

Vinegar Yard

Basement Impact Assessment
October 2021





4188 Vinegar Yard
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October 2021

Consulting Structural and Civil Engineers

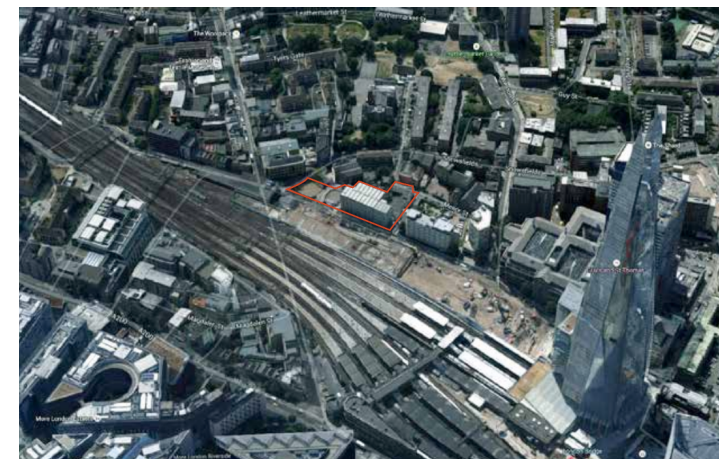
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1 Introduction

St Thomas Bermondsey Limited (the "Applicant") submitted an application for full planning permission for the redevelopment of Vinegar Yard (the "Site") to the London Borough of Southwark ("LBS" or the "Council") on 21 December 2018 under reference 18/AP/4171 (the "Application"). The Application was considered by the Council's Planning Committee on 29 June 2020. Officers recommended the Application for approval subject to conditions and S106, but LBS resolved to refuse.

On 24 August 2020 the Mayor notified LBS and the Applicant of his intention to recover the Application for his own determination (GLA ref. GLA/6208/S2). Since then, the Applicant has been working with officers at the GLA and Southwark to amend the development proposals, seeking amongst other changes to address LBS' heritage concerns that had led to the local refusal. This basement impact assessment has therefore been prepared as part of a package of materials for submission to the GLA for the purposes of public consultation and consideration of the Revised Scheme by the Mayor.

The proposed development is located in Central London within the boundary of Southwark City Council. It is bounded by St Thomas Street, Fenning Street, Vinegar Yard and Snowfields Street, including No. 1-7 Fenning Street and No. 9 Fenning Street at the postcode of SE1 3QR.

The site is currently vacant with the exception of 2No. warehouses which occupy the south western portion of the site, fronting Fenning Street. Guy's and St Thomas' Hospital, The Shard and London Bridge Station are notable structures located nearby.

This basement impact assessment report covers the anticipated substructure construction currently proposed to form the basement of the Vinegar Yard development and should be read in conjunction with other Structural, Architectural and Consultant drawings, sketches and reports submitted with the planning application. It should be noted that this report is based upon preliminary information and is subject to change as the design develops.

This report is prepared for the exclusive use of AKT II and our Client. The content of this report is based upon readily available information in accordance with the project stage requirements, budget and timescales. No assurances can be made regarding the accuracy or completeness of the sourced data. All comments and conclusions in this report are based upon the assumption that the data is reliable. AKT II accepts no liability for any inaccurate conclusions, assumptions, or actions taken resulting from inaccurate information supplied to AKT II by others.

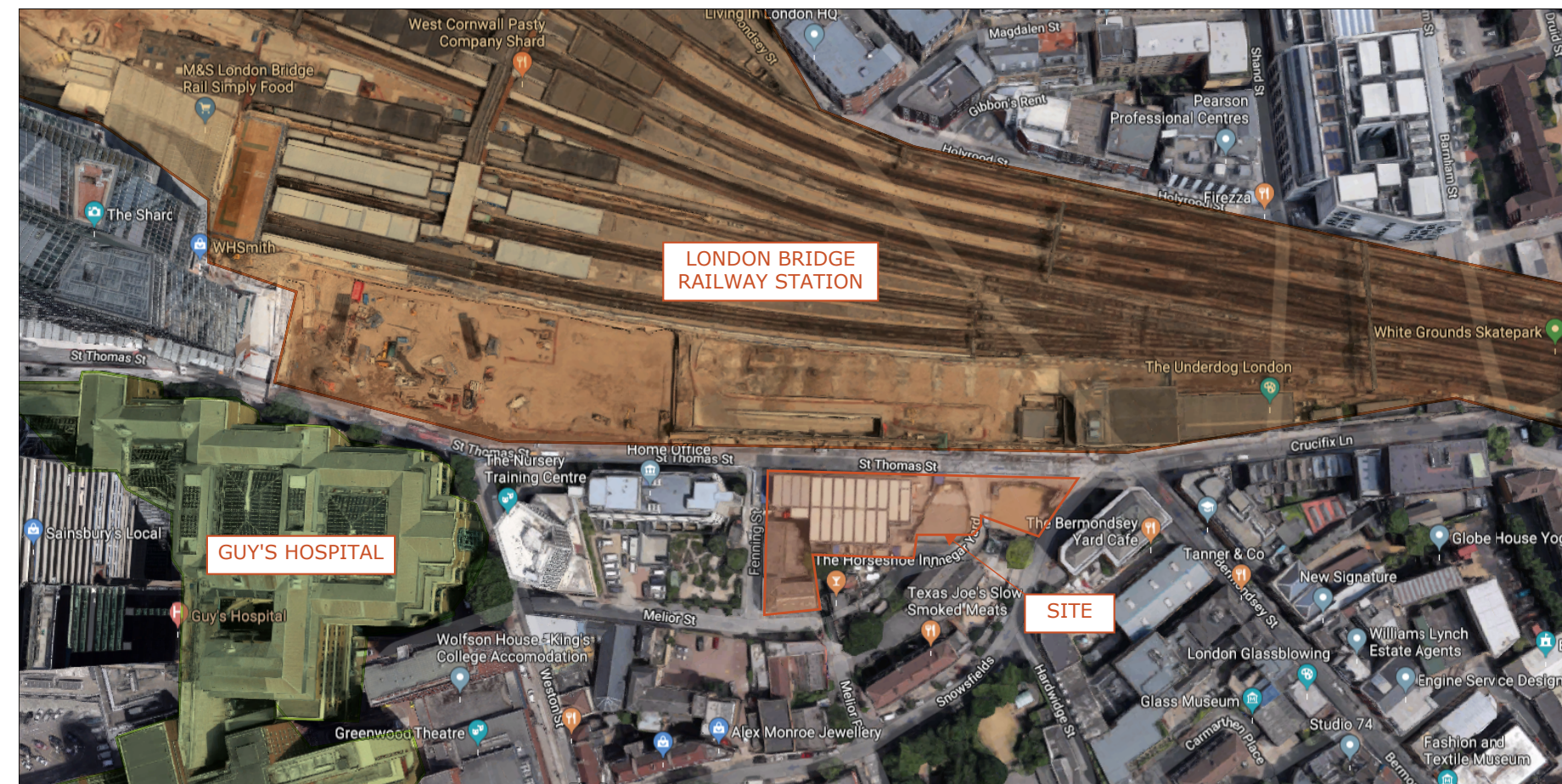


Figure 1.1 Aerial view of the site

2 The Project

The site comprises of an irregular shaped plot of land located in the Southwark City Council, in the London Bridge area. The site is bounded by St Thomas Street, Fenning Street, Vinegar Yard and Snowfields Street, including No. 1-7 Fenning Street and No. 9 Fenning Street at the postcode of SE1 3QR. The site is located in a commercial area and is surrounded by buildings of various ages. London Bridge mainline railway station is located immediately to the north of the site on the opposite side of St Thomas Street. The site is currently vacant with the exception of 2No. warehouses which occupy the south western portion of the site, fronting Fenning Street. Notable structures in local vicinity include Guy's and St Thomas' hospital and The Shard.

The new development, designed by KPF Architects, is part of the proposed Masterplan for St Thomas Street East and responds to the evolution of London Bridge Station and the opening up of the railway arches for commercial use.

The redevelopment of the site includes the demolition of existing buildings, retention and refurbishment of the warehouse and the erection of a ground, mezzanine and 18 storey building (with plant at roof) and 3 basement levels, comprising of café and community space within the warehouse and within the new building office, flexible medical and research and development, and flexible retail and affordable workspace, alongside cycle and disabled car parking, servicing, refuse and plant areas, public garden (including soft and hard landscaping), highway improvements and all other associated works.

The architectural proposals are illustrated in Figure 2.1.



Figure 2.1 Architects render of the Vinegar Yard development view from south east

3 The Site

3.1 Site Location

The site is bounded by St Thomas Street, Fenning Street, Vinegar Yard and Snowfields Street, including No. 1-7 Fenning Street and No. 9 Fenning Street, SE1 3QR. The National Grid reference for the site is TQ 330799, Latitude 51.502604 and Longitude -0.084233608. The site is located in a commercial area and is surrounded by buildings of various ages. London Bridge mainline railway station is located immediately to the north of the site on the opposite side of St Thomas Street.

The wider contextual location (see Fig 3.1) shows the site located 400m south of the River Thames, in close proximity to The Shard and Guy's and St Thomas' Hospital.

3.2 Site Description

The development application site comprises of an irregular shaped plot of land located in the Borough of Southwark City Council, in the London Bridge area, occupying an area of around 0.30 Ha.

The majority of the site is currently being used as car park area with a small security cabin located on the easternmost side.

A low rise industrial building is located at 1-7 Fenning Street. It is a two storey brick warehouse with a concrete ground floor. An electrical substation is present on the western side of Fenning Street approximately 20m to the west of the site.

3.3 Surrounding Land Use

The most historically noteworthy structure in the vicinity of the site is the London Bridge Railway Station, the oldest railway station in London. Situated immediately north of the site, London Bridge Station provides London with one of its major interchange stations, served by both the Northern and Jubilee London Underground lines, Overground lines, and several National Rail services. It is one of the most important and bustling transport hubs in London.

The Station occupies a large area on three levels immediately south-east of London Bridge. The main line station is the oldest railway station in Central London and was opened in 1836. It has been developed and enhanced several times to provide an increased capacity for the station.

The tracks are located above the street level on top of the London Bridge - Greenwich Railway Viaduct. This old viaduct consists of a series of nineteen brick viaducts linked by road bridges with a total 5.55 km in length. The structure consists of 851 semi-circular arches and 27 skew arches or road bridges. It is a Grade II listed structure.

The arches along St. Thomas Street have undergone extensive restoration and are now used as retail units.

Immediately to the south of the proposed development there is The Horseshoe Inn pub. This three storey brick building was constructed in 1897 and is well preserved. Despite of not being listed it has an important role on the redevelopment of the area.

Becket House and Capital House are located to the west side of the plot, separated by Fenning Street. Becket house is occupied by the Home Office Border and Immigration Agency. Recently a planning submission has been submitted to demolish this to enable the construction of a high rise office building part of the Edge London Bridge development. Capital House was originally occupied by King's College London and is currently undergoing development as part of the St Thomas east Masterplan to construct a residential tower. Both buildings were originally constructed during the late 1960s and early 1970s.

