or from overtopping the Regents Canal. The impact of basements on each of these types of flooding is considered in the surface flow and flooding scoping section of the BIA.

2.6.4 River or tidal flooding

Because the Site is elevated well above the flood plain of the River Thames at about +27mODmOD, it is shown as being outside Flood Zone as defined on the Environment Agency Flood Zone maps (Environment Agency).

2.6.5 Surface water flooding

Camden’s flood risk management strategy (London Borough of Camden, 2013) describes how, in highly developed areas, such as London, surface water flooding occurs when intense rainfall is unable to soak into the ground or enter drainage systems, because of failure of the pipes or where drainage capacity has been exceeded. It concludes that the risk of surface water flooding in Camden South is much lower than in the north of the borough.

In addition the Site is not located near any of the areas that were flooded in 1975 or 2002 or identified as areas with the potential to be at risk of surface flooding as shown in Figure 5.1 of the LFRMS (London Borough of Camden, 2013).

2.6.6 Sewer flooding

Most of Camden is served by combined sewers which receive foul water, water from roofs, hard standing and sometimes highways. Many of these combined sewers were designed by Sir Joseph Bazalgette in the 1860's. During periods of heavy rain the sewers fill up and can overflow. Sewer flooding events are a London wide issue. Thames Water holds details of incidents of sewer flooding for individual properties in a Sewer Flood database. This database has not been interrogated as part of this assessment but it is understood that very few properties have experienced flooding from sewers in the W1T post code area.

Sewer systems in the Borough are often very old. These older sewers were sometimes designed to convey storms of relatively low return periods, typically a 1 in 10 year rainfall event. Even new surface water systems are designed to a minimum standard of 1 in 30 years, much less than the 1 in 100 year standard of protection expected from fluvial flooding. As a result sewer flooding events, where they occur, can often be frequent, although the scale of impact is generally smaller than those associated with fluvial flooding.

The London Regional Flood Risk Appraisal (2009) advises that foul sewer flooding is most likely to occur where properties are connected to the sewer system at a level below the hydraulic level of the sewage flow, which in general are often basement flats or premises in low lying areas.

2.6.7 Groundwater flooding

Groundwater flooding most commonly occurs in low lying areas which are underlain by permeable rock (aquifers) or may be localised sands or river gravels in valley bottoms underlain by less permeable rocks. Flooding occurs when the local water table rises up from the permeable rocks to the ground surface, flooding low lying areas or occurring as intermittent springs. Flooding is most likely to occur after prolonged periods of rainfall when a greater volume of rain will percolate into the ground, causing the groundwater table to rise above its usual level.

The Site is underlain by the London Clay formation which fully confines the underlying Chalk aquifer at depth and therefore the risk of groundwater flooding is considered negligible.

2.6.8 Flooding from canals, water features and water mains

In Camden this type of flooding is most likely to result from burst water mains or from infrastructure failure in an artificial watercourse or water bodies, i.e. canals or other water features. Many of the water mains in the area date from Victorian times. Detailed records of the exact locations and incidents are held by Thames Water.

The site is not close to any canals or other water features.

2.7 Hydrogeology

London is underlain by two aquifers; the deep Chalk aquifer which is present across the entire London Basin and a shallow superficial aquifer comprising of the River Terrace Deposits, which is variably distributed across London.

The two aquifers are hydraulically separated by the London Clay and lower permeability parts of the Lambeth Group. The Lower Aquifer is predominantly comprised by the Chalk Formation but also includes the overlying Thanet Sand Formation, and permeable parts of the lower Lambeth Group (where present). The Lower Aquifer is classified as a principle Aquifer. The Lower Aquifer is located at significant depth (approximately 43m at the Site based on BGS well records).

Information obtained from the Environment Agency annual report in 2015 indicates that that the piezometric level in the Lower Aquifer is at approximately -35mOD (EA, 2015). The proposed basement excavation is restricted to the shallow sub-surface and will not impact upon the Lower Aquifer. The remainder of this section will focus on the shallow superficial deposits near the surface.

Previous desk study and ground investigations in the vicinity of the Site have indicated that the River Terrace Deposits (RTDs) are approximately 3.5m thick at the Site. Groundwater levels associated with the RTDs are at approximately +22mOD, roughly the same elevation as the top of the unit. Groundwater level data indicates that there is very little gradient across the Site.

Groundwater data from nearby Sites indicates that hydraulic gradient is in roughly a north to south direction which is consistent with the regional hydrogeological conditions. Groundwater in the shallow aquifer in central London tends to flow toward the River Thames, following the general dip in the surface of the London Clay as well as the local topography.

Groundwater level at the nearby Tottenham Court Road Investigation was at approximately +24mOD decreasing to around +23mOD at the Fitzrovia Phase 2/3 located to the south west of the Site. However, the observed groundwater levels are not all from the same time period and may not be fully representative of current conditions.

3 The Amended Scheme

3.1 Description

The Site is to be re-developed into commercial office space. The approved redevelopment scheme includes demolition of the existing buildings and replacement with a new building. The amended scheme increases the amount of demolition proposed. The superstructure for all replacement buildings will be founded on new piles and includes between 6 and 10 floors plus a basement. Heights of 6 to 7 floors are situated around the edge of the building, with 8 to 9 floor heights over the majority of the site and the central core areas rise to 10 floors.

The existing basement which has a typical floor level of about +25.4mOD will be deepened as part of the amended scheme by approximately 2m to have a structural slab level of about +23.5mOD.

A section through the amended scheme between Charlotte St and Whitfield St is shown in Figure 12. A sketch illustrating the variations in storey height of the building is shown in Figure 13.

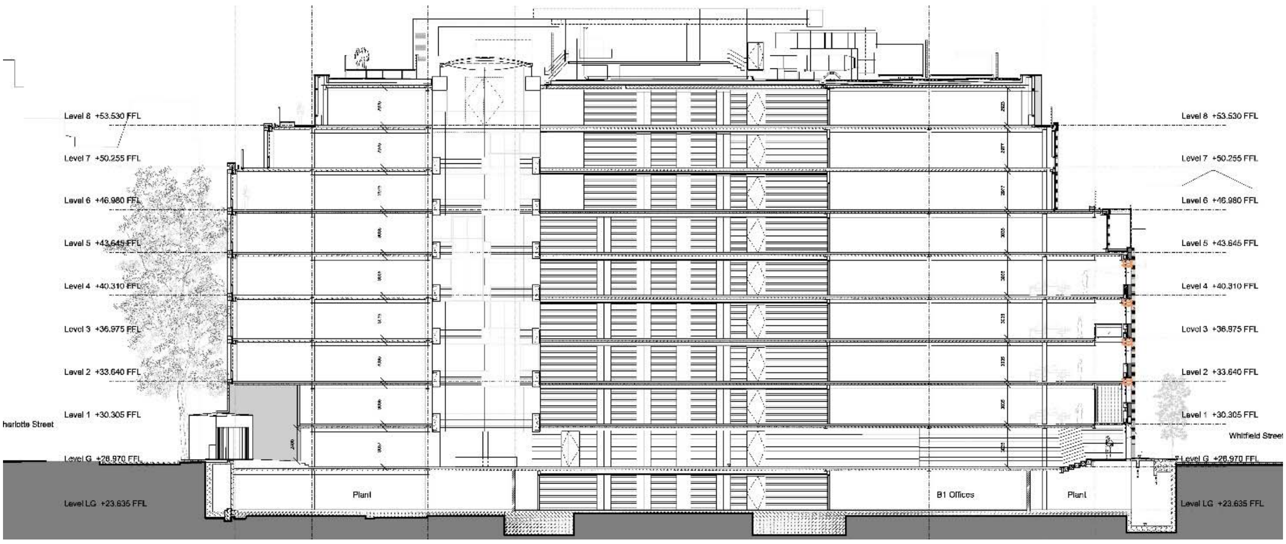


Figure 12 Section through amended scheme between Charlotte St and Whitfield St

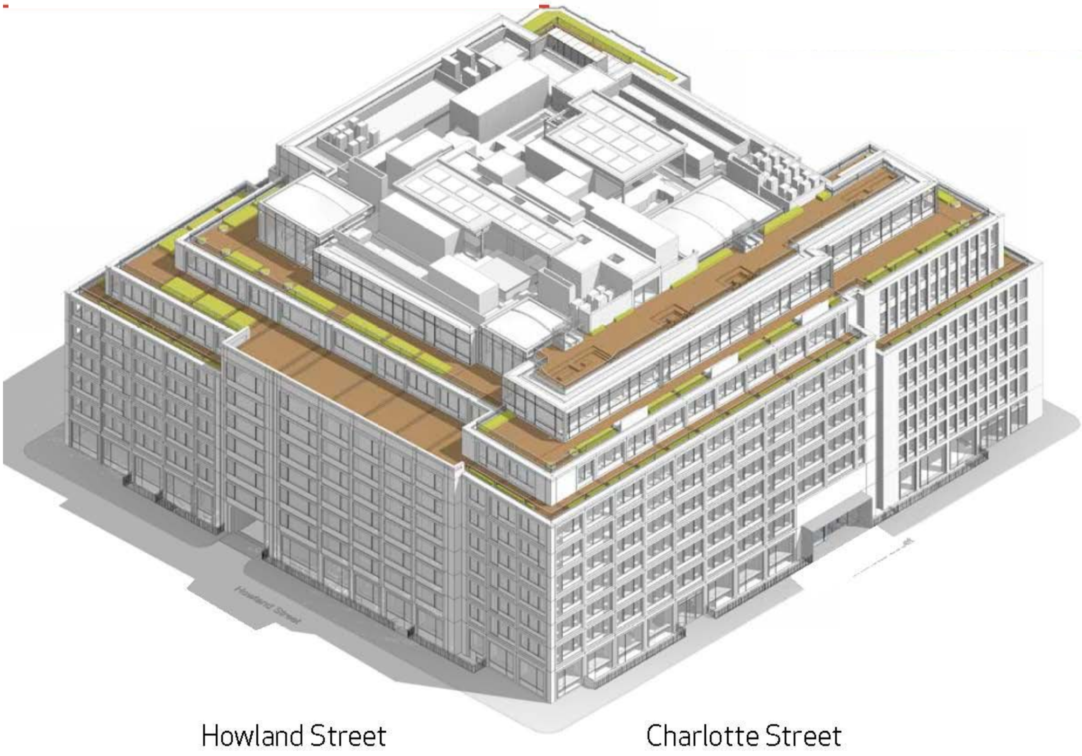


Figure 13 Sketch of outside of approved scheme, illustrating variations in numbers of storeys

The deeper basement will extend towards the lightwell wall in several areas as indicated in Figure 14 to ensure adequate routes for fire escape. There are 3 proposed configurations:

a) The existing lightwell depth and extent is largely maintained. The depth of basement deepening would be supported by a contiguous piled wall of at least 600mm diameter piles at 750mm centres. A new cast in situ wall will be constructed above, and connected into, the pile cap to maintain the existing lightwell ground retention.

b) Over short lengths of wall, the basement level needs to take the full plan extent of the existing lightwell. In this case the existing retaining wall would need to be replaced or piled through. Various options are being considered including piled solutions potentially utilising king posts and temporary supports or using underpinning techniques. The various options are being evaluated. The new wall will be propped throughout construction and in the long term.

c) The basement level takes up most of the plan extent of the existing lightwell, but there is space to locate a new retaining wall in front of the existing. In this case, for the depth of the basement deepening, a potential solution is for a contiguous mini piled wall constructed of 280mm diameter (cased section)/235mm diameter below casing at 400mm centres is proposed. A new cast in situ wall will be constructed above, and connected into, the pile cap to support the existing lightwell wall. The new wall will be propped throughout construction and in the long term.

Where the configuration of the basement retention system has not yet been decided, as shown in purple on Figure 14, the retaining wall will be similar to configurations a) or c) above and of sufficient stiffness to limit ground movements to within the predicted movements for those configurations.

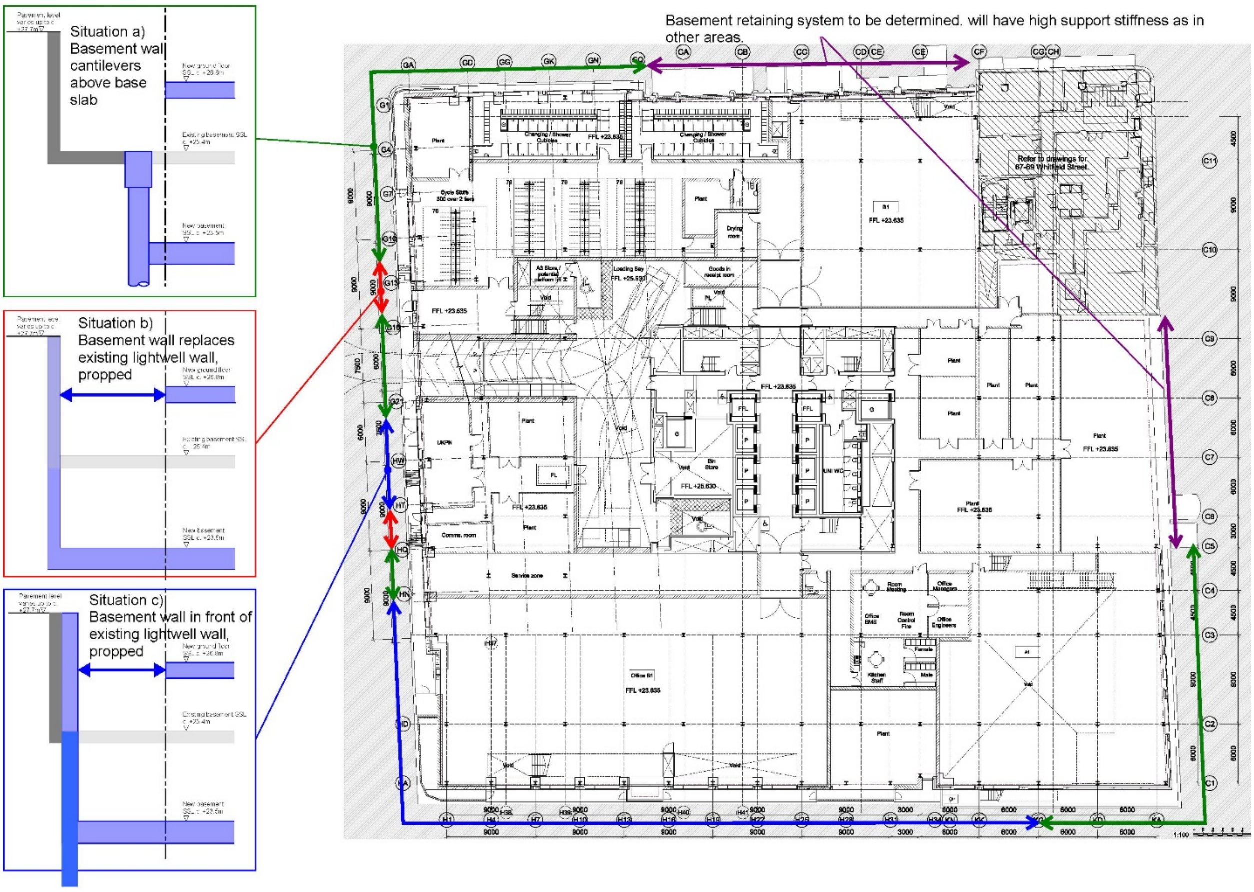


Figure 14 Diagram showing configurations of basement retaining wall around the site.