It is worth noting Guy's and St Thomas' Hospital and its campus further west from the plot. The hospital dates from 1721 and the original buildings formed a courtyard facing St Thomas Street. The original hospital expanded and new buildings were added in 1850, 1974 and 2016. Located in the centre of the complex there are two tall buildings with 34 and 29 storeys known as Tower Wing and Guy's House respectively, which were designed by Watkins Gray in 1974.

The Shard was opened to the public on 2012 and it is a 95 storey tower designed by Renzo Piano. It is currently the tallest building in the UK and the most iconic building in the area.

Residential buildings of 3 to 5 storeys are located to the south-east side of the site with Snowfields passing between the proposal and these properties.



Figure 3.1 Aerial view looking north capturing The Shard, Guy's and St Thomas' Hospital and London Bridge Railway Station.



Figure 3.2 St Thomas street, view looking west from union car park.



Figure 3.3 Looking north from union car park to Horseshoe inn.

3.4 Site History

3.4.1 Historical Background

The history of the site and its surrounding area has been assessed using extracts from John Rocque's 1746 Map of London, historical Ordnance Survey (OS) maps from 1851 to the present day and other reliable information. Note that with regards to the OS maps, the published date is given and not the survey date, therefore they only indicate information on the date the survey was carried out and they do not give a continuous record of development.

Other sources used include archive information from the London Metropolitan archives and from reports commissioned for the local area and research online.

3.4.2 Site context and history

The site and its surroundings forms part of an area historically known as the Pool of London which is part of the River Thames stretching between London Bridge and Tower Bridge.

It is suggested that this zone was first occupied as part of a Roman settlement. During this time the area was actually an islet sitting within the course of the Thames. During its peak, in around the 2nd century, it is estimated as many as 3000 people may have lived there. London Bridge was the first bridge built across the River Thames by the Romans.

During the 18th century the history of the area is intrinsically connected with that of Guy's and St Thomas' Hospital and by the 19th century had become an important trade and commercial centre. However, during the 1960s the area started to lose its importance as an international port and warehouses and port buildings fell into disrepair. Since the 1980s the area has been in constant development. Old warehouses were refurbished and converted into offices and retail units.

Nowadays London Bridge is a globally significant central London district which is home to international business headquarters.

In addition, the development of the London Bridge Railway station has converted the area in one of the UK's busiest transport hubs, that has increased rail capacity and vastly improved the quality of the public realm.

The earliest readily available OS map covering the site is dated to 1875 and shows that half of the site was occupied by a Hop Warehouse and some houses. The centre of the plot is greenfield without any constructed building on it (Figure 3.4).

By 1887 the entire site footprint was occupied by a combination of dwelling, commercial units and warehouses (Figure 3.6)

The site remained largely unchanged up to the 1940s. As a result of the Blitz and the extensive German bombing during the Second World War, the site and much of the area in the immediate vicinity was significantly damaged (Figure 3.5).

By 1951 the site is still occupied by commercial units and warehouses but residential units had been demolished and space made for car parking (Figure 3.7).

In 1916 a tannery can be seen some 150m south of the site, but generally, the site remained largely unchanged up to the 1940s with maps around this time show minor modifications.

Since the end of the 20th century until now the site remained largely undeveloped. During the extension of the London Bridge Station to accommodate the Thameslink programme the site was used as a storage and welfare compound by Network Rail (Figure 3.8).

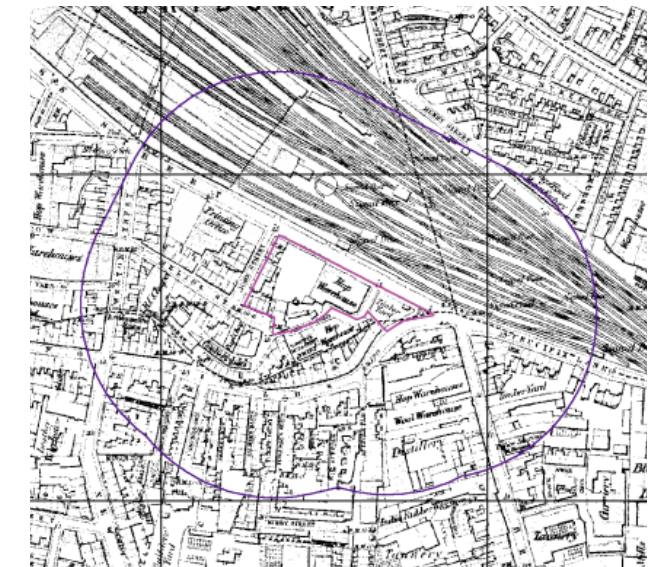


Figure 3.4 1878 London Bridge area map

Black	Total destruction
Purple	Damage beyond repair
Dark red	Seriously damaged, doubtful if repairable
Light red	Seriously damaged, but repairable at costs
Orange	General blast damage; not structural
Yellow	Minor damage; minor in nature
Light Green	Clearance areas
Small circle	V2 bomb
Large circle	V1 bomb

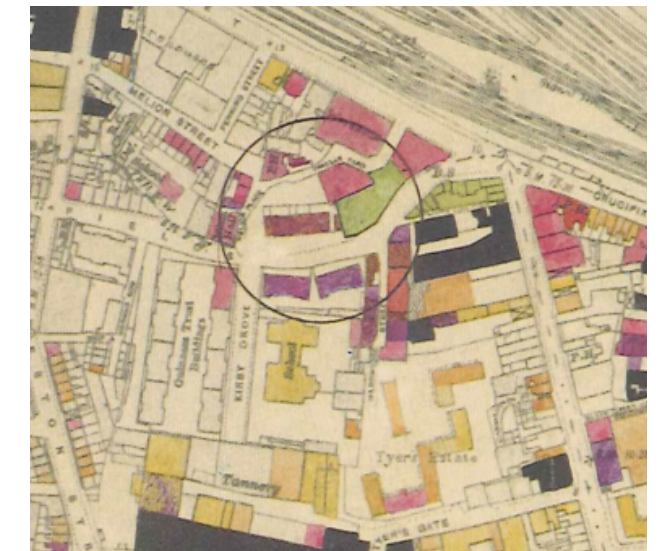


Figure 3.5 Bomb map and damage to surrounding area

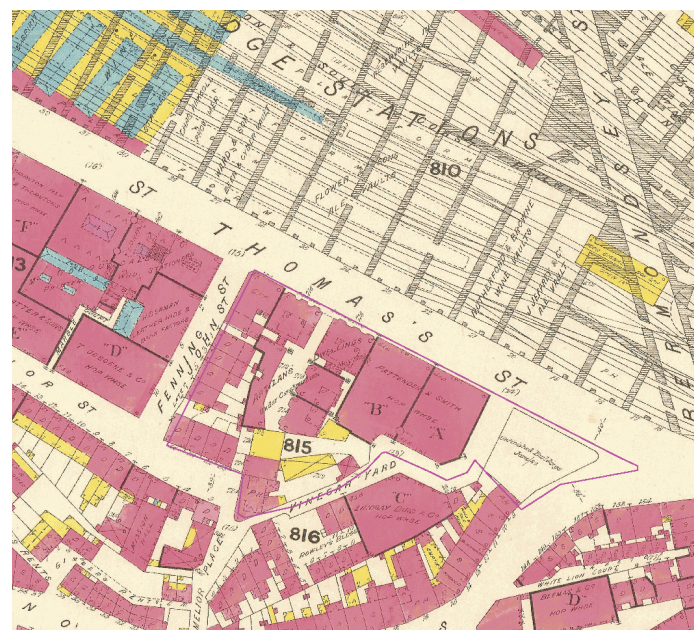


Figure 3.6 Vinegar Yard historic plan published 1887.

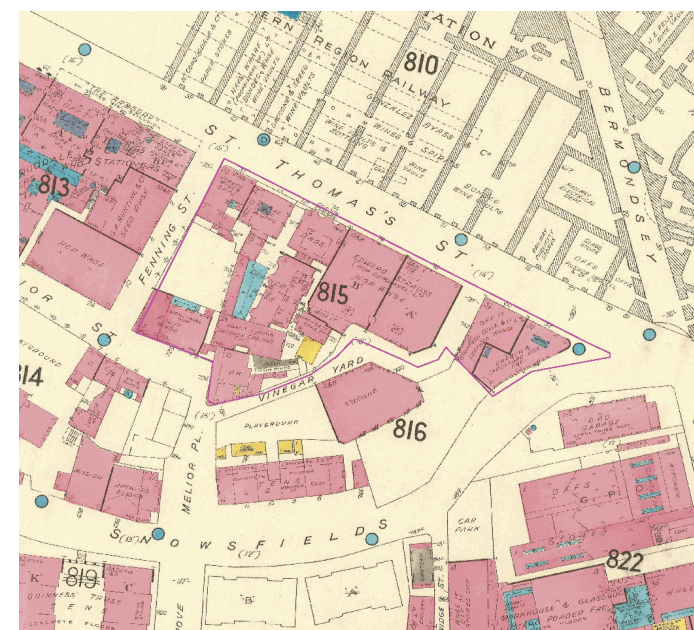


Figure 3.7 Vinegar Yard historic plan published 1951.



Figure 3.8 Aerial view of site and Network Rail compound

4.1.5 Archaeology

Archaeological deposits are present beneath most towns and cities in the UK and can date back over 2000 years to the Roman period.

The site lies within the 'Borough, Bermondsey and Rivers' archaeological priority zone. Very significant archaeological remains are known within the immediate area. The area, which is defined by the London Planning Authority (LPA), extends to Bankside, Bermondsey, and Rotherhithe.

- The site is likely to contain archaeological remains given the historical finds in the local area.
- It is suggested that the upper circa 2.0m of recent made ground will be of little/no archaeological interest due to historic bomb damage and subsequent redevelopments.
- The development impact for the site's archaeological resources is dependent on the extent of basements and foundation solutions. Deep basements and piling activities are likely to have a 'potentially major impact'. This is thought to influence any archaeological requirements placed on the development.

It is recommended that any geotechnical pits that are excavated for engineering purposes should be closely monitored by a competent archaeological organisation. This will provide further information on the nature and levels of deposits beneath the existing ground level.

It is recommended that engagement with the LPA and their archaeological department is undertaken at an early opportunity to understand the specific requirements. From previous AKT II project experience, it is generally expected that the LPA will impose a number of conditions on any planning permission granted. Generally, these can include:

- Submission of an implementation programme of archaeological mitigation works in accordance with a scheme of investigations to the LPA.
- Submission of an assessment report detailing proposals for post-excavation works.
- Submission of a detailed scheme of proposed foundations for approval.

It is difficult to predict at this stage the extent of site investigations required and associated programme length. Depending on the extent of archaeological interest of the site, the scope of site activities can range from a watching brief over a number of trial pits, to an extensive archaeological excavation site being established.

Perusal of publicly available information shows that archaeological remains have been found, in recent nearby developments, including:

- Iron Age ditches on St Thomas Street.
- Roman age remains at 4-26 St Thomas Street and King's Head Yard.
- Former graveyard south of St Thomas Street, to the east of the site.

4.2 Key site boundary conditions

4.2.1 NWR -London Bridge Station Viaduct

Immediately north of the site is The London Bridge - Greenwich Railway Viaduct, which consists of a series of nineteen brick railway viaducts linked by road bridges between London Bridge railway station and Deptford Creek and is classed as a Grade II listed structure.

The location of the key station directly north of Vinegar Yard on the opposite side of St Thomas Street may pose a number of constraints including:

- Issues of cranes over-sailing Network Rail assets and live railway lines.
- A portion of the London Bridge station facade facing on to St Thomas Street is known to be Grade II listed. It is also known that large sections of this were replaced recently as part of the Thameslink programme.

Engagement with NWR will be required in the next design stage to discuss the proposed development. It is likely that a Basic Asset Protection Agreement (BAPA) will be required, which will be followed by the submission of a Front End Pack that describes the scheme and its interfaces with NWR assets. This will then allow the commercial agreements to be put in place that will lead to the technical submissions to justify that the proposed scheme will not adversely impact any NWR assets.

4.2.2 St. Thomas Street - TfL Red Route

St. Thomas Street, running along the north side of the site, is a designated TfL Red Route (Figure 4.6). These routes are controlled and enforced by Transport for London as opposed to the local authorities in which the road is situated and may pose the following constraints:

- Restricted access to site
- Delivery of on-site materials and equipment.
- On-site storage may be restricted.

The appropriate approvals process with TfL should be started to gain clarity on TfL requirements and restrictions.

4.2.3 Party Wall Conditions & Adjacent buildings

The proposed development site is surrounded by minor roads and a number of buildings (see Figure 4.5).

The adjacent buildings range in size, height and type of construction. All buildings within the influence zone of the new basement construction (a 45 degree line from the base of the new foundation of the development) will be assessed for the impact of the excavation and construction of the new retaining structure in accordance with CIRIA C760. This assessment will be used to develop a performance criteria for the retaining wall movements which will be issued to the piling contractor to inform their design. This performance criteria will ensure that movements of the surrounding soils during piling, excavation and reload of the soils do not impose strains onto the adjacent building in excess of specified limits as defined in Table 6.4, CIRIA C760 (limits to be agreed), but it is aimed to achieve categories 'Very Slight' to 'Slight'

Building 1: Six storey office block, appears to be of approximately 1960s construction, which will be replaced by the new office building part of the Edge London Bridge development

Building 2: Three storey block on the corner of Melior St. of masonry construction

Building 3: The Horseshoe Inn: Three storey pub of masonry construction.

Building 4: Four storey industrial/ex industrial building with masonry facade

Building 5: Four storey ex-industrial building used as cafe/offices with masonry facade

Building 6: London Bridge Station: 1-2 storey building with retained masonry facade (recently refurbished)

Building 7: Six Storey residential apartment building, appears to be of recent (last 5 years) construction

Further investigation works are required to ascertain the construction typology, presence of basements and condition - these will be undertaken in later project stages.

4.3 Proposed investigations

The following outlines AKT II recommendations for specialist investigations to be undertaken at the earliest available opportunity. These investigations are intended to provide further information on the parameters required for detailed structural design as well as to reduce/mitigate risk associated with the site. The recommended investigations include:

- Project specific site investigation
- CCTV drainage surveys
- Condition surveys of the Thames Water sewers below the surrounding highways
- Condition survey of the immediate adjacent structures (to be agreed)
- An asbestos survey should be carried out, prior to any demolition works.



Figure 4.6 TfL map of red routes.



Figure 4.5 Key plan of adjacent buildings - Vinegar Yard site in RED

5 Ground Model

The purpose of this section of the report is to highlight any risks associated with geotechnical studies and underground structures.

5.1 Published Geology

London belongs to the Thames basin which is a broad syncline of chalk occupied in its centre by sands and clays. Fluvial deposits associated with the former deposits of the River Thames lie on top of the bedrock. These different terraces are the remains of the river's floodplains.

The Envirocheck Superficial Geology map (Figure 5.1) indicates that the site is underlain by Alluvium. The Envirocheck Bedrock Geology map in (Figure 5.2) indicates that the site is underlain by London Clay Formation.

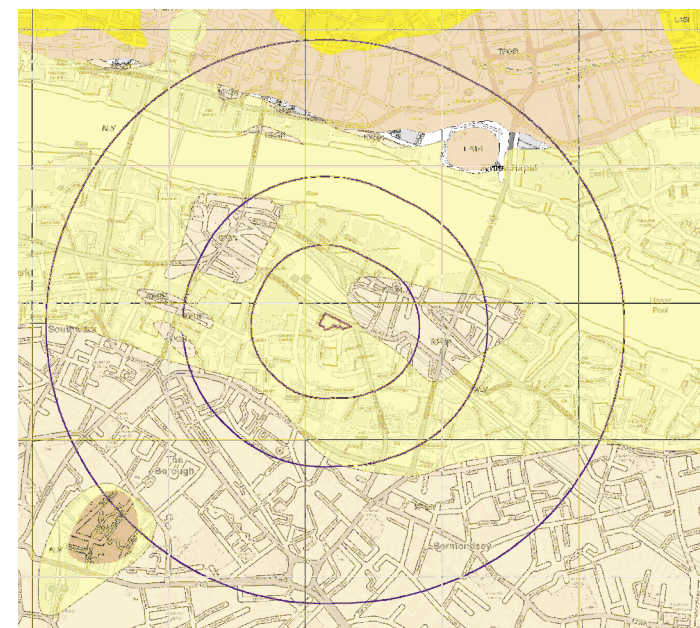


Figure 5.1 Envirocheck Superficial Geology map

5.2 Previous Site Investigations

AKT has reviewed a previous ground investigation and combined geotechnical and contamination assessment of the site carried out in March 2012 by Ashdown Site Investigation Limited. The purpose of the works was to:

1. Assess the expected geology and hydrogeology underlying the site;
2. Establish the development history and most recent site use;
3. Identify potential sources of on-site and off-site contamination;
4. Establish the potential for on-site migration of contamination from off-site sources;
5. Identify sensitive receptors that may be at risk from any contamination migrating from the site and develop a preliminary conceptual model;
6. Assess ground and groundwater conditions prevailing at the site;
7. Provide information to assist others in undertaking design of foundations, ground floors and road pavements;
8. Test for the presence of potentially hazardous contamination and gas in the ground;
9. Provide a quantitative contamination risk assessment; and
10. Provide a quantitative site specific conceptual model.

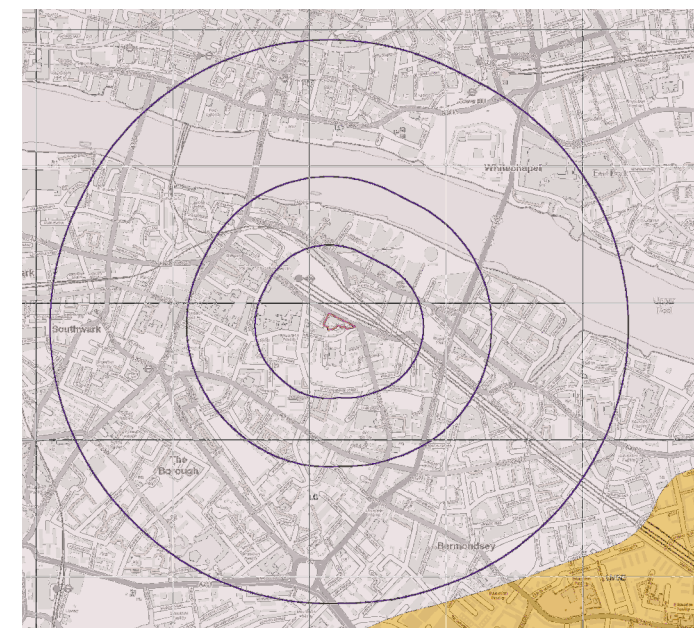


Figure 5.2 Envirocheck Bedrock map

5.3 Anticipated Ground Conditions

Boreholes located in close vicinity to the site were obtained from the BGS archives. The locations of the boreholes and their relative ground models are illustrations in Figures 5.3 - 5.4.

Although BGS boreholes provide a good indication of the likely conditions on the site, it is recommended that a full site investigation is carried out in order to investigate the ground conditions specific to the site and inform the foundations and substructure design.

Published British Geological Survey maps, borehole records and previous site investigation report indicate that the geology underlying the site comprises Made Ground, Alluvium, Kempton Park Gravel formation, London Clay and Woolwich & Reading Beds (Lambeth Group).

Made Ground

Made ground, generally comprising silty sandy fine to coarse gravel but locally comprising silty sandy clay with a variable gravel content and cobbles was recorded at each exploratory hole position. The gravel and cobble component of the soils variously comprised brick, flint, concrete, coal, ash, clinker, crystalline rock, quartz and glass. Wood, shells and metal were also locally recorded within the made ground soils.

Made Ground, has been recorded to a depth varying from 1.20m to 2.00m below ground level. The previous phase of works also recorded several obstructions including concrete slabs and buried foundations to be present within the made ground.

Alluvium

Underlying the made ground, firm silty clay containing a variable proportion of carbonaceous matter, flint, shell fragments and calcareous nodules was recorded 2.00m below ground level. These soils are considered to represent the Alluvium deposits indicated on the published geological map.

Alluvium is predominantly made up of soft, compressible clays, with localised organic pockets, silt and sand partings and peat bands. They, along with the River Terrace Gravels, represent

ancient flood plain deposits that have become isolated as the river cut down to lower levels. Alluvium is associated with the production of methane ground gas due to a high level of organic material.

The Alluvium deposits were recorded to depths of between 4.30m and 6.40m during the previous phase of works undertaken at the site.

Kempton Park Gravel formation

Underlying the Alluvium, historic Boreholes generally found medium dense to dense sands and flint gravels to depths of between 8.90m and 10.05m below ground level.

Although not indicated to be present on the site, these soils are considered to be representative of the Kempton Park Gravel Formation soils shown in the vicinity of the site on the published geology map.

London Clay Formation

Beneath the Kempton Park gravel formation, the London Clay formation comprises a stiff to very stiff grey fissured clay. With depth, the clay becomes very stiff/hard, is frequently fissured and can contain some inclusions and beds of weak mudstone and siltstone.

Generally, the London Clay layer varied between 24m and 29m in thickness, where the depth of the borehole was sufficient to reach the change in layer.

Woolwich & Reading Beds (Upper Mottled Clay)

Woolwich & Reading Beds were recorded on the deepest boreholes -29m to -36m OD.

Thanet Sand

From boreholes on nearby sites, it is believed that Thanet Sands lie at -48m OD.

Superficial Geology

Map Colour	Lex Code	Rock Name	Rock Type	Min and Max Age
[Light Yellow]	ALV	Alluvium	Clay and Silt	Flandrian - Pleistocene
[Light Orange]	KPGR	Kempton Park Gravel Formation	Sand and Gravel	Devensian - Ipswichian
[Yellow]	LASI	Langley Silt Member	Silt	Devensian - Ipswichian
[Light Brown]	TPGR	Taplow Gravel Formation	Sand and Gravel	Wolstonian - Chokierian
[Dark Brown]	PEAT	Peat	Peat [Unlithified Deposits Coding Scheme]	Quaternary - Ryazanian

Historic Boreholes

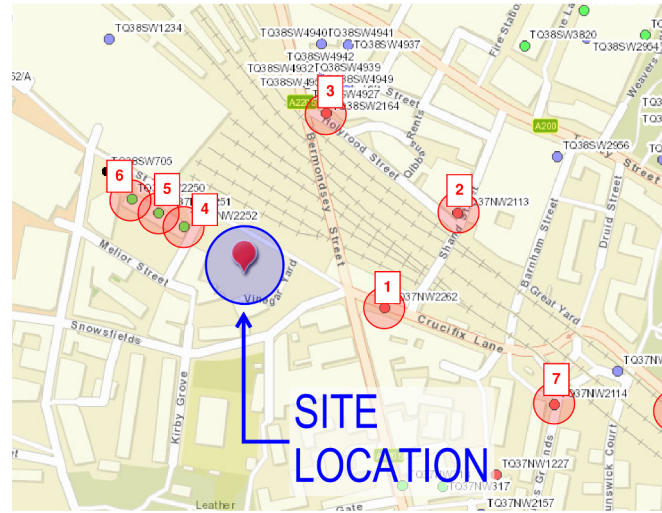


Figure 5.3 BGS Borehole Locations.