3.2 Construction methodology

The construction methodology is currently under discussion but final solutions will ensure that retaining wall movements and the impact on surrounding infrastructure and structures remain within those assessed in this BIA.

The potential methodology for constructing the 3 different configurations of basement retaining wall are described in the following figures.

The sequence has been developed with a stiff support system maintained throughout to limit ground movements and associated impact on nearby structures.

Supporting calculations have been carried out to establish the structural requirements for the retaining wall and associated ground movements due to construction. These are described in Section 9.

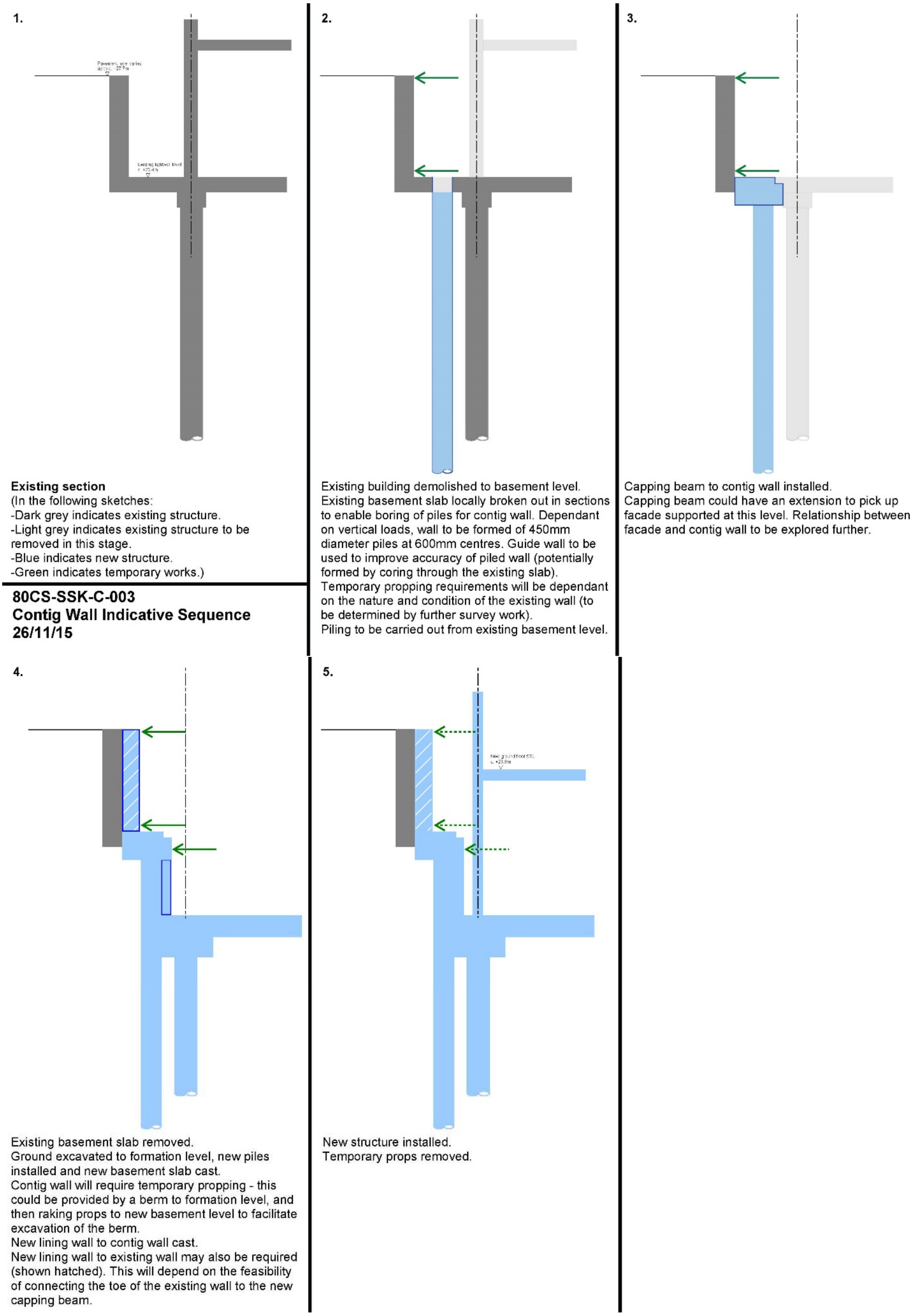


Figure 15 Retaining Wall Configuration a) Proposed construction sequence

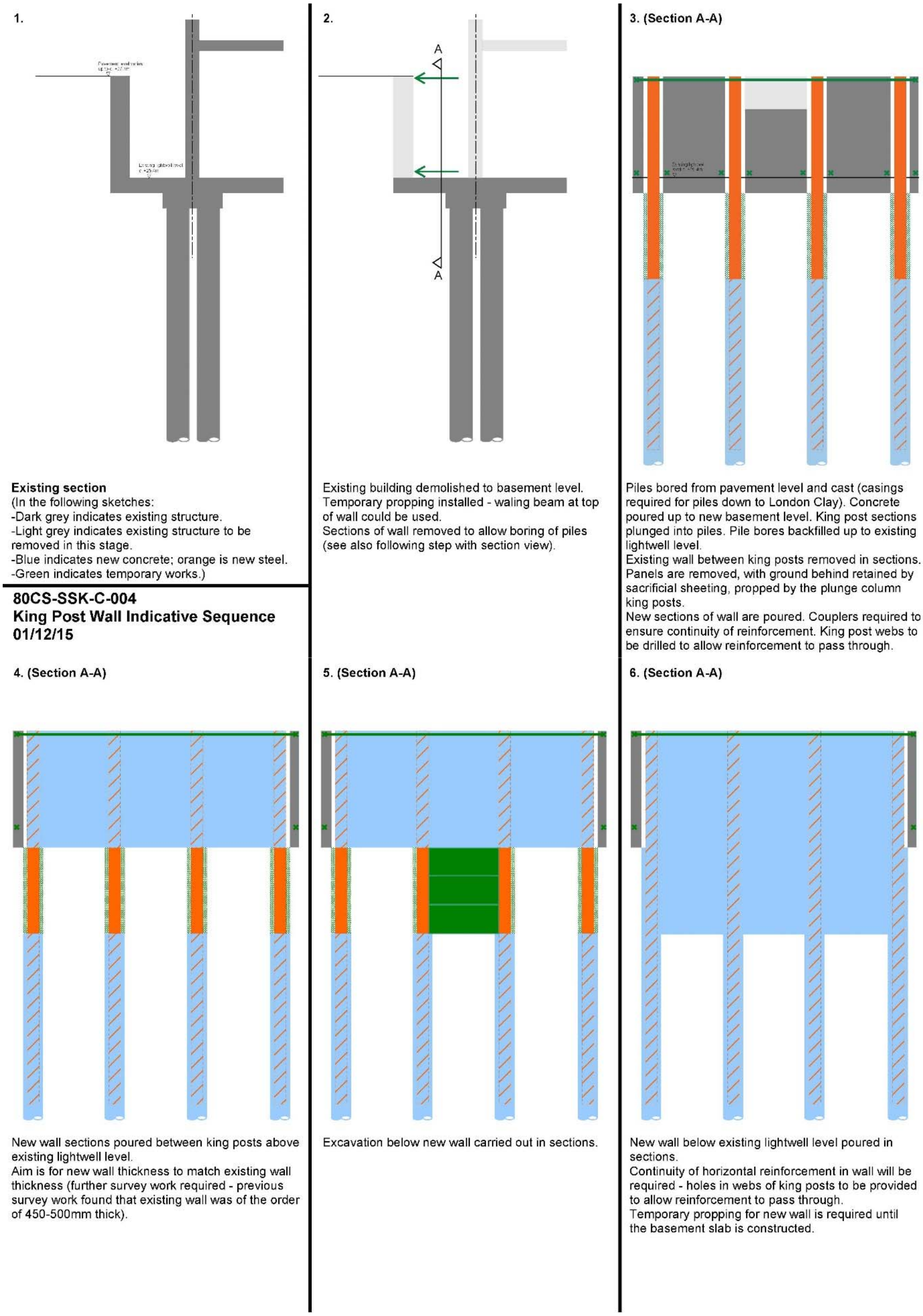


Figure 16 Retaining Wall Configuration b) Proposed construction sequence – part 1

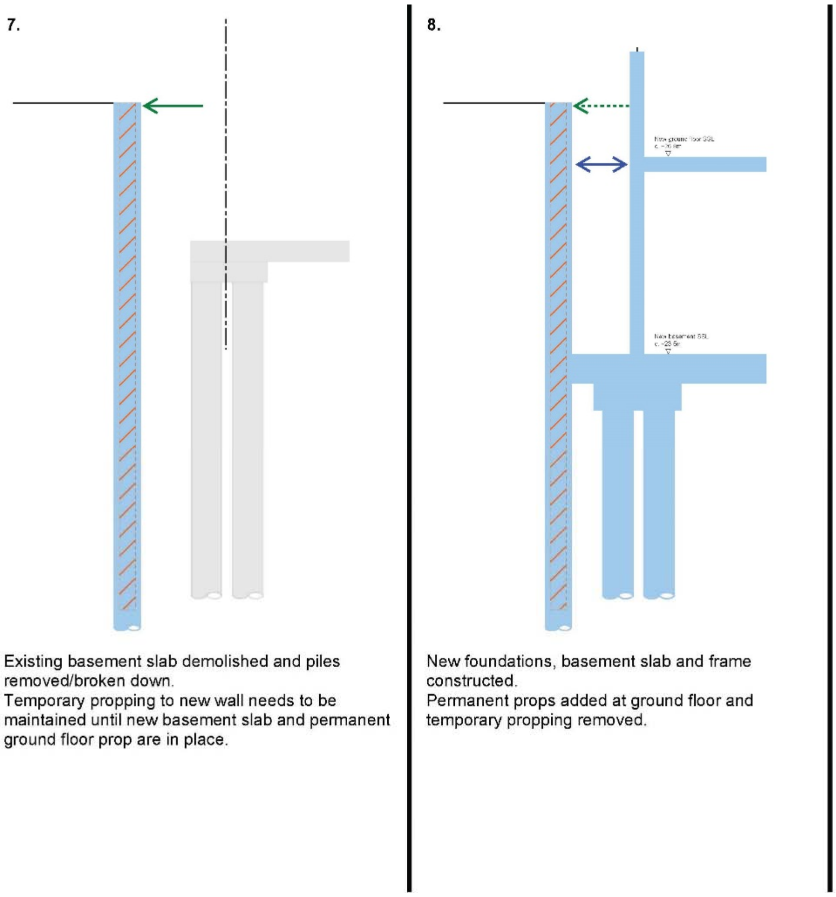


Figure 17 Retaining Wall Configuration b) Proposed construction sequence – part 2

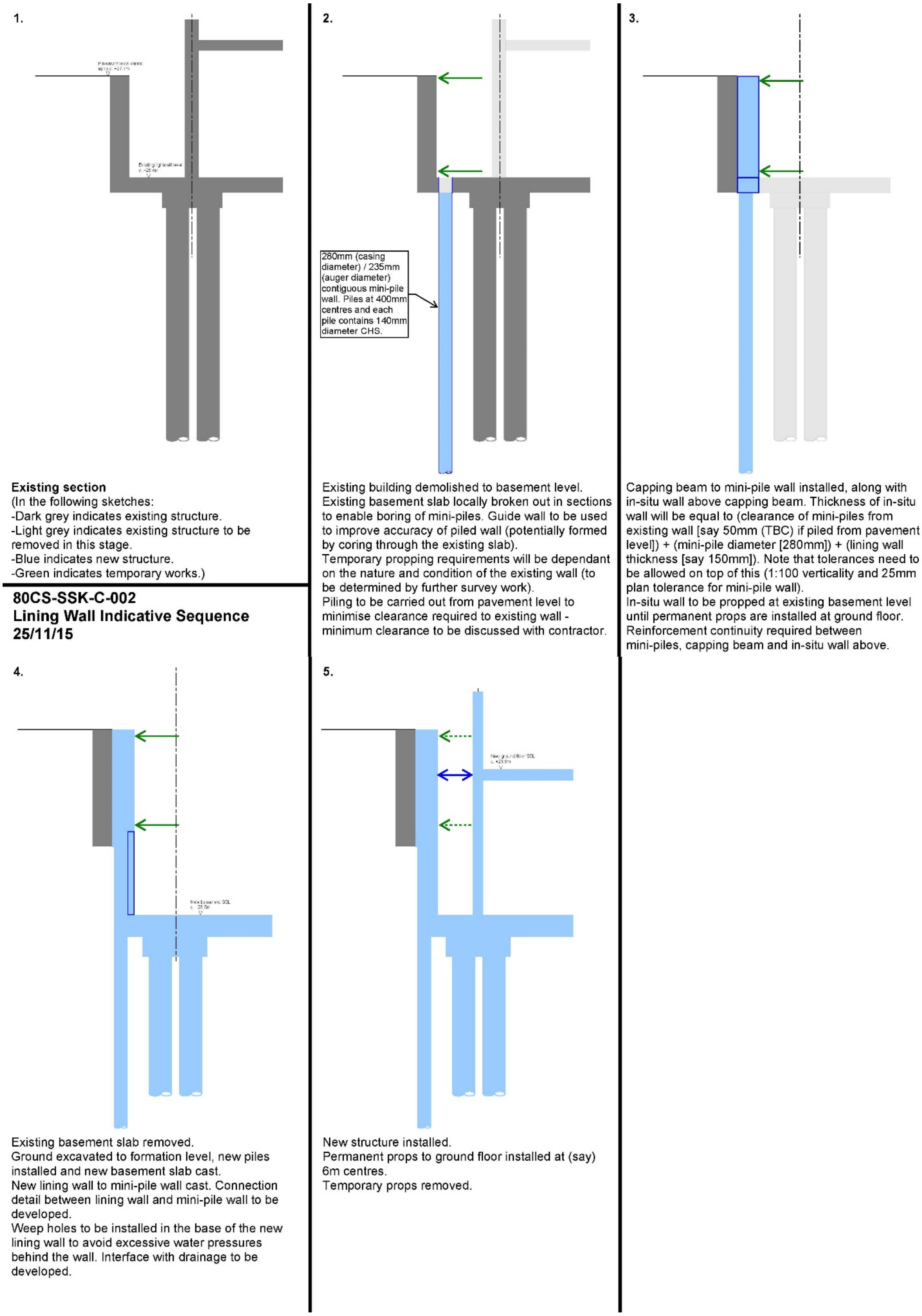


Figure 18 Retaining Wall Configuration c) Proposed construction sequence

4 Surface flow and flooding

The impact of the amended scheme on the surface water environment and need for flood risk assessment is considered here.