Legend

- Made Ground
- River Terrace Deposits (Alluvium + Gravels)
- London Clay
- Lambeth Group/Woolwich & Reading Beds
- Thanet Sand

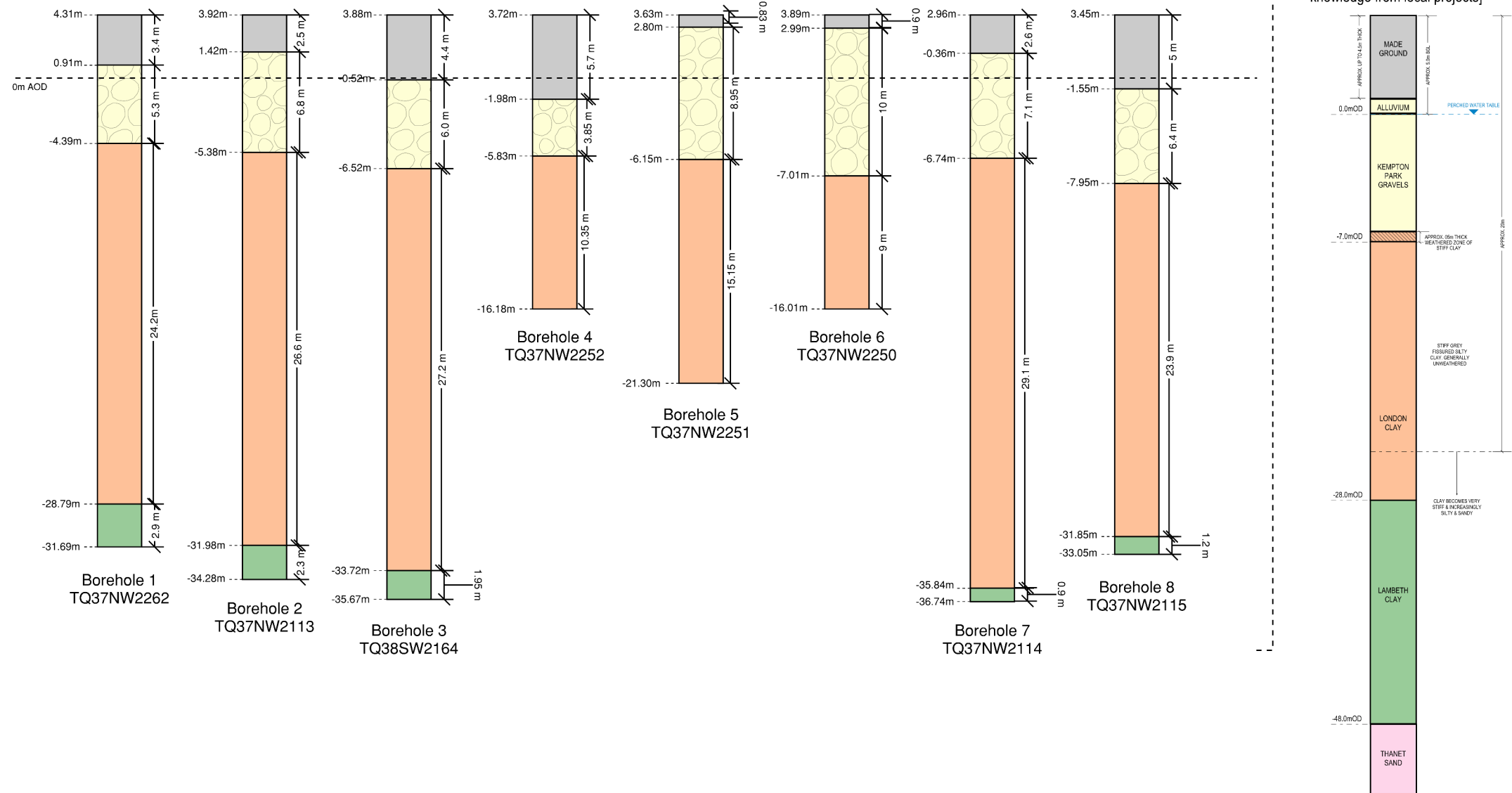


Figure 5.4 BGS Borehole profiles

5.4 Hydrogeology and Hydrology

5.4.1 Environment Agency Classification

A Principal Aquifer is defined by the Environment Agency (EA) as layers of rock or drift deposits that have high inter granular and/or fracture permeability - meaning they usually provide a high level of water storage. They may support water supply and/or river base flow on a strategic scale.

A Secondary A Aquifer is defined by the Environment Agency 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.

A Secondary B Aquifer is defined by the Environment Agency as predominantly lower permeability layers which may store and yield limited amounts of groundwater due to localised features such as fissures, thin permeable horizons and weathering.

A Secondary Undifferentiated (U) Aquifer is assigned by the Environment Agency in cases where it has not been possible to attribute either category A or B to a rock type.

Unproductive Strata have negligible permeability and are generally regarded as not containing groundwater in exploitable quantities.

5.4.2 Hydrogeology

The superficial Alluvium expected beneath the site is classified as a Secondary Undifferentiated Aquifer. The Secondary Undifferentiated Aquifer designation is usually given in cases where it has not been possible to attribute either Secondary A or B status to a rock type. In most cases, this means that the layer in question may have not previously been designated as both 'minor' and 'non-aquifer' in different locations due to the variable characteristics of the rock type.

The London Clay Formation expected at depth beneath the site is classified as Unproductive Strata, as per the Envirocheck Groundwater Vulnerability map (Figure 5.5). Unproductive Strata are deposits with low permeability that have negligible significance for water supply or river base flow.

5.4.3 Source Protection Zone

The EA have defined Source Protection Zones (SPZ's) for groundwater sources such as wells, boreholes and springs used for public drinking water supply. The zones are defined by the EA as outlined below:

- The Inner Protection Zone is the distance travelled by groundwater from any point below the water table to the abstraction in 50 days for a particular area. It has a minimum radius of 50m.
- The Outer Protection Zone is the distance travelled by groundwater from any point below the water table to the abstraction in 400 days for a particular area. It has a minimum radius of 250m.
- The Total Catchment Zone is the area around the abstraction within which all groundwater recharge is presumed to be discharged to the source.

The SPZ Map from Envirocheck indicates that the site is not located within a Protection Zone (Figure 5.8). It is likely that the ground water abstractions are from the Chalk Aquifer. This means that there is no risk of pollutants or contaminants from the site making their way into a source of drinking water.

5.4.4 Ground Water Level

The cessation of abstraction from the chalk after the mid-1960s for industrial purposes caused groundwater levels to rise throughout the London basin. General Aquifer Research, Development and Investigation Team (GARDIT) was developed in order to minimise and ultimately halt the groundwater rise, but the EA's view is that rising groundwater no longer poses a problem and underground infrastructure is no longer threatened by inundation.

BGS borehole logs record groundwater first being struck at 3.40m below ground level to 4.60m below ground level, which indicates the water table to generally be found within the Alluvium layer. It should also be noted that groundwater levels are subject to variations caused by changes in the local drainage conditions and also by seasonal effects and as a result will be confirmed by standpipes in the ongoing SI.

5.4.5 Hydrology

The subterranean or underground rivers of London are the tributaries of the River Thames and River Lea that were built over during the growth of the metropolis of London. The rivers now flow through underground culverts.

The nearest significant surface water feature is River Thames located circa 400m to the north of the site.

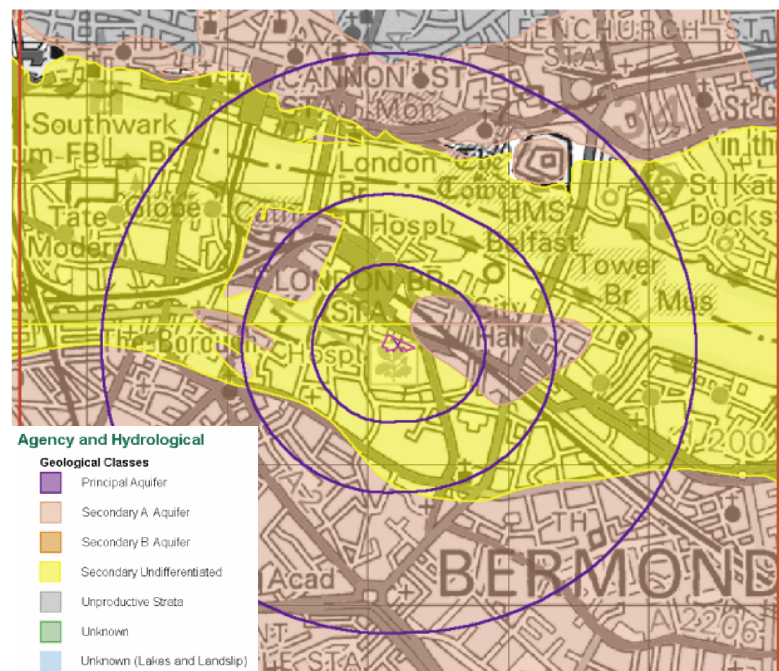


Figure 5.5 Envirocheck superficial Hydrology map.

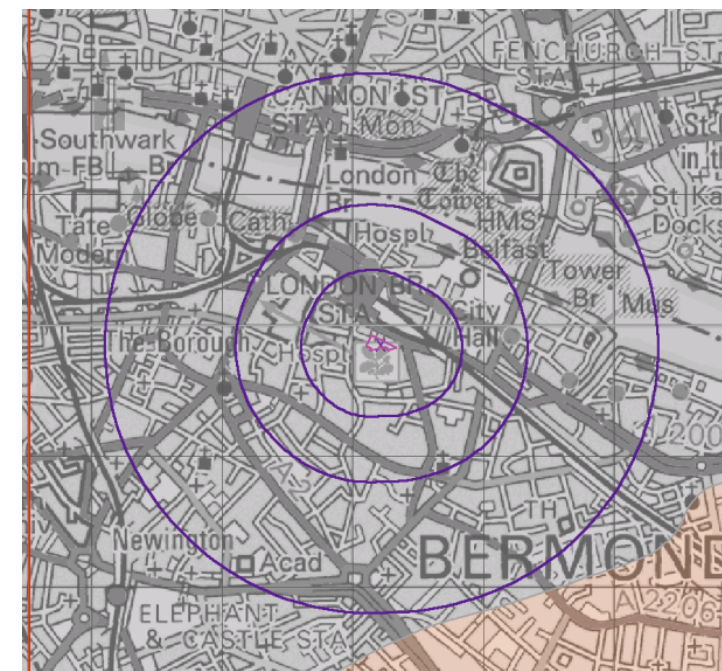


Figure 5.6 Envirocheck Bedrock Aquifer map.