4.1 Stage 1: Initial screening

The first stage in assessing the impact of any proposed basement development is to recognise what issues are relevant to the proposed site and to identify the matters of concern which should be investigated further. This is done by using the screening flowchart and guidance found in Appendix E of the Arup guidance for Subterranean Development [3] and in the Camden Planning Guidance – Basements and Lightwells (CPG4) including Camden Development Policies DP27 – Basements and Lightwells [4].

4.1.1 Surface flow and flooding screening

No.	Screening Question	Impact	Source/Comment
1.	Is the site within the catchment of the pond chains on Hampstead Heath?	No	The Site lies well outside the Hampstead Heath surface water catchment area as defined by Figure 14 of LBC guidance [3] and there are no other equivalent sensitive water features in the vicinity of the site.
2.	As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?	No	Surface water mainly discharges to the local storm water sewer.
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	No	The majority of the existing site is hard standing. There will be a small portion which will be changed to soft landscaping
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous & long-term) of surface water being received by adjacent properties or downstream water-courses?	No	Currently much of the Site consists of hard-standing or roofed areas and all surface water flows are routed, at present to the Thames Water storm-water sewers. The approved scheme will infill the existing internal courtyard, but the courtyard is already covered with hard standing so that there will be no increase to the extent of impermeable surface area.

No.	Screening Question	Impact	Source/Comment
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream water courses?	No	The surface water quality will not be affected by the development. During construction a Construction Management Plan (CMP) will be followed to minimize the risk of excess runoff and contamination of surface water.
6	Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature?	No	The Site is not at risk of surface flooding.

4.2 Surface flow and flooding, matters to be carried forward

The following impacts have been identified during screening:

- The existing surface water drainage system will need to be modified as part of the amended scheme.

4.3 Stage 2 Scoping

The potential impacts which will need to be considered include:

- More detailed examination of net surface water flows and discharges from Site. This will require:
    - A more detailed description of the existing and proposed future drainage design,
    - Estimate of total area of hard surface/paved or roofed areas; and
    - Estimate of net runoff and consideration in detailed design of how net runoff will remain the same as at present, for example by incorporation of SUDS into the design.

The above issues will need to be covered in detailed design.

The detailed design of possible mitigation of surface water, surface water flow storage and other systems should be completed during the next design stage; however it will be necessary to demonstrate that the design discharge conditions are, as a minimum, like for like.

5 Subterranean (groundwater) flow

5.1 Stage 1: Initial screening

The impact of the amended scheme on groundwater flows and levels is considered here.

5.1.1 Subterranean (groundwater) flow screening flowchart

No.	Screening Question	Impact	Source/Comment
1a.	**Is the Site located directly above an aquifer?**	**Yes**	**The geotechnical desk study (Arup, 2010) and Geotechnical Interpretative Report (Arup, 2014) indicate that the Site is underlain by the River Terrace Gravels, a Secondary A aquifer which is considered to be relatively high permeability aquifer.**
1b.	**Will the proposed basement extend beneath the water table surface?**	**No***	**Groundwater levels at the Site are approximately +22mOD, which is roughly the same level as the top of the River Terrace Deposits. The basement extends to a maximum elevation of +23.5mOD with the base of the slab set approximately 0.5m lower.** ***Contiguous pile walls are planned to be installed around the majority of the development. These extend into the London Clay and will present a partial cut-off to the aquifer.**
2.	Is the Site within 100m of a watercourse, well (open/disused) or potential spring line?	No	The nearest major water feature is Regents Park Lake which is located approximately 500m to the north west of the Site. A 'lost river', a tributary of the former River Fleet is located approximately 250m to the north of the Site, although its exact location is unknown.
3	Is the Site within the catchment of the pond chains on Hampstead Heath?	No	The Site lies well outside the Hampstead Heath surface water catchment areas and there are no other equivalent sensitive water features in the vicinity of the Site.
4	Will the proposed basement development result in a change in the proportion of hard-surfaced/paved areas?	No	The majority of the existing Site is hard standing. There will be a small portion which will be changed to soft landscaping.

No.	Screening Question	Impact	Source/Comment
5	As part of the Site drainage, will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via soak-away and/or SUDS)?	No	Currently most of the Site consists of hard-standing or roofed areas and all surface water flows are routed to the Thames Water storm-water sewers. There will be no increase to the footprint of the existing building and the surface permeability is unlikely to be materially changed by the amended scheme. The volume of surface water discharged to the ground is not expected to be significantly altered by the amended scheme however a detailed examination of the net surface water flows and discharges from the Site will be undertaken as part of the detailed design.
6	Is the lowest point of the excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond or spring line?	No	There are no nearby surface water or springline features for the basement to be affected by.

5.2 Subterranean flow, matters to be carried forward

The following possible impacts on groundwater have been identified during screening:

- The Site is located above an aquifer and the proposed retaining structure (contiguous piles) will extend across its entire thickness. These structures have the potential to impact on groundwater flow though the aquifer and groundwater levels adjacent to the structure.

5.3 Stage 2 Scoping

The potential impacts which will need to be considered include:

- Consideration of any impacts of the proposed retaining wall structures acting as an impermeable, or partially permeable barrier to water flow. This is addressed in the impact assessment in Section 9 of this report.

6 Slope stability

6.1 Stage 1: Initial screening

6.1.1 Slope stability screening flowchart

No.	Screening Question	Impact	Source/Comment
1.	Does the existing site include slopes, natural or manmade, greater than 7^{o}? (approximately 1 in 8)	No	The land slopes very gently upwards to the north-west at a gradient of approximately 1:500
2.	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7^{o}? (approximately 1 in 8)	No	The extra basement dig of 2m depth is within the current basement footprint.
3.	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7^{o}? (approximately 1 in 8)	No	
4.	Is the site within a wider hillside setting in which the general slope is greater than 7^{o}? (approximately 1 in 8)	No	The site is not located within a wider hillside setting and there are no slopes adjacent to the property boundary.
5.	Is the London Clay the shallowest strata at the site?	No	London Clay is overlain by Terrace Gravels and Made Ground.
6.	**Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree protection zones where trees are to be retained? (Note that consent is required from LB Camden to undertake work to any tree/s protected by a Tree Protection Order or to tree/s in a Conservation Area if the tree is over certain dimensions).**	**Yes**	**No trees will be lost as a result of this proposal; as the proposal involves construction processes in close proximity to trees, tree protection measures and construction restrictions are recommended and should to be implemented in order to protect the trees and their existing growing environment.**
7.	Is there a history of seasonal shrink-swell subsidence in the local area (Claygate Beds), and/or evidence of such effects at the site?	No	We are not aware of the area having a history of shrink-swell subsidence. The effects of shrink-swell subsidence are not evident at the site.
8.	Is the site within 100m of a watercourse or a potential spring line?	No	Barton’s “The Lost Rivers of London” shows a possible tributary to the River Fleet originating about 220m to the north east of the site and flowing north

No.	Screening Question	Impact	Source/Comment
9.	**Is the site within an area of previously worked ground?**	**Yes**	**The 1746 Rocque Map shows that the site lies at the edge of an old quarry to the north. There is about 6m of Made Ground between ground level at about +28mOD and the Terrace Gravels at about +21.5 to +22mOD. However, the ground around the sides of the excavation will remain well supported throughout.**
10.	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	No	Formation level for the site will be at about +23mOD. The level of the upper aquifer is at about +22mOD.
11.	Is the site within 50m of the Hampstead Heath ponds?	No	-
12.	**Is the site within 5m of a highway or pedestrian right of way?**	**Yes**	**The site is surrounded by Charlotte Street, Howland Street, Whitfield St and Chitty Street.**
13.	**Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?**	**Yes**	**67-69 Whitfield St is directly adjacent to the basement of the amended scheme, although this building is also part of the Site.** **Other nearby properties are more than 10m from the proposed basement. The excavation of about 2.5m is not anticipated to cause any significant ground movement at the buildings across the roads. Nevertheless, potential ground movements related to deflection of basement retaining walls will be quantified and controlled.**
14.	Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?	No current tunnels	The site is partially within the safeguarded zone for Crossrail 2. The site is 6m in plan from a BT tunnel beneath Howland St. The site is over 40m from the LUL Northern Line tunnels.

6.2 Slope stability, matters to be carried forward

Potential impacts have been identified in the screening process and these must be evaluated and assessed. The issues are summarised below.

- Basement excavation and superstructure loading have the potential to cause ground movements in the surrounding infrastructure and buildings.

- As the proposal involves construction processes in close proximity to trees, tree protection measures and construction restrictions are recommended and should to be implemented in order to protect the trees and their existing growing environment.

6.3 Scoping

Ground movement assessments need to be carried out to consider impact on the surrounding infrastructure and buildings including:

- Public highways Charlotte Street, Howland Street, Whitfield St and Chitty Street
- TW mains and sewers beneath surrounding roads;
- Cast iron gas main beneath Howland St
- BT tunnel beneath Howland St
- 67-69 Whitifeld St; and
- Buildings across the roads from the site. Note that structures on the other side of Charlotte St and Howland St have been recently redeveloped and are understood to be on piles. Structures on the far side of Whitfield St are older and could be on shallow foundations. Along Chitty St, Asta House on the corner with Whitfield St is known to be on piles. Foundations for the other buildings across Chitty St are also not known and could be on shallow foundations.

The ground movements and their impacts are addressed in this report in Section 9.

Tree protection measures will be developed during detailed design and construction planning.

Note that structures more than 15m from site, which is three times the full depth of the new basement (of 4.7m), have not been considered as they are outside the zone of any measurable impact.

7 Geology and Ground Investigation

7.1 Published Geology

Records obtained from the British Geological Survey (Sheet 256 of the Geological Survey of Britain – Solid and Drift Edition) indicate the Site to be underlain by:

- River Terrace Deposits;
- London Clay;
- Lambeth Group;
- Thanet Sand; and
- Upper Chalk.

7.2 Investigation information available from nearby sites

Arup have specified and procured a number of ground investigations in the immediate vicinity of the Site (shown in Figure 19). The scope of works relating to each phase of investigation listed below:

Fitzrovia Phase 1, 2001

- Two cable percussion boreholes to approximate depths of 25m;
- Five window sample holes to approximate depths of 6m; and
- Geotechnical testing.

Fitzrovia Phases 2 and 3, 2005

- Two cable percussion boreholes to approximate depths of 25m;
- Three window sample holes to approximate depths of 5m;
- One machine excavated pit;
- Four hand excavated pits; and
- Geotechnical and contamination testing.

105 Tottenham Court Road, 2005

- Two cable percussion boreholes to approximate depths of 25m;
- One cable percussion borehole with rotary follow on to 45m;
- Six machine excavated trial pits;
- Four hand excavated pits; and
- Geotechnical and contamination testing.

Noho Square, 2007

- Nine cable percussion boreholes to maximum depth of 32m;
- Two cable percussion boreholes with rotary follow-on to maximum depth of 71m;
- 24 machine excavated pits; and
- Geotechnical and contamination testing

Project Glimmer, 2011

- 2 cable percussion borehole, to depths of 12m and 48m; and
- Geotechnical and contamination testing

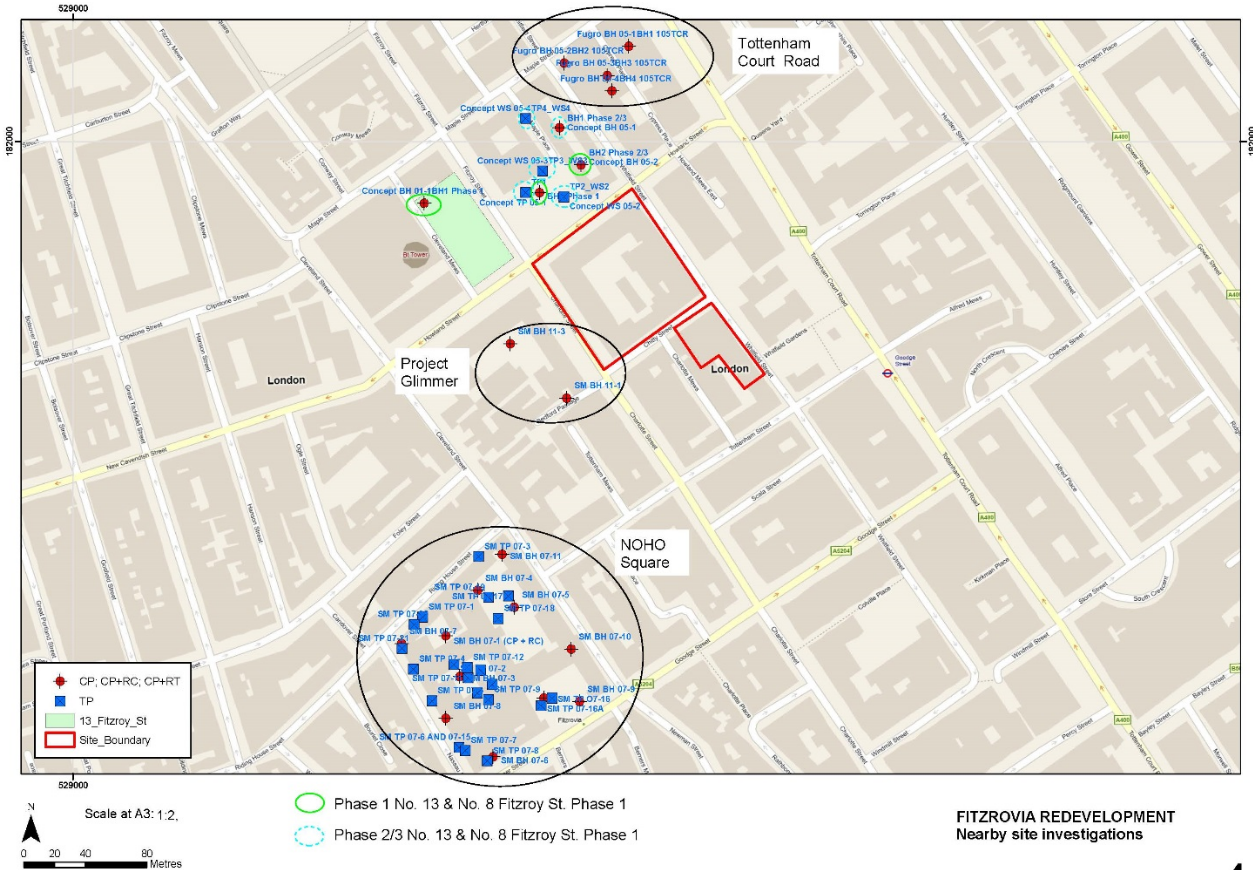


Figure 19 Nearby site investigations

7.3 Site specific GI

7.3.1 General

Two phase of ground investigation have been carried out at the site. The investigations also covered 67-69 Whitfield St and 65 Whitfield St which are not affected by this amendment scheme.