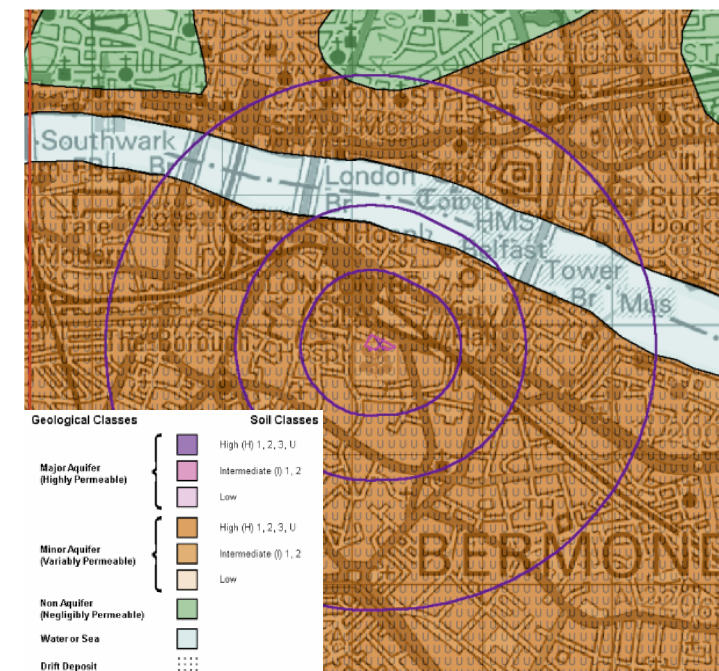


Figure 5.7 Envirocheck Combined Surface Geology map.

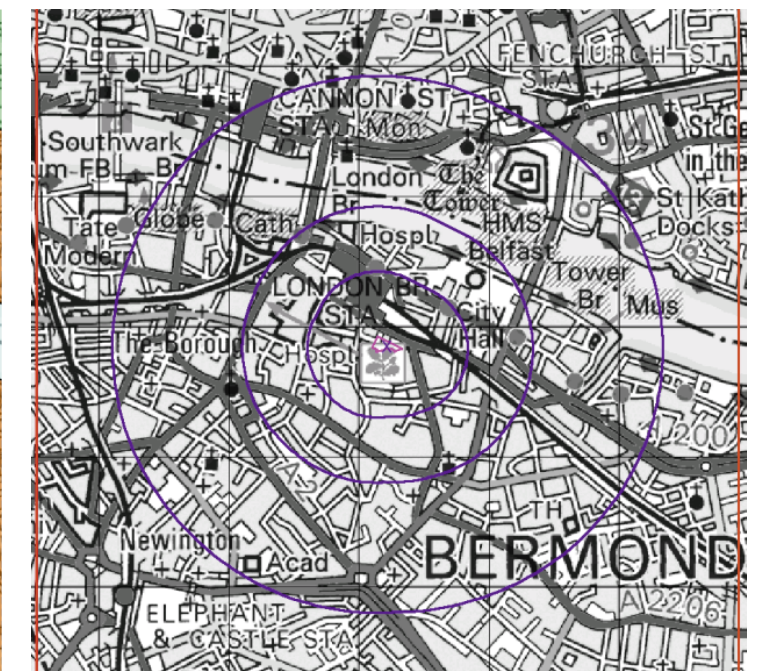


Figure 5.8 Envirocheck Source Protection zone map.

6 Superstructure

This section provides a summary of the above ground structure for the Vinegar Yard development based on the current proposed scheme.

Redevelopment of the site includes the demolition of the existing buildings, excluding the warehouse to be refurbished, and the erection of an eighteen-storey mixed use building, plus ground floor and mezzanine, with 3 basement levels. The maximum height is 97.140m (AOD), with plant at roof level, and there is the retention of the two-storey warehouse, of a maximum height of 17.440m (AOD).

The lower floors of the building above ground level will be used as medical spaces or research and development, mentioned in this document as the clinical floors, whilst the top floors will have an office use. The basement level B1, B1 mezzanine and ground floor mezzanine will accommodate affordable workspace. The ground floor level is where the retail areas and the lobby for the office and clinical floors will be located. The plant areas will be located at the basement levels B2 and B3 and on the top floors of the building. On-floor plant will also exist at levels 3 and 8.

The existing warehouse to the south west of the site is being retained and refurbished to be used as retail and community hall.

The building has a trapezoidal shape on plan and comprises a stepped massing rising from the east of the site. The

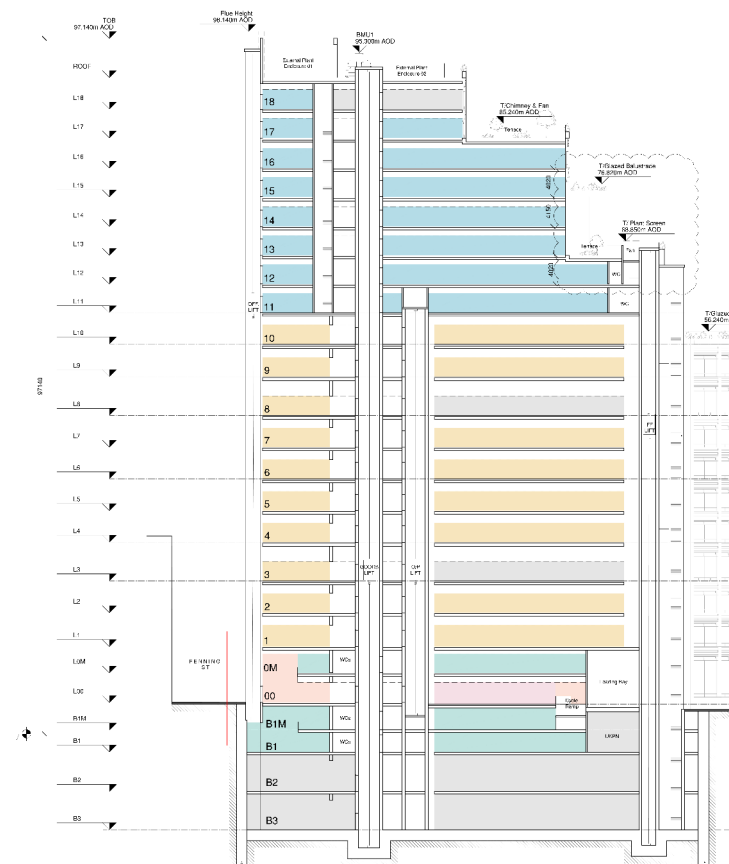


Figure 6.1 Section of the proposed Vinegar Yard development and internal use

superstructure of the building consists of a reinforced concrete (RC) frame designed to allow for future flexibility and adaptability.

6.1 Structural grid

The size of the structural grid directly influences the floor construction, structural zone, slab thickness, column sizes and foundation requirements. Following initial analysis and assessment to accommodate the clinical grids and planning modules, the design has progressed with a primary structural grid of approximately 9.0 m x 7.2 m x 9.0 m in the direction north-south and variable spacing in the direction east-west.

6.2 Stability system

The global stability of the building is provided by the two cores (both eccentric to the floor plan and located to the south of the building) together with the contribution of the in-situ RC frame. The south elevation of those cores is to be made of structural steelwork integrated with the façade. The RC slabs at each level will transfer the wind loads to the cores and columns through diaphragm action. The vertical elements of the building will take lateral support at ground and basement levels through the diaphragm action of the RC floor slabs transferring loads to the retaining walls and foundations.

Concrete walls thickness currently proposed varies between 450mm and 250mm. Wall thickness transition to be optimised in the next stage.

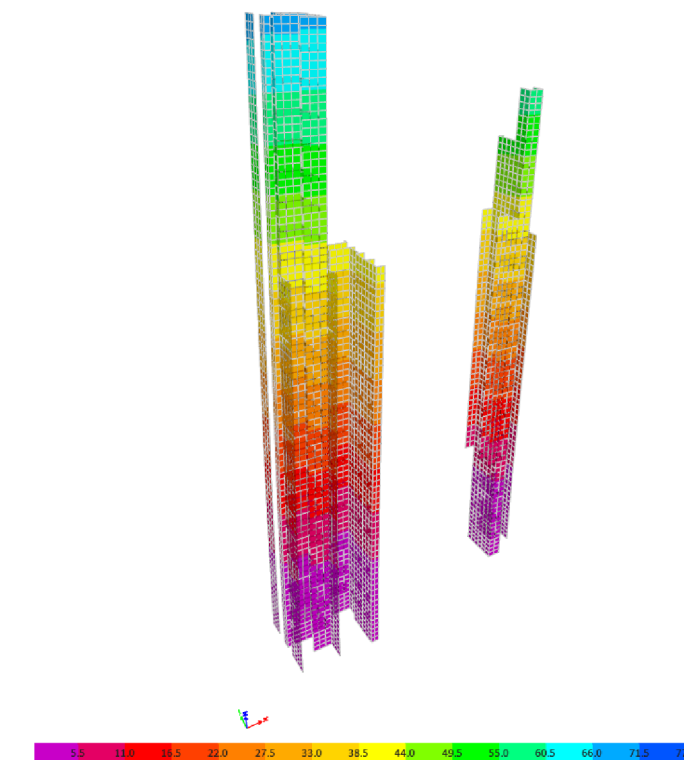


Figure 6.2 Extract from FE model with horizontal displacement of the cores

Analysis undertaken at this stage demonstrates that the total anticipated horizontal deflection of the building at its highest point will be less than the deflection limit of height/750 based on the current arrangement of the cores.

There is also an additional bank of scenic lifts to the west of the building that serves the clinical and office floors. This steel structure will provide vertical support to the floors and support to the lift and façade with no global stability function to the building.

6.3 Typical floor plates

In order to allow for the flexible service distribution a flat soffit construction was set as a constraint which when coupled with the strict vibration limits for the clinical floor dictated that a reinforced concrete flat slab was the optimum solution due to its inherent stiffness and mass.

The clinical floors have been designed for enhanced floor loadings with hard structural zones and to meet strict floor vibration limits. These floors can in the future be converted to offices.

Typically the clinical floors will have RC slabs 375mm thick and the office floors 320mm. It should be noted that there is the opportunity to adopt post-tensioned slabs for the offices floors which will reduce the slab thickness to 275mm.

In order to deal with the long spans along the façade of the building and the winter gardens slab steps, 750 mm deep downstand fabricated steel beams have been introduced to control the slab edge deflections with provision for openings 700 x 400 mm to allow the connection of the services in the ceiling void with the façade louvres and service chimneys.

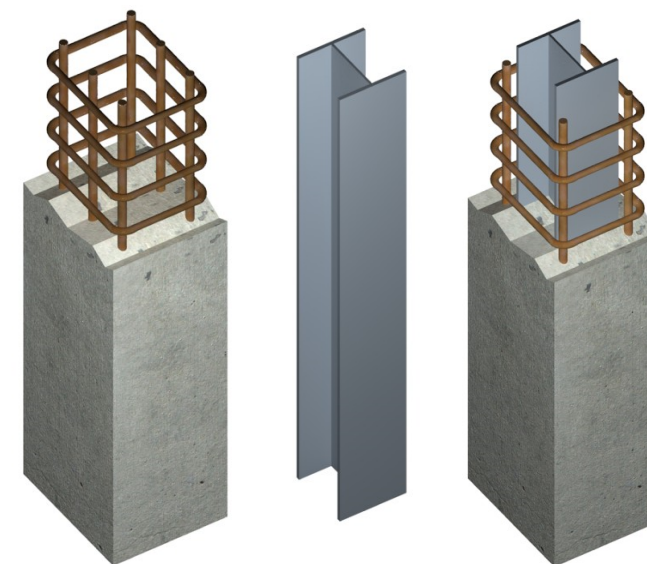


Figure 6.3 Column types a) RC column, b) Steel column, c) Composite column

The composite action between these beams and the slabs will be achieved via the shear studs welded to the top flange of the steel beams.

6.3.1 Upper floors

As the building steps back at the upper levels this provides external terrace areas. In order to accommodate the thicker finishes of the terraces, including a blue roof system, steps in the slab have been introduced.

A few transfer beams have been also adopted at those levels to provide support to the slab edges of the floors above as the building steps.

6.3.2 Columns

Typically the internal columns are RC sections whilst the perimeter columns that will pick up the long span edge beams will be steel columns encased in concrete. The overall dimensions decrease as the columns go up the building.

6.3.3 Floor Dynamic performance

A preliminary assessment of the slab vibration has been carried out for both typical clinical and office floor plates which indicate that response factors (RF) to footfall vibration is likely to be within vibration limit criteria agreed with the Client, RF=2 for the clinical floors and RF=8 for the office floors. It should be noted that for the office floors the vibration analysis has been carried out for the 275mm thick PT slab as this will provide the worst case results for the vibration when compared with the 320mm thick RC slab.

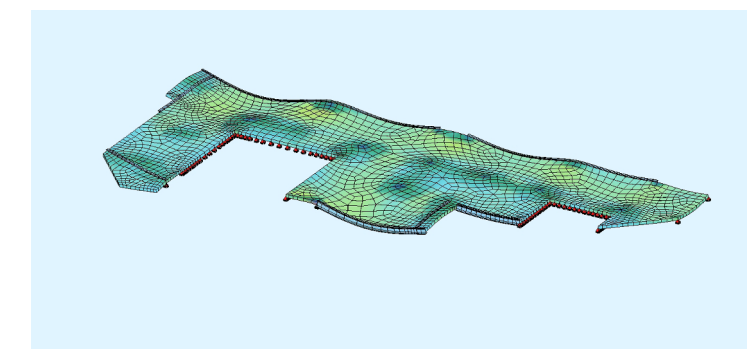


Figure 6.4 FE floor plan model

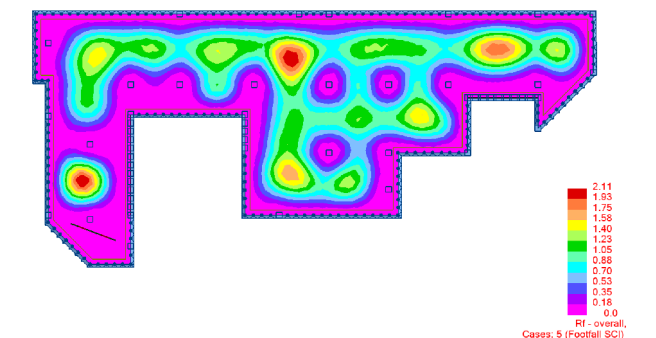


Figure 6.5 Response factor plot for the typical clinical floor

7 Substructure

7.1 Proposed Basement

As part of the Vinegar Yard development, the tower will sit above three levels of basement. The basement depth will be approximately 18 m below ground level, providing space for affordable workspace, back of house areas, cycle stores, and plant.

The development will also include a standalone underground silo to the east of the site, for additional cycle storage. This is a proprietary circular cofferdam of small area which will be formed as a series of sheet piles strengthened by circular wailer beams that retain surrounding earth pressures.

It should be noted that the following information presented is preliminary and only offers indicative solutions to the proposed basement scheme and is therefore subject to design development. Completion of the site investigation will inform the detailed design of the basement and foundations in the subsequent stages.

7.1.1 Slabs and columns

The basement level B1, B1 mezzanine and ground floor mezzanine will accommodate affordable workspace, with the Plant typically located in the levels B2 and B3.

In addition to the vertical loadings associated with the basement uses, the slabs also act as lateral props to the perimeter retaining wall. As such the slabs must be designed to resist these compressive forces as well.

A flat slab solution is preferred as this results in the minimum structural depth. Post-tensioned slabs are more difficult to construct in basements where there is no slab edge from where the tendons would typically be stressed. As a result a traditional reinforced concrete slab has been chosen. It is currently proposed that the basement slab should consist of a 350mm thick reinforced concrete slab typically, potentially increasing to 450 mm thick under areas where the loads are higher. There is no movement joint planned within the ground floor slab and all thermal and shrinkage strains will be dealt with by specifying adequate reinforcement and concrete mixes with cement replacement.

The proposed thicknesses should be review and confirmed once the layout of the basement and the weight of the required services in each area is confirmed and build up requirements defined.

It is anticipated that steel flying shores will be provided to support the excavation in the temporary case, however this is subject to design development with contractor input.

7.1.2 Piled raft

Pile raft foundations take advantage of the capacity of both, raft and piles to resist the combination of vertical, lateral and over turning forces from tall buildings. Piled raft foundations utilize piled support for control of settlements with piles providing most of the stiffness at serviceability loads, and the raft element providing additional capacity at ultimate loading. Consequently, it is generally possible to reduce the required number of piles when the raft provides this additional capacity. In addition, the raft can provide redundancy to the piles by redistributing the load from one weak pile to those surrounding it.

Another feature of piled rafts is that the pressure applied from the raft on to the soil can increase the lateral stress between the underlying piles and the soil, and thus can increase the ultimate load capacity of a pile.

The raft also offers flexibility for column positions. The piles under the raft will be rotary bored piles executed from the ground level down to the Thanet sand.

7.1.3 Perimeter Walls

Several factors influence the form and size of basement retaining walls including:

1. Strength.
2. Movement control - primarily in the temporary condition near the NWR viaduct, Thames Water Sewer and surrounding buildings.
3. Propping requirements.
4. Buildability.
5. Vibration and noise levels during installation.
6. Cost.
7. Thickness of the wall buildup.
8. Water resistance - temporary and permanent.

All these factors have been considered during the preliminary design stage to compare the different construction options for the retaining walls.

Based on the limited ground investigation data, among all options, secant pile hard-firm wall appeared as the favoured solution. The sheet piled wall has been dismissed due to its low axial capacity which would not be sufficient where superstructure columns are supported on the wall. Diaphragm walls have also been considered which provide the required stiffness to limit deflections and work as part of the global basement waterproofing strategy. To this end the treatment of joints becomes critical.

At this stage, perimeter retaining walls are proposed to be rotary bored 1200/1050 @ 1900 secant piled walls, with toe within London Clay at circa -28mOD. Female piles to extend at least to formation level, or 2 m into the clay, whichever is greatest.

The secant wall will offer temporary and permanent support to the surrounding retained ground and existing buildings identified in Section 4 of this report. An initial assessment of the retaining walls has been undertaken with WALLAP. The proposed design will be reviewed in greater detail as deflections limits are developed to suit the various boundary conditions as noted in Section 4 of this report.

The excavation and construction of the new development will result in an unloading and reloading of the existing sub soils which will influence ground movements beyond the site boundary. Amount of heave and settlement needs to be predicted over time to provide assurances that neighbouring properties will not be adversely affected. This will require a detailed site investigation to understand the local site soil profile and properties.

7.1.4 Capping beam

The construction of the capping beam on top of the secant wall will require temporary retaining of the soil. This is to allow for breaking of the piles and the construction of the capping beam.



Figure 7.1 Typical piled raft solution

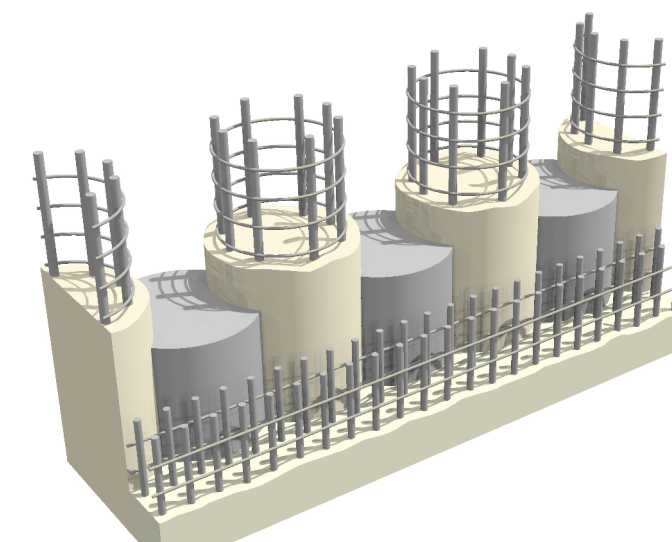


Figure 7.2 Secant wall and liner wall typical construction



Figure 7.3 Site construction example of secant wall

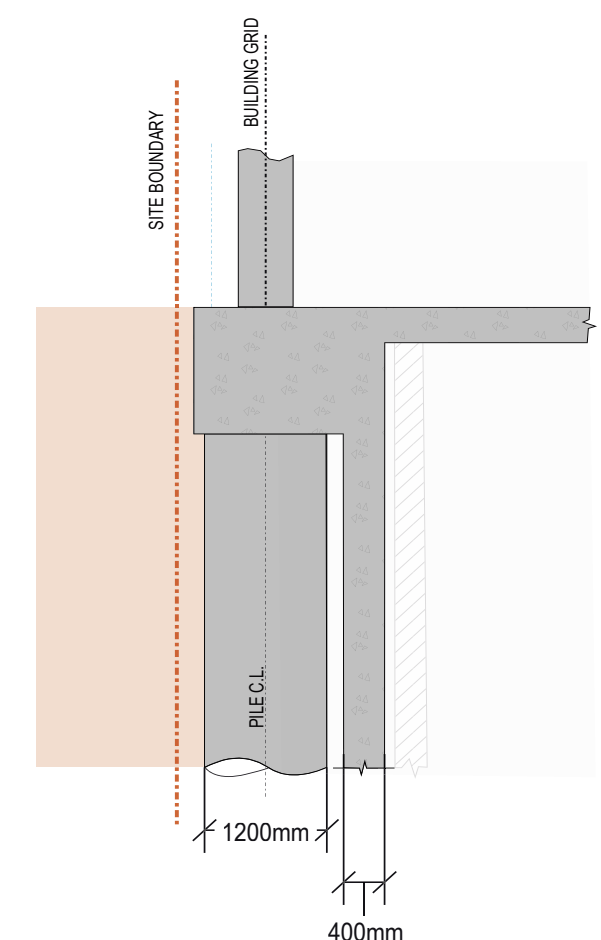
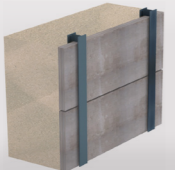

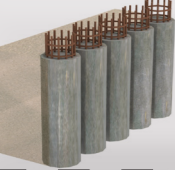
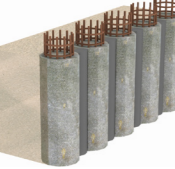
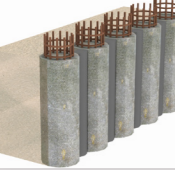
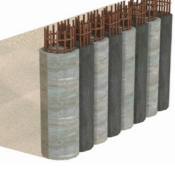




Figure 7.4 Typical secant wall section with capping beam and ground floor diaphragm

7.2 Embedded walls comparative assessment

Options for the basement retaining wall construction are summarised in this section providing a general overview on typical characteristics, advantages/disadvantages, and additional comments. The solutions will be narrowed down as more detailed design is undertaken however, an initial zone has been coordinated to allow the flexibility of all available options. At the present stage, it is thought that a secant piled wall construction would provide an adequate solution due to the depth of the basement and the likely ground conditions anticipated on site. This will need to be confirmed in the next stage of design.