At the time of the investigations, reuse of the existing foundations was a potential option and the GI was targeted at both design parameters for new foundations and understanding the capacity of the existing foundations. A first phase of ground investigation was carried out between May 2012 and July 2012. The Site investigation was carried out while the buildings were occupied. This impacted on the scope of the site investigation, as not all of the intended investigation could be completed due to restrictions of working hours, noise, location etc. as the building was still occupied by tenants. Whilst the information obtained was sufficient for the design of the new foundations, capacities of existing foundations were generally lower than anticipated and in some instances the information was inconclusive. This led to a requirement for a second phase of GI to investigate some of the foundations further. In addition further investigation into contamination beneath the Site was also required as part of the second phase GI. The second Phase of GI took place between November 2012 and April 2013.

Factual reports on the Phase 1 and Phase 2 ground investigations were issued by Geotechnical Engineering in October 2012 and June 2013 respectively.

7.3.2 Site works

Figure 20 shows the completed site investigation locations in Phases 1 and 2 of the GI.

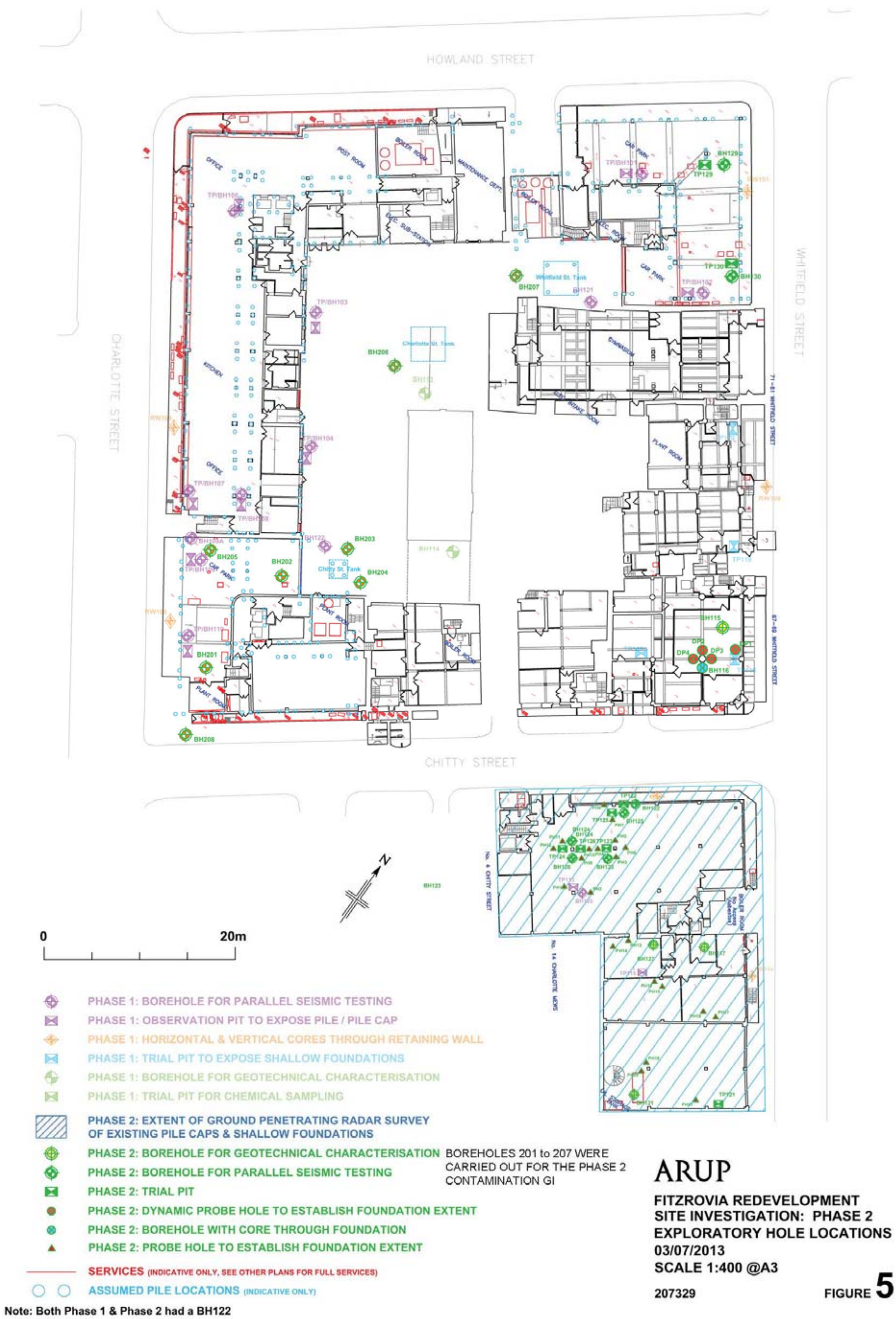


Figure 20 Exploratory hole locations in Phase 1 and Phase 2 GI

7.3.2.1 Phase 1

The onsite phase of the ground investigation undertaken by Geotechnical Engineering was conducted between the 12th May and July 11th 2012.

The site works comprised the drilling of boreholes using rotary and cable percussive techniques and the excavation of trial pits. Investigation of the existing structures involved concrete core holes and window breakouts. Table 2 shows the extent of works, both intended and completed. The disparity in planned and completed SI is due to the restrictions imposed as the site was still occupied by tenants for the duration of the investigation and there were complaints of noise at night.

Description	**No. of Planned SI**	**No of Completed SI**
Borehole to ~20m bgl next to existing pile (restricted headroom) – BH nos. 101, 102, 103, 104, 105, 106, 107, 108, 109, 109A, 110	9	11
Borehole to ~20m bgl next to existing pile (unrestricted headroom)	2	0
Borehole to ~30m bgl next to existing pile	1	0
Borehole to ~30m bgl (unrestricted headroom) – BH 114	1	1
Borehole to ~50m bgl (unrestricted headroom) – BH 113	1	1
Observation pits to investigate foundations. TP nos. 101, 102, 103, 104, 106, 107, 108, 109, 110, 113, 115, 116, 118, 119, 120,	20	15
Horizontal core through pile	15	12
Vertical core through pile	1	0
Trial pits (for contamination soil sampling only)	2	2
Investigation of retaining wall including, where possible, probing, horizontal core through retaining wall and window breakout to expose reinforcement – RW nos. 101, 104, 105, 109, 111, 114	14	6
Parallel seismic testing of exposed piles	12	11
No. combined gas & groundwater standpipes in the Made Ground or River Terrace Deposits	3	2
Standpipe to measure the water level in the Chalk	1	1
Contamination boreholes (completed with rotary rig) in the courtyard area – BH 121, BH122	2	2

Table 2 Phase 1 GI planned and completed site investigation

7.3.2.2 Phase 2

The Phase 2 GI consisted of the following:

- a ground penetration radar survey followed by small diameter drilled "probe holes" to investigate the dimensions of the foundations at No. 65 Whitfield Street;
- trial pits and boreholes to further investigate the geology and lengths of piles at 80 Charlotte Street and 65 Whitfield Street, with associated lateral cores through the piles;
- Dynamic probe holes, boreholes and a concrete core to further investigate the foundations at No.s 67-69 Whitfield Street; and
- additional boreholes to investigate further the potential presence of contaminated ground

Table 3 shows the extent of works, both intended and completed. A planned trial pit/borehole in the medical room and a borehole in the entrance ramp at 80 Charlotte Street could not be completed due to the presence of services.

Description	**No. of Planned SI**	**No of Completed SI**
Borehole to ~20m bgl next to existing pile (restricted headroom) – BH nos. 115, 116, 117, 121, 122, 123, 124, 125, 126, 127, 129, 130	14	12
Observation pits to investigate foundations. TP nos. 121, 122, 123, 124, 125, 126, 129, 130	9	8
Horizontal core through pile	9	7
Parallel seismic testing of exposed piles	9	8
Contamination boreholes – BH Nos. 201, 202, 203, 204, 205, 206, 207, 208	8	8
Dynamic probe holes and core through pad at Nos. 67-69 Whitfield St	4	4
Core through pad at Nos. 67-69 Whitfield St	1	1
Ground penetration radar survey	1	1
Probe holes	9 locations	9 locations

Table 3 Phase 2 GI planned and completed site investigation

7.3.2.3 Field Testing

Standard Penetration Tests (SPTs) were carried out in the cable percussion boreholes and samples were retrieved for laboratory testing.

Seismic parallel testing was carried out by Testconsult to determine the length of the existing piles.

7.3.2.4 Instrumentation and Monitoring

Gas and Groundwater monitoring standpipes were installed in boreholes BH114, BH113c, BH121- Phase 1, BH122 – Phase 1, BH121 – Phase 2 and BH127 to monitor the gas and groundwater levels of both the upper and deep aquifer.

Monitoring of the water levels was done during both Phases of fieldwork and several times after each phase of fieldwork was completed.

7.3.2.5 Laboratory Testing

A laboratory testing programme was carried out including tests for classification and shear strength as well as contamination tests. 74 No. unconsolidated undrained triaxial compression tests were carried out samples recovered from boreholes.

Chemical analysis of soil and water samples was carried out for Geotechnical Engineering by Chemtest.

The concrete testing of 23 No. cores of piles and retaining walls was carried out for Geotechnical Engineering by Sandberg Laboratories.

8 Ground Conditions

8.1 General Stratigraphy

The site specific investigation (Figure 20) shows the following stratigraphy (Table 4). A geological cross-section in the N-S direction adjacent to Charlotte Street is shown in Figure 21. Other geological cross sections taken in the N-S and E-W directions are shown in Appendix A Figures 6 to 9.

Stratum	Thickness at location investigated (m)	Top surface at location investigated (mOD)
Made Ground	0.6 to 6.5	+25.2 to +27.5
River Terrace Deposits	1.3 to 5.0	+19 to +23.9
London Clay	15.6 to 16.0	+18.1 to +20.6
Lambeth Group & Thanet Sands	21.7	+2.8 to +3.0
Chalk	unproven	-18.7

Table 4 site stratigraphy

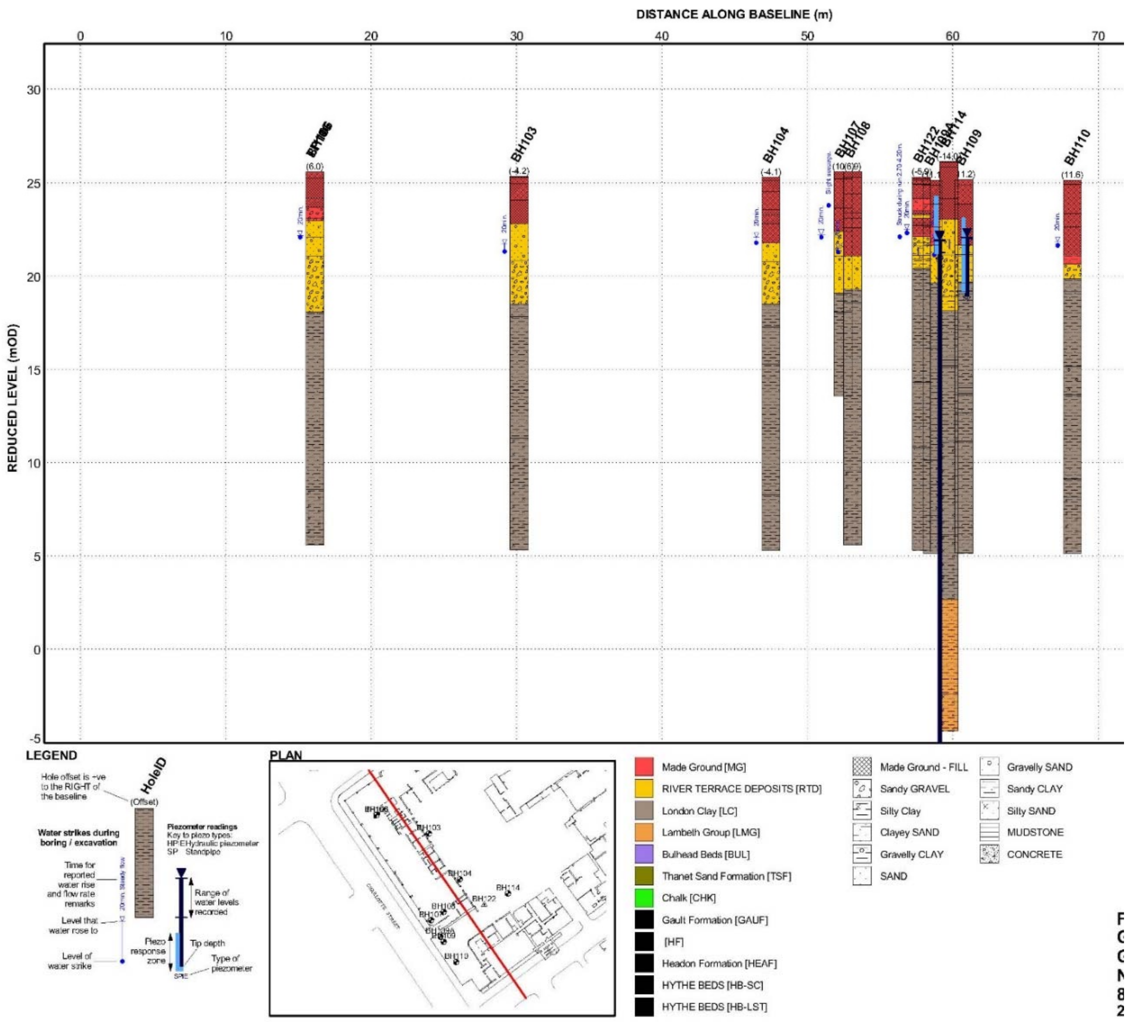


Figure 21 Geological cross-section taken N-S adjacent to Charlotte Street

8.2 Soil description and Parameters

For ease of reference, the borehole logs from the Phase 1 and Phase 2 GI are given in Appendix A.

The figures showing the measured GI data are given in Appendix A.

In the following, the description has focused on the GI at 80 Charlotte Street (rather than issues specific to 67-69 and 65 Whitfield St which are not part of this amended scheme).

8.2.1 Made Ground

8.2.1.1 Description

Exploratory holes were carried out in the basement of the existing buildings and courtyard. Concrete slabs, brick fill, tarmac surfacing are included in the category term Made Ground.

Beneath these structural materials, the logs description of the Made Ground varies. It is combination of fine and coarse material, containing brick, concrete cobbles, bone, metal, flint, charcoal, ceramic and shell fragments and as well as organic material.

The results of the tested material indicate the Made Ground to contain a high percentage of sand and gravel, as shown in the particle size distribution test results in Appendix A Figure 10.

In the majority of boreholes at 80 Charlotte Street there is a layer of soft clayey material or clay (typical SPT N value = 3 to 6), which ranges from 0.2m to 1.7m thick, and is typically 1.2m thick when present. Originally most of the logs described this material as soft clay, but some of the descriptions were revised to a clayey granular material following inspection of the grading curves.

8.2.1.2 Bulk Density

No tests have been undertaken to determine the bulk density of the Made Ground. A value of $19kN/m^3$ is recommended for design.

8.2.1.3 Standard Penetration Test

The SPT blowcounts (N) for the Made Ground range from 1 to 29, with the majority between 1 and 20. The N values are plotted on Appendix A Figure 11.

The layer of soft clayey material or clay found at most borehole locations at 80 Charlotte Street is tested in BH101, BH102, BH104, BH105, BH107, BH108, BH129 and BH130 and gives N values in the range 3 to 6.

8.2.1.4 Angle of Friction

Due to its variability and the presence of soft material, it is not recommended that the strength of the Made Ground is used for bearing capacity. In instances where the Made Ground acts on a structure, such as a retaining wall, then the following value of angle of friction is recommended for design:

$\phi' = 25^\circ$

This should be checked with reference to borehole information local to the structure concerned.

8.2.1.5 Stiffness

Due to the presence of very soft or loose materials it is not recommended that any stiffness of the Made Ground is used for support in bearing capacity.

In any design instance where a stiffness value is necessary, such as retaining wall design, then it is recommended that reference is made to the SPT and soil description information from the boreholes in the vicinity to deduce an SPT N value appropriate for that location and for the elevations in the ground concerned. The following relationships may be adopted, based on Stroud (1988).

- Vertical Young's Modulus, E_v'= 2N (MN/m^2)
- Horizontal Young's Modulus, E_h'= 2N (MN/m^2)

For a typical N value of 5 this gives Ev'= Eh'= 10MPa

8.2.2 River Terrace Deposits

8.2.2.1 Description

A typical description is a dark orangish brown slightly clayey very sandy angular to subrounded fine to coarse fine to coarse flint GRAVEL.

8.2.2.2 Classification and Consistency

The Terrace Gravels beneath 80 Charlotte Street are typically slightly clayey very sandy gravel. The Particle Size Distribution tests are shown in Appendix A Figure 12.

The plasticity results for the clayey and clay layers within the Terrace Gravels (clay layers were mainly found beneath 65 Whitfield Street where layers of soft or very soft clays were found) are shown in Appendix A Figure 13. Plasticity index varies between 8 and 32%.

8.2.2.3 Bulk Density

No tests have been undertaken to determine the bulk density of the Terrace Deposits. A value of 19kN/m^3 is recommended for design.

8.2.2.4 Standard Penetration Test

The standard penetration test N values range between and 1 and 30. The average value is about 13. The SPT N values are plotted on Appendix A Figure 14.

A moderately conservative N value of 10 is generally recommended for design. The scatter in test results is large and therefore this value may be varied locally with reference to nearby borehole information appropriate to the elevations in the ground concerned.

8.2.2.5 Angle of Friction

Based on recommended relationships presented in CIRIA C580:

φ'peak (°) = 30 + A + B + C = 33^0

φ'crit (°) = 30 + A + B = 33^0

Where:

A = 1 (Angularity: ‘subangular to subrounded’)

B = 2 (Grading: to take into account that the material is well graded, but also has a relatively high clay content)

C = 0 (SPT N: derived based on characteristic SPT N of 10)

8.2.2.6 Stiffness

Use of stiffness for the Terrace Gravels must be treated with caution because of the presence of soft and loose layers. In any design instance where a stiffness value is necessary then it is recommended that reference is made to the SPT and soil description information from the boreholes in the vicinity to deduce an SPT N value appropriate for that location and for the elevations in the ground concerned. The following relationships may be adopted, based on Stroud (1988).

- Vertical Young’s Modulus, E_v’= 2N (MN/m^2)
- Horizontal Young’s Modulus, E_h’= 2N (MN/m^2)

For a moderately N value of 10 this gives Ev’= Eh’= 20MPa

8.2.3 London Clay

8.2.3.1 Description

The weathered London Clay is typically stiff extremely closely fissured sandy clay dark orangish brown

The unweathered London Clay is typically described as very stiff extremely closely fissured dark grey slightly sandy CLAY with rare lenses of silt and frequent fine and medium selenite crystals.

In the vicinity of Site, at the base of the London Clay, a sand layer has been found to be intermittently present. During the site investigation at University College London, located west of Charlotte Street, the base of London Clay was found to not contain a sand layer. However, during piling works in the eastern part of the site, a sand layer was present at the base of the London Clay, at approximately +1mOD.

At Noho square, south west of the site, a sand layer is present, at approximately +0m. At Fitrovia Phase 3, north of the Site, the silty clay became "very sandy" at +5mOD and water seepage was noted below this level.

In BH101, at a level of +9mOD frequent selenite lenses, and rare coarse gravel sized pockets lead to a higher sand content.

8.2.3.2 Classification and Consistency

The particle size distribution test results indicate the deposit is typically silty CLAY, with variability of 0% to 42% sand. The results are shown on Appendix A Figure 15.

The material with 42% sand content is found in BH101, at a level of +8mOD; it is described in the borehole log as frequent selenite lenses (<3mm), and rare coarse gravel sized pockets of black carbonaceous material.

The sandier material, with a sand content of >30% is found in several boreholes (BH101, BH103 and BH110) at a level of +8mOD.

The London clay Plasticity ranges between 33% and 55% and Liquid Limit between 50% and 80%. This indicates clay of high to very high plasticity.The atterberg limits data and plasticity chart are shown in Appendix A Figures 16 and 17 respectively.

8.2.3.3 Bulk Density

A review of the sample densities reported for the triaxial (UU) tests on London Clay indicate that the bulk densities range between 18.6 kN/m^3 and 21.1kN/m^3. A value of 20kN/m^3 is recommended for design purposes.

8.2.3.4 Standard Penetration Tests

The variation of SPT blowcounts (N) for London Clay with elevation are shown on Appendix A Figure 18. The same results are shown in Appendix A Figure 19 plotted as depth below top of London Clay. Appendix A Figure 19 also shows the line back-calculated from the selected average undrained shear strength design line $c_u = 100 + 6.5z$ (see 4.2.3.5) using a correlation of $c_u = 5N$ which is typical for clays of high plasticity index:

$N = 20 + 1.3z$

Where z = depth below the design surface of the London Clay

The back-calculated line $N = 20 + 1.3z$ follows the trend of the data and is in reasonable agreement, although slightly higher than the average of the data (see 4.2.3.5).

8.2.3.5 Undrained Shear Strength

Quick unconsolidated undrained triaxial compression results are plotted against elevation and against depth below top of London Clay in Appendix A Figures 20 and 21 respectively.

Also plotted in Appendix A Figure 21 is the selected design line for average undrained shear strength:

$c_u = 100 + 6.5z$

Where z = depth below the design surface of the London Clay

This design line is slightly below an average through the undrained shear strength test results.

Appendix A Figures 22 and 23 show both the c_u values interpreted from the SPT N values using $c_u = 5N$ and measured in the triaxial tests plotted against elevation and against depth below the top of the London Clay respectively. In Appendix A Figure 23 the above design line is also shown. Based on all the data, the design line is considered generally appropriate.

<u>Design Recommendation</u>

For design of piles for vertical capacity and for heave/settlement calculations it is recommended that the following is adopted for the design line for the London Clay:

$c_u = 100 + 6.5z$

where z is the depth below the top of the London Clay.

8.2.3.6 Stiffness

For vertical loading for pile and foundation design:

Short term:	$E_{uv} = 400c_u$
Long term:	$E'_v = 320c_u$
Poisson's ratio:	$\nu = 0.2$ (drained, long term conditions)
	$\nu = 0.5$ (undrained, short term conditions)

These are derived from a database held by Arup of surveyed ground movements relating to unloading and loading of excavations provides an insight to ground stiffness profiles. The database comprises long term movements published by Hewitt (1989) and end of construction movements by Ho (1991).

For lateral loading in retaining wall design:

Short term:	$E_{uh} = 1000c_u$
Long term:	$E'_h = 800c_u$

These are based on back-analysis of case history data.

8.2.4 Lambeth Group and Thanet Sand Formation

It should be noted that the Lambeth Group and Thanet Sand Formations were only investigated in the Phase 1 GI.

8.2.4.1 Description

The Lambeth Group at this location consists of 10m of dark grey and mottled silty clay, with layers of clayey silt at depth. This overlies 8m of sand interbedded with very sandy clay. Only BH13 penetrated the sand layers below the upper Lambeth Group Clay.

Underlying the Lambeth Group is 4m of very dense greyish green sand (Thanet Sand).

8.2.4.2 Classification

The particle size distribution results (see Appendix A Figure 24) indicate high clay content at shallow depths, and higher silt and sand contents with increasing depth.

8.2.4.3 Bulk Density

The bulk density ranges betweent18kN/m^3 and 22kN/m^3 with an average of 21kN/m^3 recommended for design.

8.2.4.4 Undrained Shear Strength

From standard penetration testing:

Standard Penetration Tests (SPT) in the Lambeth Group measured ‘N’ values range between 32 and 430, with a typical value of 50, shown on Appendix A Figure 25.

To derive the undrained shear strength of the cohesive Lambeth Group, correlations with SPT ‘N’ blow counts suggest that cu = 4.5N be used for clays of intermediate to high plasticity, which would give c_u = 225kPa.

From undrained Shear Strength (Laboratory Testing):

Quick unconsolidated undrained triaxial compression results are plotted in Appendix A Figure 26. A typical value for the undrained shear strength from triaxial testing is c_u = 200kPa.

A shear strength value of 200kPa is recommended.

8.2.4.5 Stiffness

For the purpose of vertical movement assessment, the same stiffness correlations as used for London Clay are considered appropriate for the upper clayey part of the Lambeth Group:

End of construction:	$E_{uv} = 400.c_u$
	$E_{uv} = 400 \times 200\text{kPa} = 80\text{MPa}$
Long term:	$E'_v = 320.c_u$
	$E'_v = 320 \times 200\text{kPa} = 64\text{MPa}$

Poisson's ratio	ν = 0.2 (drained long term conditions)
	ν = 0.5 (undrained short term conditions)

Stroud's relationship of SPT N and Young's Modulus (1989) is considered appropriate for the Lambeth Group sand below -8mOD. Taking E′v = 2N, for a conservative value of SPT of N = 100 this gives:

E'_v = 200MPa

8.3 Groundwater

The site is underlain by two distinct aquifers that are confined by the London Clay/Lambeth Clay. These relatively impermeable strata act as an aquiclude which separates and impedes the groundwater table in the deep aquifer coming into hydraulic equilibrium with the shallow aquifer. Perched water tables may also exist in the Made Ground occurring at surface.

Measurements of the groundwater levels were taken during July and August, 2012 for the Phase 1 GI (BH122, Bh114)and for Phase 2 GI (BH121, BH127 and BH207) during May 2013.

8.3.1 Shallow Aquifer

The monitoring results of the shallow aquifer are shown in Table 5. A design ground water level is taken as +22mO.

Groundwater levels from previous site investigations in the vicinity of the site are shown in Table 6. The groundwater monitoring data is from locations to the north of the site. The direction of ground water flow is thought to be towards the south in this area.

Borehole	Depth of Water (bgl)	Water Levels (mOD)
BH122	3.41 to 4.05	+21.9 to +21.3
BH114	4.05 to 4.16	+22.0 to +22.1
BH121	5.45	+22.1
BH127	2.36 to 2.40	+22.09 to +22.05
BH207	4.42 to 4.45	+22.18 to +22.15

Table 5 Shallow aquifer monitoring results

Site	Borehole	Water Levels (mOD)	Date of Measurements
105 Tottenham Court Road	BH1	23.8	Jan – Mar 2005
	BH2	24.1	Feb – Mar 2005
	BH4	23.9	Feb – Mar 2005
Fitzrovia Phase 1	BH2	22.3	Jan-2005
Fitzrovia Phases 2&3	BH1	23.9	Jul – Aug 2005
	BH2	23	Jul – Aug 2005
Fitzrovia Redevelopment	BH114	22.1	Jul-2012
	BH122	21.9	Jul-2012
University College London	SM BH3	22	Jul-2011

Table 6 Shallow aquifer monitoring for previous SI's in the vicinity of the Site.

8.3.2 Deep Aquifer

The deep aquifer is located within the Upper Chalk bedrock and Thanet Sand. It is classed as a major aquifer by the Environment Agency and constitutes the principal aquifer for the Thames region.

The environment agency continually monitors the water levels in the deep aquifer. From January 2000 to January 2010, the water level decreased by approximately 5m at this location (Environment Agency, 2010).

Borehole	Depth of Water (bgl)	Water Levels (mOD)
BH113	48.4	-22.0

Table 7 Deep aquifer monitoring results during July – August 2012

8.4 Summary of Ground Parameters for Geotechnical Design

Stratum	Design Level for Top of Stratum (mOD)	γ_b (kN/m^3)	ϕ' (deg)	c_u (kPa)	E'_v (MPa)	E_u (MPa)	E'h (MPa) Retaining wall	E_{uh} (MPa) Retaining Wall
Made Ground		18	25	-	10	-	10	-
River Terrace Deposits	+21.5	19	33	-	20	-	20	-
London Clay	+18.5	20	24	100+6.5z, where z is depth below top of London Clay	320c_u	400c_u	800c_u	1000c_u
Lambeth Group - Clay	+3.0	21	Not used in design	200	64	80	Not used in design	Not used in design
Lambeth Group - Sand	-8.0	21	Not used in design	-	200	-	Not used in design	Not used in design

Table 8 Summary of ground parameters for geotechnical design.

9 Stage 4: Impact Assessment

The key issues highlighted in Sections 5.2 and 6.2 for which the impacts are considered here are:

- the basement piles acting as an impermeable barrier to potential water flow ; and
- the ground movements and their impacts on surrounding infrastructure and buildings.

A groundwater flow assessment in Section 9.1 addresses the former and ground movement and impact assessments in Sections 9.2 and 9.3 address the latter.

9.1 Groundwater flow assessment

Groundwater in the upper aquifer beneath the Site is at approximately +22mOD. The basement slab is expected to be approximately 1m above this level and therefore will not directly affect the flow of groundwater in the aquifer. However, the retaining structures at the Site are proposed to extend through the River Terrace Deposits into the London Clay beneath.

Details of the proposed retaining wall design are presented in Section 3. Two different contiguous piled wall designs are planned at different locations around the amended scheme. These are 280mm piles at 400mm centres (120mm gap between piles) and 600mm diameter piles at 750mm centres. The latter has been taken as 750mm piles at 900mm centres (150mm gap between piles) for the purposes of this assessment, which is a worse case for presenting a barrier to groundwater flow.