	WALL CONSTRUCTION	TEMPORARY / PERMANENT SUPPORT	TYPICAL WALL DEPTH	TYPICAL RETAINED HEIGHT	USUAL INSTALLATION TOLERANCE: VERTICALITY	ADVANTAGES/DISADVANTAGES	FURTHER REMARKS	Suitability
	King Post Wall: Steel UC soldiers and timber or R.C. (or R.C./P-S.C. + grouting) skin wall/lagging	Usually only temporary support	King posts typically 6 to 20m	3.5 m as cantilever 12 to 15m anchored	1:100	Where good construction tolerances apply, the wall surface may be used as a permanent back shutter to an R.C. wall	Generally only used where groundwater is below formation level. Not feasible in soft and loose soils.	✗
	Steel Sheet Piling	Temporary or permanent support (e.g. in car park basements)	Typically 10 to 15 m. Max pile length: ~30 m	8 to 12 m as single propped wall	1:75	Significant noise and vibration from installation. Vibration and noise can be overcome in some soils by use of hydraulic press equipment.	Re-use of sheet piles will often determine cost viability of temporary sheet piling. Difficulties installing through dense gravels. Likely unfeasible due to known buried obstructions posing a risk of declutching.	✗
	R.C. Piles Contiguous Piles	Temporary and permanent support (where R.C. facing wall is used)	12 to 20 m	6 to 15 m propped or anchored	1:100	Cheapest form of R.C. piles when installed by cfa equipment. Not a water resistant wall	Non-interlocking of piles means little or no water resistance. Can be used with jet grouting to provide permanent water and soil exclusion	?/✓
	R.C. Piles Hard/Soft Secant	Temporary and permanent support, see note regarding durability	12 to 20 m propped or anchored	6 to 15 m	1:100 to 1:200 depending on construction method	The use of a weak concrete mix to allow economical excavation of secant by male piles may also have durability disadvantages long term	May only be considered water resistant in the short term	✓
	R.C. Piles Hard/Firm Secant	Temporary and permanent support, see note regarding durability	12 to 20 m propped or anchored	6 to 15 m	1:100 to 1:200 depending on construction method	The use of a stronger mix for female piles than that used for hard/soft secants may improve water resistance and durability long term	Possible suitable solution due to extent of basement and likely ground conditions to be encountered	✓✓
	R.C. Piles Hard/Hard Secant	Temporary and permanent support, usually permanent	15 to 30 m	10 to 20 m propped or anchored	1:100 to 1:200 depending on construction method	Depth limited by vertical tolerances which influence depth of cut secant joint, and their water resistance. Avoids the use of slurry	Female pile may be reinforced with UB section, male by UB or circular rebar cage. Shear plates may be welded to UBs before insertion for floor connections	✓
	Diaphragm Walls installed by Grab	Permanent (if temporary, then left in place)	15 to 30 m	12 to 25 m propped or anchored	1:200	Heavy installation plant and increasing difficulties in disposal of slurry pose disadvantages	Solution to deep walls in variable soil conditions with water retention. Difficulties may arise with excavation of obstructions, natural or otherwise. The wall surface may serve as the final finished surface for some applications	✗
	Diaphragm Walls installed by Cutter	Permanent (if temporary, then left in place)	15 to 50 m	12 to 35 m propped or anchored	1:400	Improved installation tolerances but minimum job size influenced by large mobilisation and demobilisation costs	Solution to deep walls in variable soil conditions with water retention. Difficulties may arise with excavation of obstructions, natural or otherwise. The wall surface may serve as the final finished surface for some applications	✗

7.3 Excavation and Temporary Works

The excavation and construction of the new development will result in an unloading and reloading of the existing sub soils which will influence ground movements beyond the site boundary. Amount of heave and settlement needs to be predicted over time to provide assurances that neighbouring properties will not be adversely affected. This will require a detailed site investigation to understand the local site soil profile and properties.

The proposed basement is in effect a two/four sided cofferdam, in equilibrium based on the soil and surcharges from one side being balanced out/passively resisted by the other.

Given the shape of the site, it is anticipated that simple cross site flying shores along with steel waling beams will be incorporated to provide the temporary support to the walls at levels similar to the permanent slab positions.

Along with the permanent works retaining wall design, the combined movements of temporary and permanent cases will be limited to achieve the agreed limits on adjacent building CIRIA C760 'damage classification' as well as with Highways and Thames Water. These limits will be detailed in a performance criteria document to be developed in subsequent design stages.

Arguably the most common method of basement construction is the bottom up sequence. This method generally offers a simpler methodology and involves constructing the substructure followed by the superstructure sequentially.

A potential bottom up sequence is illustrated in this chapter which for the purposes of this report this has been simplified. Further details on the sequence and any unique items should be sought in the next stage of design. Given the extension of the basement, site logistics would require some areas to be constructed using a bottom-down sequence.

Three levels of props are proposed at this stage for the deepest areas of excavation. An indicative construction sequence is shown in Figure 7.7 and a plan view of the prop locations is shown in Figure 7.6. The proposed construction sequence is as follows:

- Construction of the piling mat and guide walls at ground floor level
- Execution of secant piles. Male piles are proposed to be 1200mm diameter (1050mm male pile uncased under formation level). Female piles are proposed to be 1200mm to formation level.
- Execution of the internal bored piles down to the Thanet sands
- Excavation and construction of the capping beam
- Dewatering of the soil and excavation below the ground floor level
- Installation of first level of temporary props
- Dewatering of the soil and excavation below the Basement level 1
- Installation of second level of temporary props
- Dewatering of the soil and excavation at mid level between Basement level 2 and level 3
- Installation of third level of temporary props
- Dewatering of the soil and excavation to the formation level of the raft foundation
- Cast the raft foundation at basement level 3
- Cast basement level 2 and removal of third level of temporary props
- Cast Basement level 1 and removal of second level of temporary props
- Cast basement level 1 mezzanine
- Cast ground slab level and removal of first level of temporary props

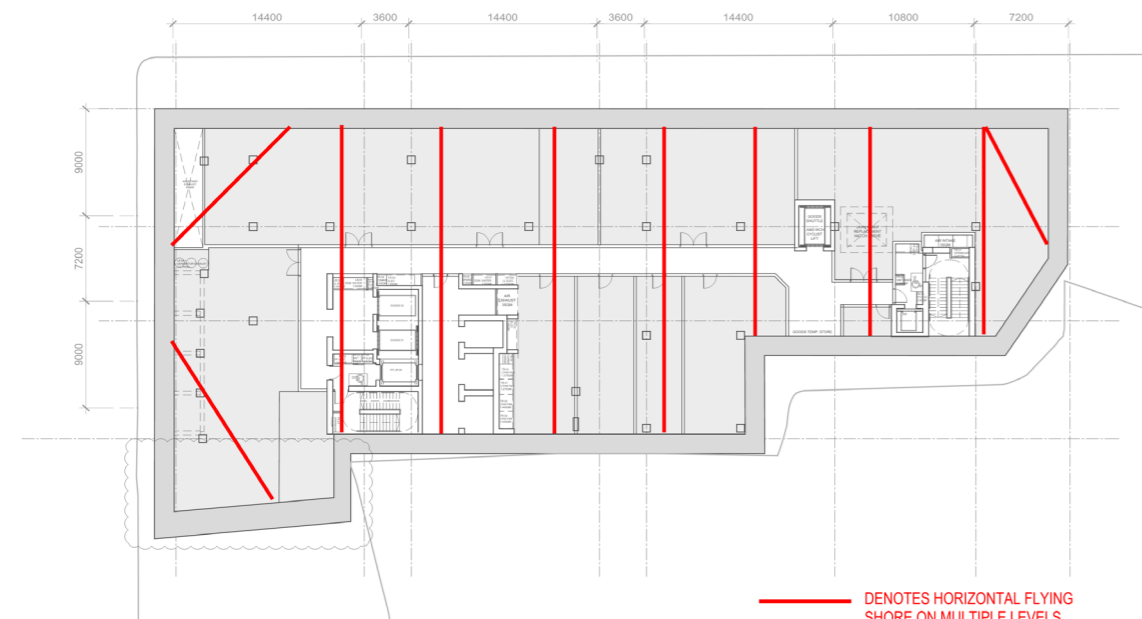
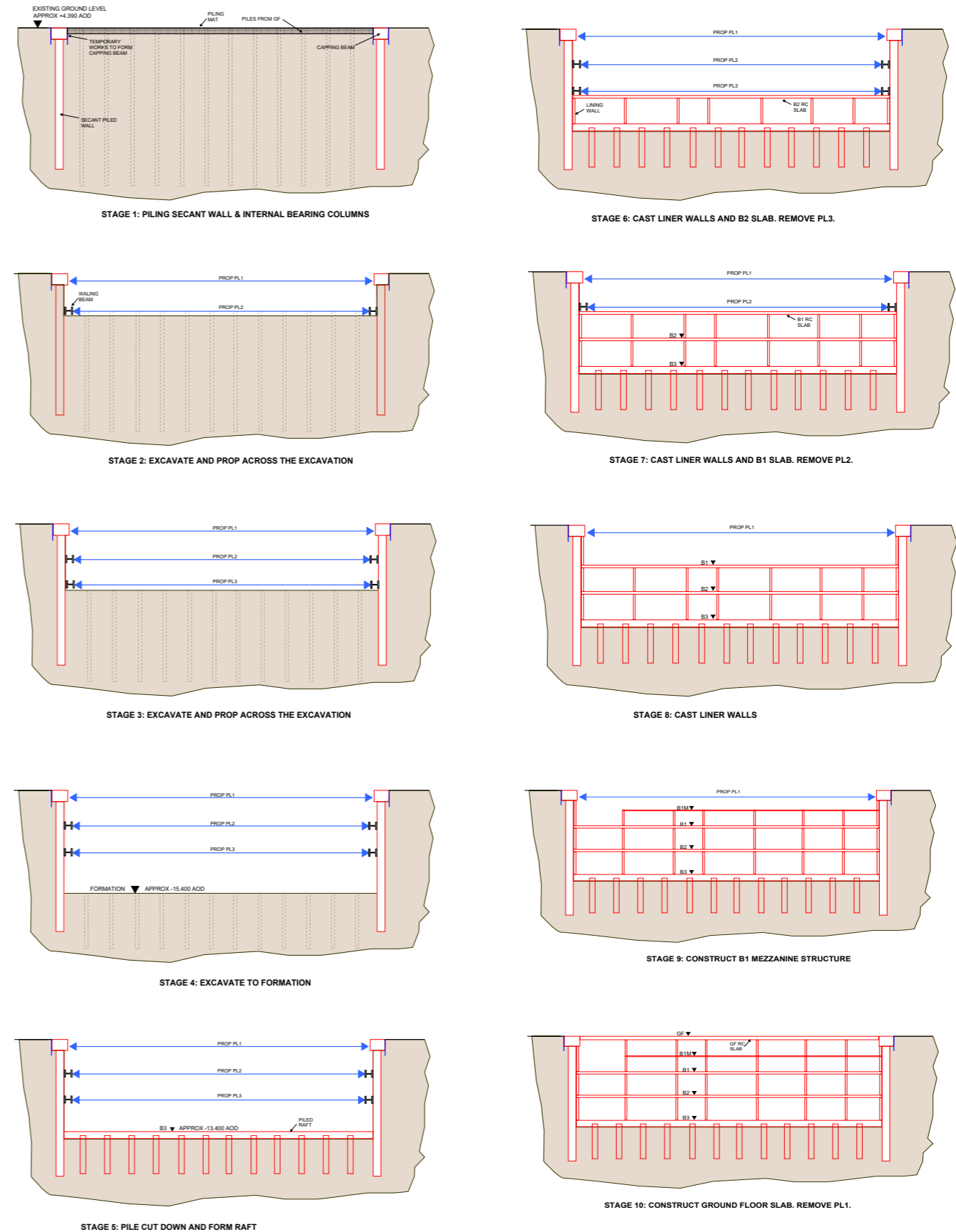