Contiguous piles are designed to allow groundwater movement between the gaps, however their installation will inevitably lead to a reduction in permeability in a zone surrounding the building. This is because the relatively permeable River Terrace Gravels will be replaced by low permeability concrete structures.

An assessment of the reduction in permeability due to the construction of the piles was undertaken based on the pile diameter and pile centres. The results are presented in Table 9.

Pile size (mm)	Pile centre (mm)	% gap of total	RTD hydraulic conductivity (m/s)	Pile hydraulic conductivity (m/s)	Effective hydraulic conductivity along line of piles
240	400	29	1×10^{-4}	1×10^{-8}	3×10^{-5}
750	900	16	1×10^{-4}	1×10^{-8}	2×10^{-5}
Notes The effective hydraulic conductivity will be restricted to a zone in close proximity to the pile wall only Hydraulic conductivity of the RTD has been assumed to be toward the lower end of the typical range due to the observed presence of clay material during ground investigation					

Table 9 Estimates of reduced hydraulic conductivity due to installation of contiguous pile wall.

Analysis of groundwater changes due to the construction of the contiguous piled walls at the Site was undertaken using SEEP/W, a 2D finite element modelling package. 2D numerical models, such as SEEP/W, provide simplified evaluations of hydrogeological conditions. The model provides a reasonable prediction of the scale of potential groundwater level changes based on the input parameters, which are a simplified version of actual conditions.

Groundwater levels in the aquifer are expected to be at or very close to the top of the unit, and confined by the overlying made ground. The 2D model was constructed in plan view mode allowing the impact of the contiguous piles to be addressed.

The model was set up as a 400m by 400m grid aligned north to south. The model mesh had a global element size of 2.5m except at interface elements where it was reduced to 1m (to improve the simulation accuracy). A single horizontal layer was used to represent the River Terrace Deposits, with the top at +23mOD at the north of the model and +20mOD along the south of the model in order to match the head boundaries. An isotropic permeability of 1 x10^{-4} m/s was used for the aquifer material.

A constant head was set along the north and south boundaries of the model. The northern boundary was +23mOD and the southern boundary was +20mOD. The east and west boundaries were set as no flow boundaries. The model set up and baseline contours are presented in Figure 22.

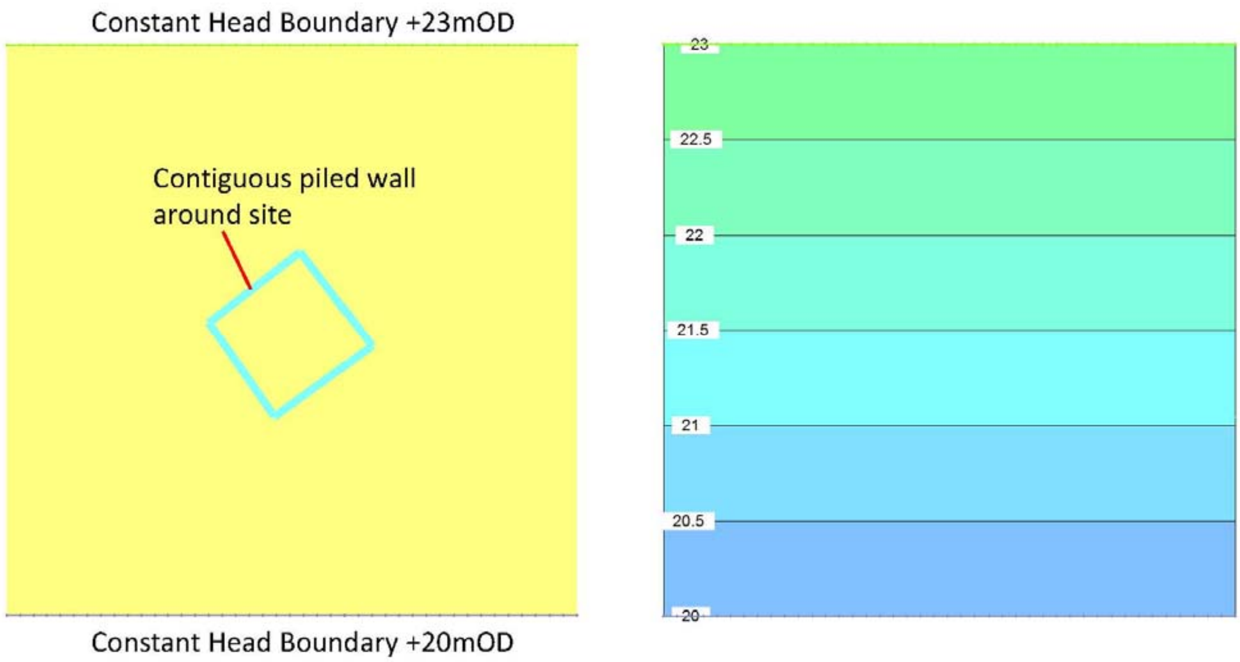


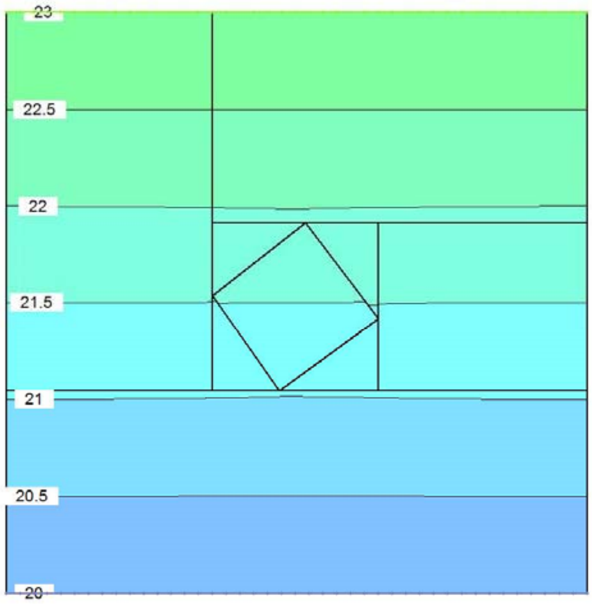
Figure 22 Seep/W numerical Model setup & Baseline head distribution

The contiguous piles will act as a barrier, with groundwater mounding against the up-gradient side and lowering on the down gradient side.

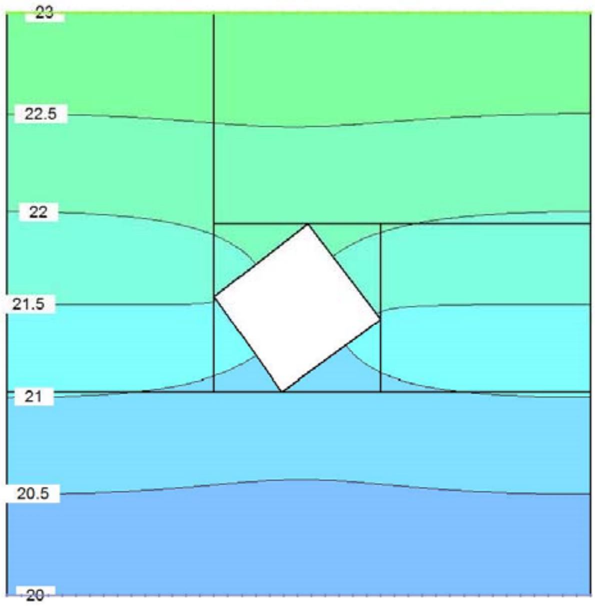
Scenario 1 was modelled under two separate conditions, one assuming the 240mm diameter piles surrounding the building and one assuming the 750mm diameter piles surrounding the building. Scenario 2 is provided as a worst case observation and is not expected to represent the conditions that will prevail at the Site since the bulk of the aquifer beneath the Site will remain unaffected. The intent of Scenario 2 is provide context for the assessed impacts due to the installation of the proposed piles.

Figure 22 shows the model groundwater contours prior to any development. Figure 23 shows the contours from scenarios 1 and 2 described above. The results indicate that the contiguous pile wall will have negligible impact on groundwater levels surrounding the Site. Under the larger pile diameter scenario, groundwater levels are expected to increase by approximately 2cm upgradient of the wall and 2cm downstream of the wall. The impact on groundwater levels is also restricted to a small area surrounding the piled wall. This compares with a 32cm change upgradient and downgradient associated with the full cut-off scenario.

The small changes in groundwater level predicted by the modelling are negligible and are expected to be within the normal range of seasonal fluctuation in the aquifer. The construction of the contiguous pile walls around the development are therefore expected to have no impact on adjacent structures or basements surrounding the Site.



Scenario 1 – 750mm piles at 900mm centres



Scenario 2 – Complete cut-off

Figure 23 Numerical model results

9.2 Ground movement assessment

9.2.1 Retaining wall analysis

Preliminary designs of retaining walls were carried out for the Site in order to understand potential wall movements and impact on adjacent structures and infrastructure. The Oasys software for retaining walls Frew was used to model 2D sections of the retaining wall on the Site.

Details of the Frew analysis including assumed excavation and propping sequence are given in Appendix B.

Three sets of analyses have been carried out to understand SLS deflections and ULS structural effects for two configurations of basement retaining wall described as cases a) and c) in Section 3. These are SLS and EC7 DA1C1 and DA1C2 analyses. For case a), a contiguous piled wall with 450mm diameter piles at 600mm centres has been analysed, with a 400mm wide cast insitu retaining wall above existing basement level (+25.4mOD). For case c), a contiguous piled wall with 600mm diameter piles at 750mm centres has been analysed, also with a 400mm wide cast insitu retaining wall above existing basement level (+25.4mOD).

Other sections of the proposed pile walls will be analysed at detailed design stage when the final scheme is developed.

Table 10 Contiguous piled walls analysed in FREW

Wall type	Pile spacing (m)	Toe level (mOD)	Maximum excavation level (mOD)
280mm diameter cased length/ 235mm diameter below casing	0.4	+16.3	+23.0
600mm diameter	0.75	+16.3	+23.0

9.2.1.1 Design of walls for bending

The results of the FREW analyses have been used to understand the structural requirements for the piled walls.

Maximum allowable bending moments in the walls have been assessed considering permissible levels of reinforcement (in accordance with BS EN 1536, 2010).

The comparison between maximum ultimate limit state (ULS) bending moment from FREW and ULS bending moment capacity of the walls is given in Table 11.

Table 11 Summary of calculated bending moments and indicative reinforcement

Case	Maximum ULS bending moment from FREW kNm/pile	Indicative pile reinforcement requirement
280mm diameter cased length/ 235mm diameter below casing	27	Central bar plus 139.7mm OD CHS (API tube)
600mm diameter piles	128	tbc

9.2.1.2 Wall Deflections

The SLS deflections of the 280mm diameter and 600mm diameter piled walls are highest when the wall relaxes in the long term and are shown in Figure 24 and Figure 25 respectively.

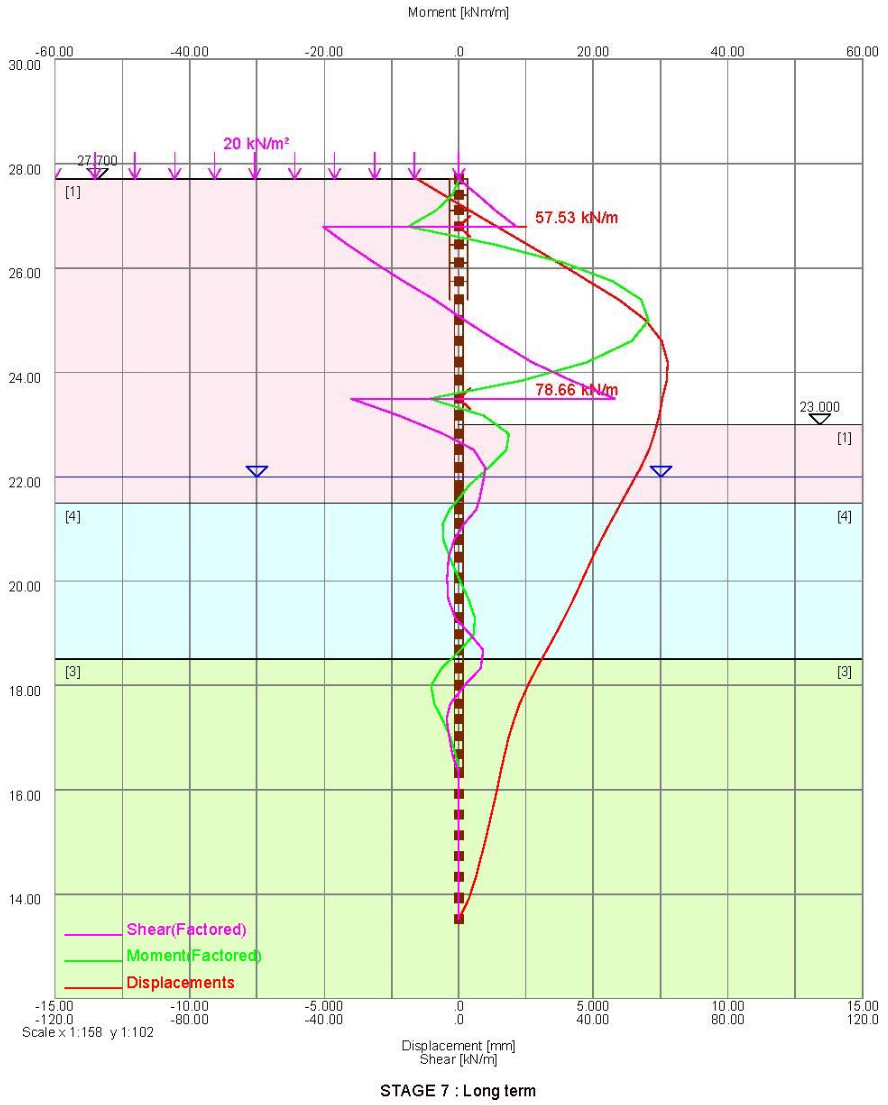


Figure 24 Long term deflections of 280mm diameter contiguous piled wall (shows also bending moment and shear force).

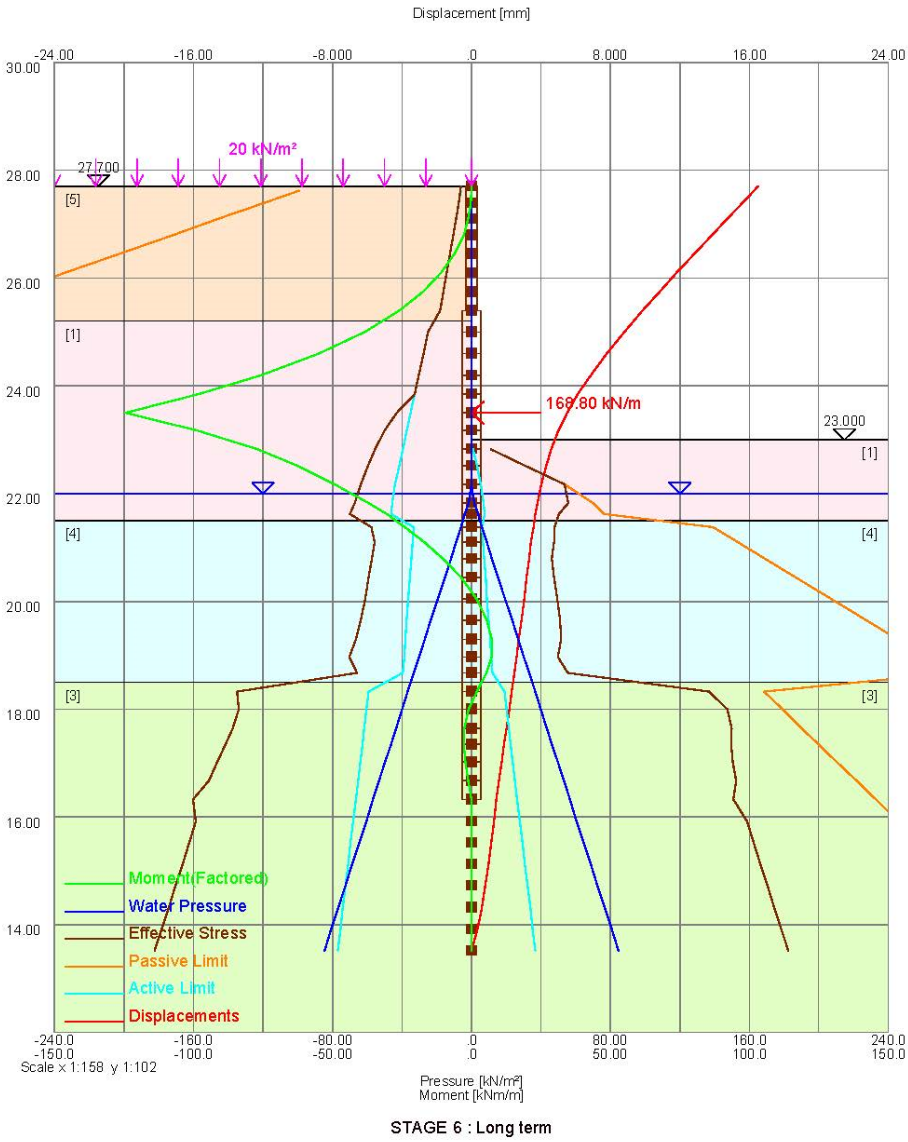


Figure 25 Long term deflections of 600mm diameter contiguous piled wall (shows also bending moment and shear force).

The maximum deflections of the walls calculated in FREW are given in Table 12.

Table 12 Maximum wall deflection from FREW

Case	**Maximum wall deflection from FREW (mm)**
280mm diameter cased length/ 235mm diameter below casing	8.0
600mm diameter piles	16.5

The deflections from the FREW analyses have also been used in the estimate of ground movements. These are discussed in Section 9.

9.2.1.3 Requirements for temporary propping

The walls have been analysed using reasonable prop levels and stiffness for the temporary propping. The number of temporary propping levels required and the levels assumed is given in Table 13. Precise prop levels and stiffness may be adjusted during detailed design to optimise the design.

Table 13 Details of temporary propping to piled retaining walls

Wall type	**No of temporary props**	**Assumed prop levels (mOD)**	**Assumed prop stiffness (kN/m/m)**
280mm diameter cased length/ 235mm diameter below casing, with 400mm insitu wall above	2	+27 +25	50,000
600mm diameter piles with 400mm insitu wall above	2	+27 +25	50,000

9.3 Ground movements behind walls

Ground movement predictions have been made in order to assess the potential impact to the adjacent infrastructure and structures. These include:

- Public highways Charlotte Street, Howland Street, Whitfield St and Chitty Street
- TW mains and sewers beneath surrounding roads;
- Cast iron gas main beneath Howland St
- BT tunnel beneath Howland St
- 67-69 Whitifeld St; and
- Buildings across the roads from the site.

9.3.1 Near surface ground movements due to wall installation and excavation

CIRIA report C580 'Embedded retaining walls guidance for economic design' gives empirical profiles of ground movements behind retaining walls due to wall installation and excavation in front. These profiles are based on numerous case histories and are widely adopted in the prediction of ground movements behind retaining walls.

The guidance has been used to consider ground movements affecting infrastructure and building foundations located within the depth of the basement. These include:

- Public highways Charlotte Street, Howland Street, Whitfield St and Chitty Street
- TW mains and sewers beneath surrounding roads;
- Cast iron gas main beneath Howland St
- Older buildings which are across the road from the site and potentially on shallow foundations, including buildings on Whitfield St and Chitty St (with the exception of Asta House on the corner with Whitfield St).

9.3.1.1 Vertical displacements

Settlement due to wall installation has been considered. Based on data presented in CIRIA C580, the calculation of vertical displacements has been calculated as 0.02% times the length of the pile (see CIRIA C580).

Following the guidance of CIRIA C580, profiles of vertical ground movement behind the piled retaining walls due to excavation have been calculated in accordance with Figure 26 using the calculations of wall movement from the FREW model. The profiles have taken the highest wall movements considering both during construction and in the long term. These profiles have been used to consider the impact on neighbouring structures adjacent to the piled walls.

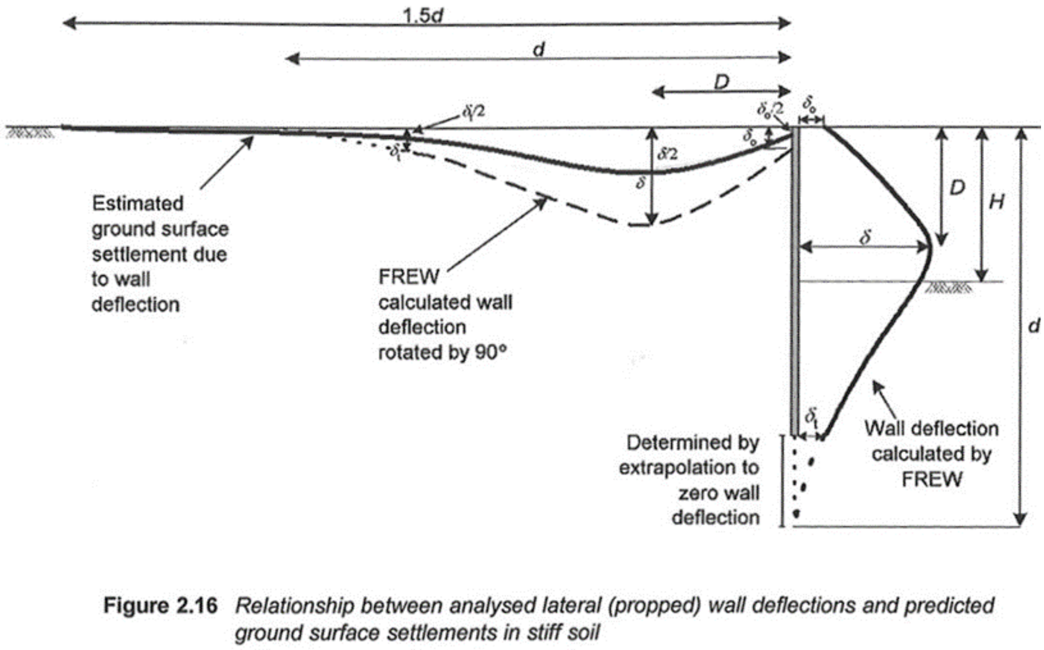


Figure 26 CIRIA C580 prediction of vertical ground movements behind a retaining wall

Resulting profiles of vertical ground movement behind the walls due to pile installation and excavation are shown for the 280mm and 600mm diameter contiguous piled walls in Figure 27 and Figure 28 respectively.

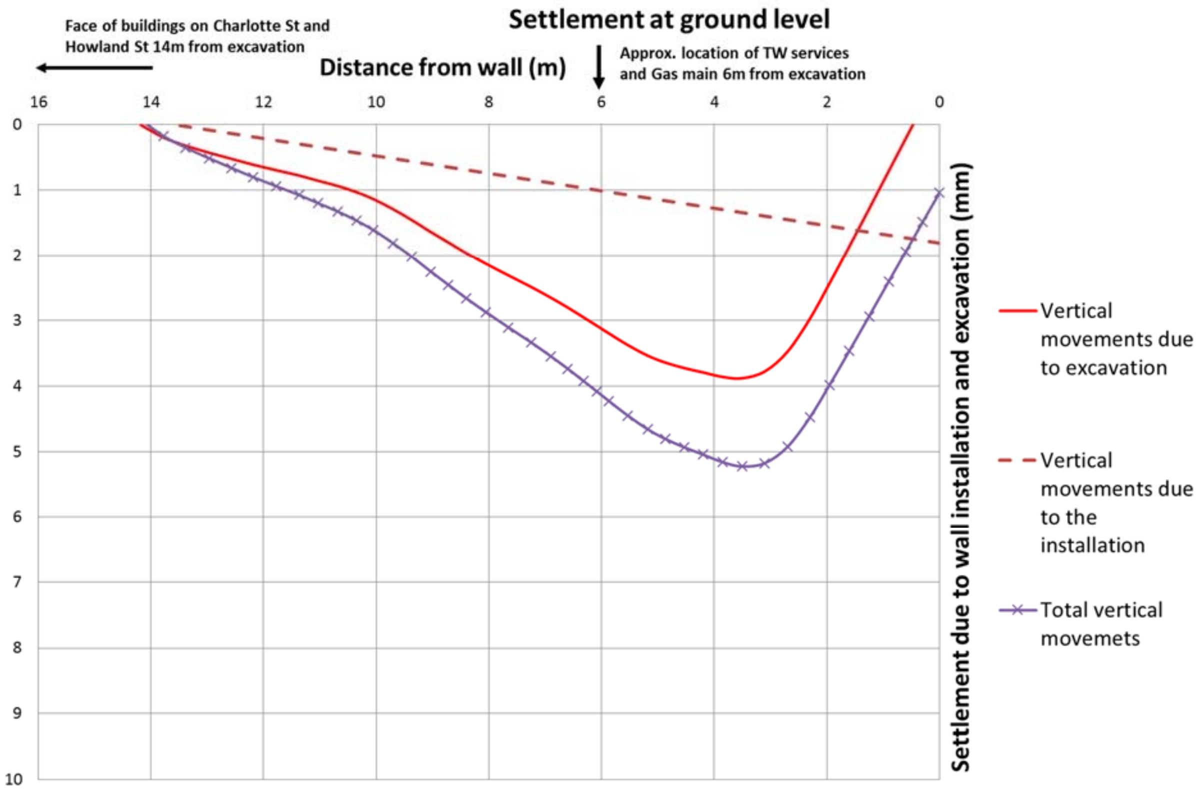


Figure 27 Profiles of vertical ground movement behind the walls due to pile installation and excavation for 280mm diameter contiguous piled wall

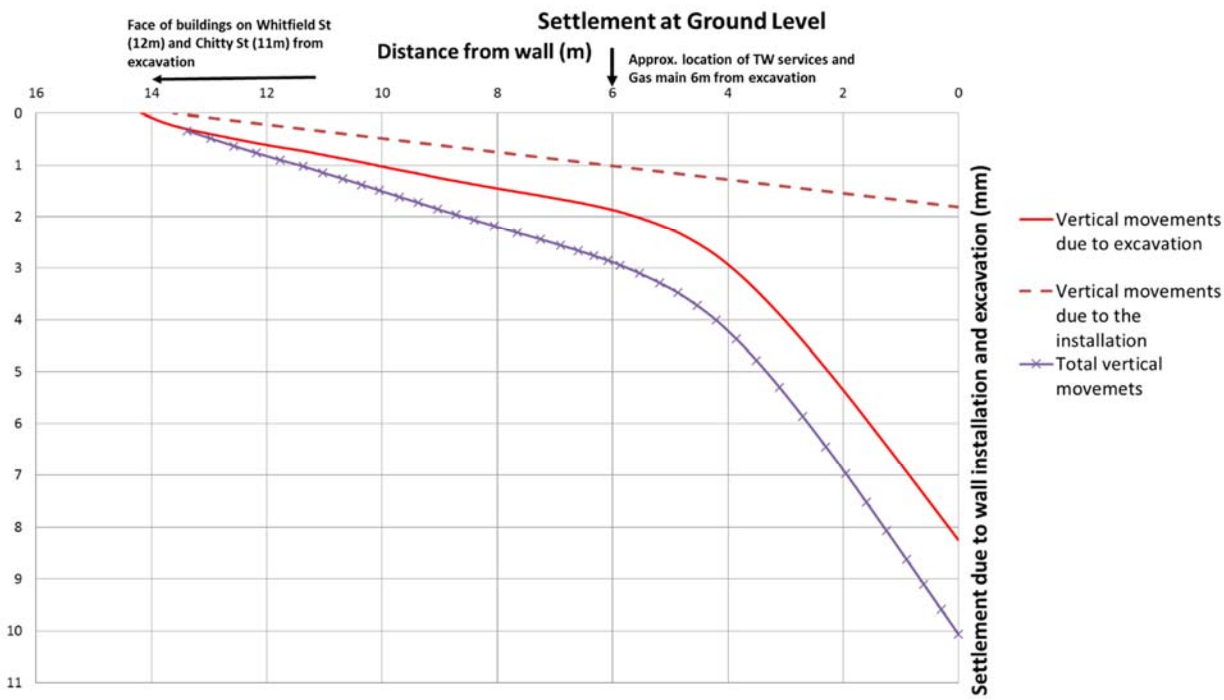


Figure 28 Profiles of vertical ground movement behind the walls due to pile installation and excavation for 600mm diameter contiguous piled wall

9.3.1.2 Horizontal ground movements

Based on data presented in CIRIA C580, horizontal ground movements due to piled wall installation have been taken equal to the vertical ground movements.

Horizontal movements due to excavation have been taken equal to the piled wall deflection directly behind the wall and extending to 4x the depth of excavation from the wall, in accordance with CIRIA C580.

Resulting profiles of horizontal ground movement behind the walls due to pile installation and excavation are shown for the 280mm and 600mm diameter contiguous piled walls in Figure 29 and Figure 30 respectively.

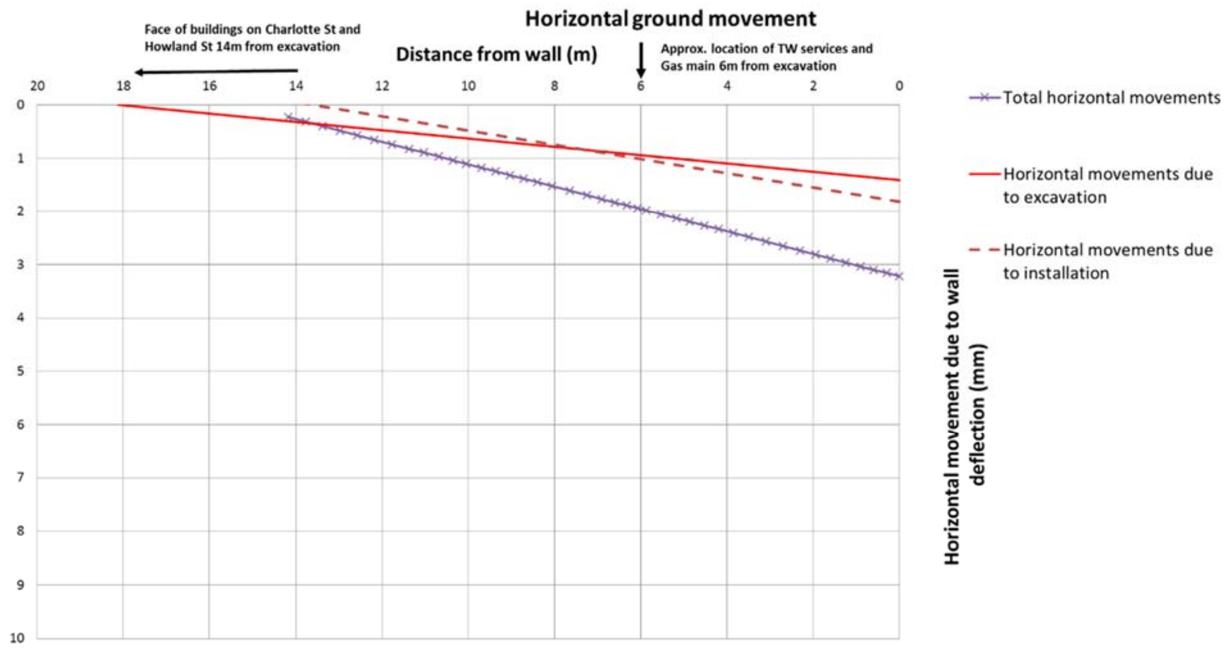


Figure 29 Profiles of horizontal ground movement behind the walls due to pile installation and excavation for 280mm diameter contiguous piled wall

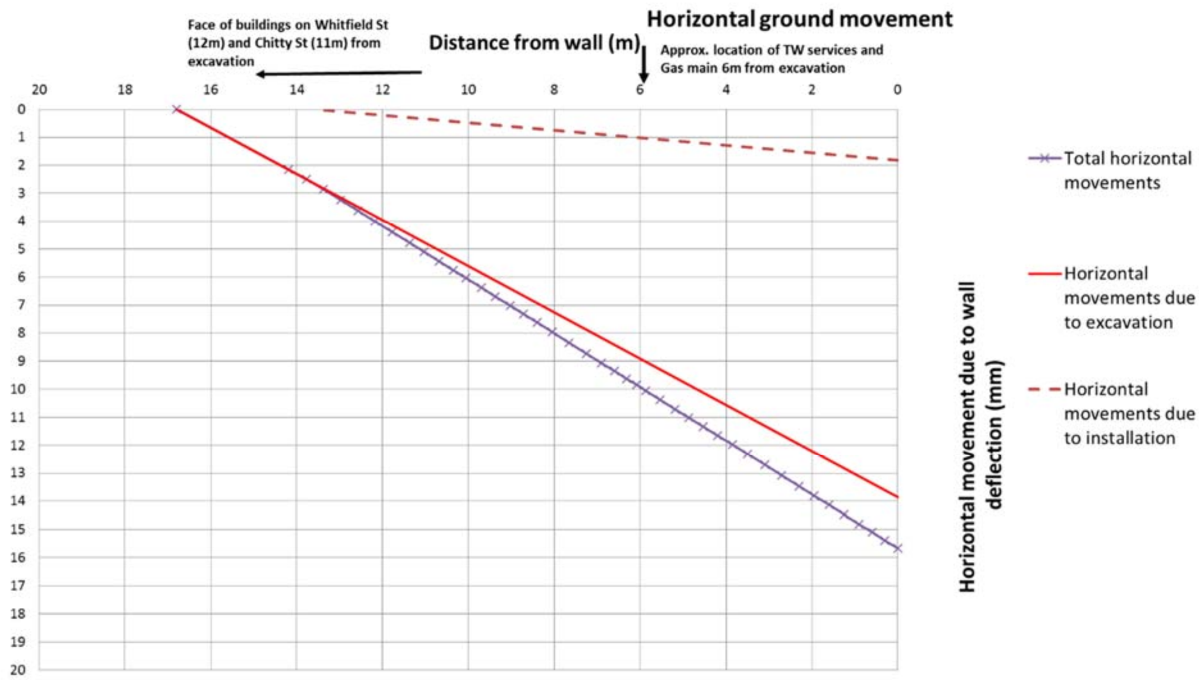


Figure 30 Profiles of horizontal ground movement behind the walls due to pile installation and excavation for 600mm diameter contiguous piled wall

9.3.1.3 Summary of calculated ground movements at shallow depth due to wall installation and excavation

Location	Vertical settlement behind 280mm piled wall (mm)	Horizontal displacement behind 280mm piled wall (mm)	Vertical settlement behind 600mm piled wall (mm)	Horizontal displacement behind 600mm piled wall (mm
Adjacent roads taken to start about 1.5m from excavation	5	3	8	14
TW mains 0.9m to 1.1m below ground level at 6m from excavation	4	2	3	10
TW sewers between +22.9mOD and +24.1mOD	4	2	3	10
Cast iron gas main	4	2	3	10
Edge of buildings across Whitfield St at 12m from excavation	N/A	N/A	1	4
Edge of buildings across Chitty St at 11m from excavation	N/A	N/A	1	5

Note: settlement positive, heave negative

Table 14 Ground movements at shallow depth due to wall installation and excavation

9.3.2 Consideration of global ground movements due to changes in load on the ground in short and long term.

Ground movements have been assessed due to demolition, excavation and net change in load on the ground due to construction of the new development (i.e. net change in load as a result of unloading and reloading). The calculations have been carried out using the Oasys programme PDISP.

9.3.2.1 Changes in load on ground

The pressures used in PDISP for demolition and from the new building are shown in Figure 31 and Figure 32 respectively. The purple lines in the figures indicate the simplified loading blocks used in PDISP. In addition to these pressure changes, unloading due to 2m of excavated ground was taken as -38kPa applied uniformly across the footprint. The basis for the pressures and description of PDISP input is given in Appendix C.

At the locations of 80 Charlotte St Blocks J, K and H, the reinforcing effect of the existing piles on the ground has been taken into account and changes in load due to demolition and excavation have been applied in the PDISP model at 2/3 of existing pile length, taken at +17.6mOD. In other areas changes in load due to demolition and excavation have been applied in the PDISP model at the top of the Terrace Gravels at +21.5mOD, which is the founding level found in the GI for the corner building 67-69 Whitfield St. Since, all the new structure will be piled, all reloading has been conservatively applied at 2/3 of the existing pile length at +17.6mOD. Resultant ground movements at the top of the Terrace Gravels at +21.5mOD have been taken as representative of movements at higher levels in the Made Ground above.

The grid used in PDISP and displacement lines taken for plotting displacements of surrounding infrastructure and structures are shown in Figure 33.

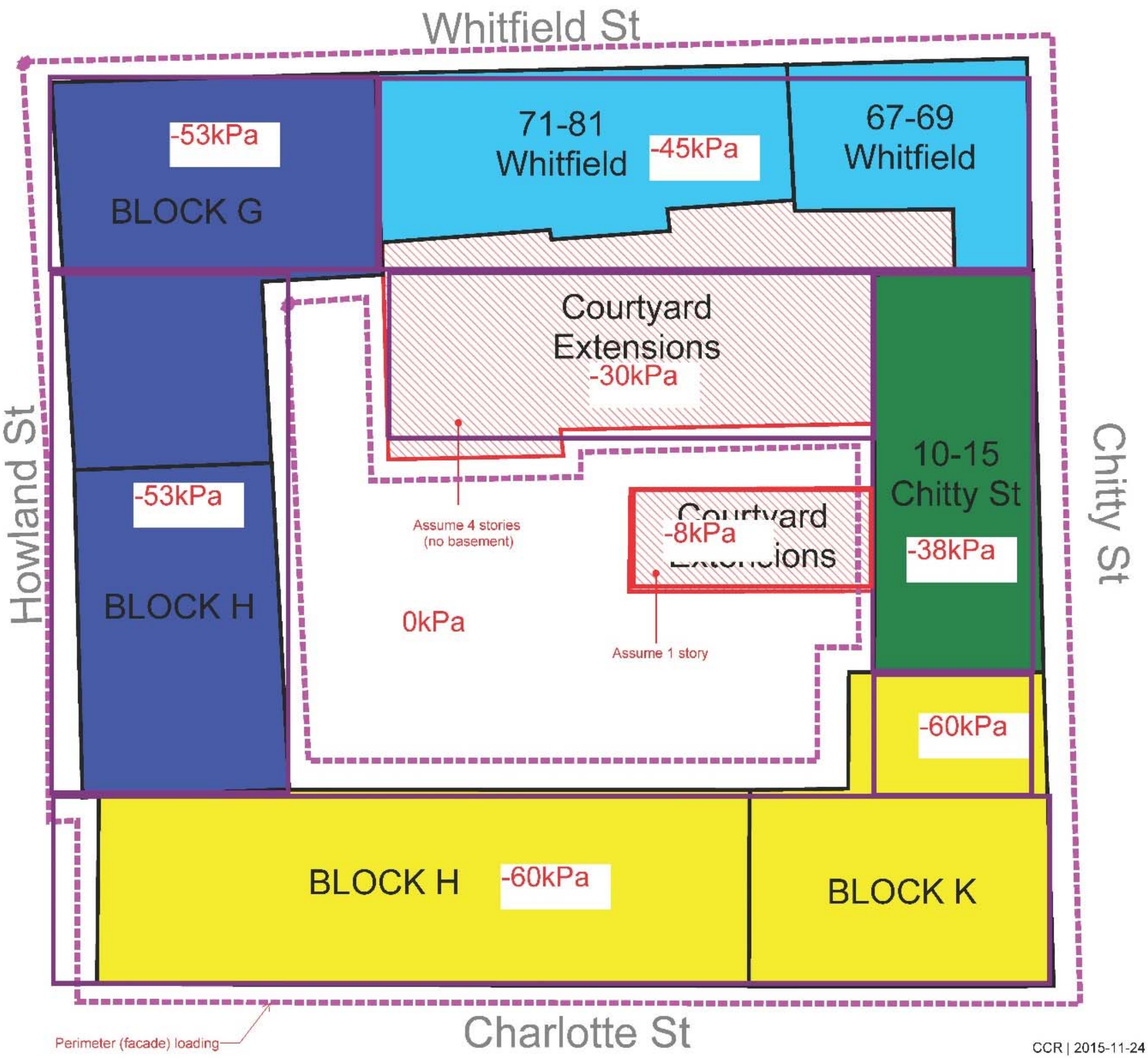


Figure 31 Unloading due to demolition

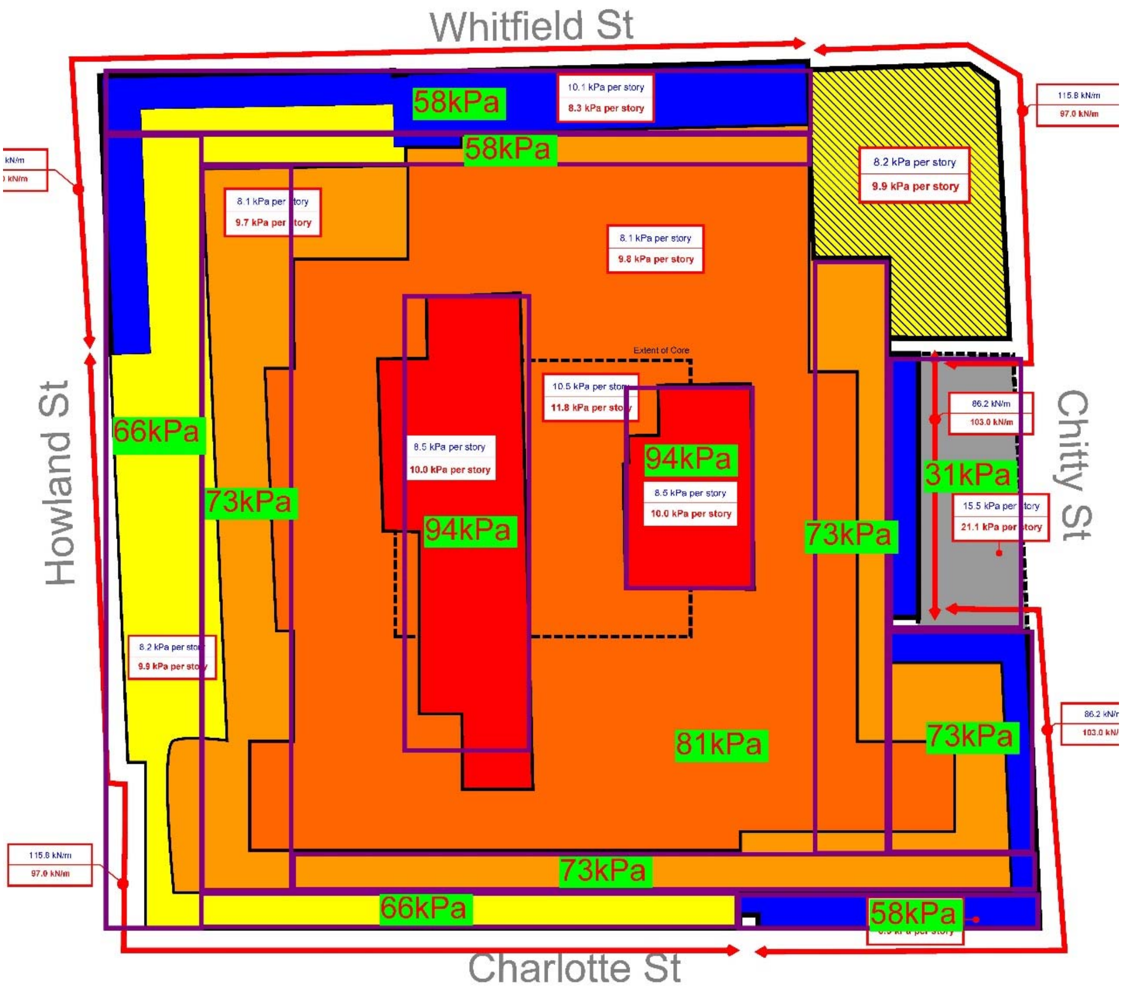


Figure 32 Pressures applied by new building