Figure 7.6 Indicative basement propping strategy

Figure 7.7 Indicative basement construction sequence

8 Design standards

8.1 Design standards and guides

The building will be designed in accordance with the following standards. For the sake of brevity, National Annexes are not listed.

- BS EN 1990 - Basis of structural design
- BS EN 1991 - Action on structures
- BS EN 1992 - Design of concrete structure
- BS EN 1993 - Design of steel structures
- BS EN 1994 - Design of composite steel and concrete structure
- BS EN 1997 - Geotechnical design

Additional guidance on geotechnical aspects and issues relating to the basement impact has been taken from the following:

- CIRIA C760 - Guidance on embedded retaining wall design, Gaba et al, CIRIA.
- Concrete Basements - Guidance on the design and construction of in-situ concrete basement structures, Narayan & Goodchild, Concrete Centre.

9 Impact Assessment

9.1 Analysis and Process

The impact assessment will be carried out in stages appropriate to the level of design. At the time of writing, the design is at the concept stage, and there are a number of assumptions that need to be validated before the impact assessment can be concluded.

The following section gives an outline of these assumptions, and the process which will be followed in order to gain the necessary third party approvals.

9.1.1 Data and Assumptions

The key information required to finalise the design can be broken into the following sub-categories. The current assumptions within each category are defined below.

Form and Loads

As the design develops, the massing of the building could be subject to change and this can affect the outcome of any analysis carried out. It is therefore essential that the form of the building is fixed prior to commencement of the approvals processes. Similarly as the use of the building changes, the applied loads will follow accordingly.

Construction Sequence

The construction phasing shall be considered in the assessment of time dependant effects. At this stage, a traditional bottom up sequence is being assumed, however as discussed in previous sections of this report, this is subject to a number of influencing factors, and while unlikely, it is possible that this may change.

Ground Conditions

A full site investigation is required in order to establish the soil parameters necessary to complete a detailed assessment of the ground movements for comparison against relevant acceptance criteria.

Third Party Assets

As dialogue continues with third parties, assumptions relating to location, fabric and condition of adjacent / underground structures may change.

9.1.2 Initial Modelling

Based on the structural solution at planning, initial models will be developed once the form, construction and ground conditions have been finalised post planning. These will be used to assess the potential impact on adjacent structures.

Once a project specific site investigation is carried out, based on the findings, the aim of these analysis models will be to establish the likely magnitude of the impact on any surrounding assets, and provide values which can be used as a basis for initial discussions with third parties as necessary.

The assessment of ground movements will typically comprise of the following analysis types, addressing both vertical and horizontal movements:

- Analysis of the proposed retaining wall considering short and long term conditions, accounting for the consolidation of the clay which is linked to variation in pore water pressures and soil properties. The method and software used shall assess the horizontal movements, forces and moments of the retaining wall during excavation and construction of the proposed development.
- Review of predicted ground movements against empirical derivations and case study data (e.g. CIRIA C760 data). The results will be assessed against relevant acceptance criteria in order to secure the relevant formal approvals for the works to be undertaken. Any resulting requirements with regard to the Contractors' methodology will be detailed and enforced through the project specifications and preliminaries. From these initial calculations and discussions, it may be concluded that the results of the initial modelling are sufficient to gain approvals, or as agreed with third parties whether more detailed modelling is necessary.

9.1.3 Detailed modelling

Where detailed analysis is considered necessary, the initial calculations will be expanded upon through more rigorous analytical processes.

The key elements of this stage will consist of Elastic plane-strain 2D section cut analyses for assessment of lateral and vertical ground movements in regions adjacent to the excavation. A full 3D analysis of the ground may be required in some cases. The requirements of the analysis are varied depending on the approvals process of the asset in consideration, and the scope will be discussed and agreed with the relevant parties as necessary.

Following the outcome of the analysis described in the previous section, and appropriate consultation with third parties, the following aspects will be addressed.

9.2 Impact on adjacent buildings

The impact of the proposed development on the adjacent buildings will be assessed and approvals secured via party wall awards where required.

A package of relevant drawings, calculations and reports shall be prepared for review by the adjacent owners appointed structural engineer and relevant third parties. Assumed temporary works designs shall be prepared prior to the Contractor completing the final design.

A summary of the existing structures in the immediate vicinity of the site (as shown in Figure 4.5):

- **Becket House** lies immediately west of the Vinegar Yard site and properties are separated by Fenning St. The existing property is approximately 8 storeys in height and encompasses an area of land roughly 88 x 27.5m. The structure is likely to be of RC frame construction with possible deep and/or shallow foundations. It is not known if there are any current basement levels. There is a staircase on the NE corner of the proposed site with light wells along the south side of Becket House. There is also a car park and landscaped garden on the southern area of the site.

Recently a planning submission has been submitted to demolish the existing building to enable the construction of a 27 storey tower with 4 levels of basement to a depth of 18m below ground level.

- **The Horseshoe Inn** lies immediately to the south of Vinegar Yard and the properties are separated the pub garden. The existing property is approximately 3 storeys in height and encompasses an area of land approximately 300m². Historical information suggests the building was constructed in 1837 of masonry construction and from visual inspections the structure appears to be well preserved.
- **London Bridge Station** and its concourse dates back to the 1800s and is likely to feature a variety of construction methods from a range of time periods. The most recent known modifications were undertaken as part of the Thameslink Programme which begun in the 2000s. It is known that a section of facade facing St Thomas St was rebuilt.
- Six storey residential apartment building with a masonry facade, which appears to be of recent construction that has replaced a previous car park. It is not known if a basement exists for this development
- Three storey block on the corner of Melior St. of masonry construction
- Four storey industrial/ex-industrial building used as cafe/offices with a masonry facade

9.2.1 Damage criteria assessment

The building damage assessment for adjacent buildings will be carried out in accordance with the methodology proposed in CIRIA C760 for key construction stages; retaining wall installation, excavation, proposed structural loading. This will be subject to party wall and 3rd party agreements determined in the next stage of design.

9.2.2 Survey and monitoring

A regime of surveys and monitoring of the surrounding building, third party assets, proposed/existing retaining walls and adjacent pavements may need to be implemented depending on the results of the movement analysis and the condition of assets.

Appropriate green, amber and red trigger levels shall be set with reference to relevant CIRIA guidance documents on the observational methodology. The scope of monitoring may include the following:

- Movement monitoring of structures in the immediate vicinity via targets surveyed using electronic levels.
- Vibration monitoring using transducers placed on the foundations of the adjacent buildings.
- Crack monitoring via the use of graduated tell-tales.
- Movement monitoring of retaining wall/capping beams via targets surveyed using electronic levels.
- Monitoring of adjacent pavement levels via studs surveyed using electronic levels.
- Monitoring of retaining wall movements via use of Inclinometers cast in secant piles .
- Potential use of extensometer bored in place to monitor heave movements in clay.

Table 6.4 Classification of visible damage to walls (after Burland et al, 1977, Boscardin and Cording, 1989, and Burland, 2001)

Category of damage	Description of typical damage (ease of repair is underlined>	Approximate crack width (mm)	Limiting tensile strain, ϵ_{tm} (%)
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	<u>Fine cracks that can easily be treated during normal decoration.</u> Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection	<1	0.05 to 0.075
2 Slight	<u>Cracks easily filled. Redecoration probably required.</u> Several slight fractures showing inside of building. Cracks are visible externally and <u>some repointing may be required externally</u> to ensure weathertightness. Doors and windows may stick slightly.	<5	0.075 to 0.15
3 Moderate	<u>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.</u> 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	<u>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.</u> 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	<u>This requires a major repair, involving partial or complete rebuilding.</u> Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	Usually >25, but depends on numbers of cracks	

Notes

- 1 In assessing the degree of damage, account must be taken of its location in the building or structure.
- 2 Crack width is only one aspect of damage and should not be used on its own as a direct measure of it.

Figure 9.1 CIRIA C760 damage criteria for varying levels of strain

9.2.3 Utilities approval

Statutory searches have been undertaken as part of the initial desk study carried out. From the findings of these searches, it is expected that the following third party approvals will be required prior to the commencement of construction activities.

Thames Water

Dialogue is to be established with Thames water with regards to assets present in the immediate vicinity of the site with the aim of obtaining an approval. These include the following combined sewers:

- A 306mm diameter combined sewer in Melior Street to South-West of the site.
- A 375mm diameter combined sewer in Snowfields to the East of the site.
- A 1143x762mm combined sewer (which it is believed to be of Victorian masonry construction) becoming 1220x770mm under St Thomas Street to the north-east of the site.

Thames Water approvals would also include the Thames Water sewer and mains that are below the site. Diversions are likely to be required and proposals will need to be developed and submitted to Thames Water for approval.

UKPN, Telecommunications & Gas

A statutory utilities search indicates that there are a number of services in the vicinity of the site which include UKPN, British Telecommunications, and Virgin Media assets and Gas. These are likely to be located beneath the surrounding pavements and will require third party approvals.

9.2.4 Highway approval

The proposed basement construction adjacent to the public roadway along St Thomas Street, Vinegar Yard, Melior Street, and Fenning Street.

An Approval In Principle (AIP) document shall be prepared in accordance with the provisions of the Highways Agency and the London Borough of Southwark. Where appropriate assumptions on temporary works shall be outlined within the AIP. Final methodologies shall be determined by the Contractor who shall be expected to adhere to the specifications of the permanent works.

The Contractor will be expected to liaise with the third parties as necessary to obtain the necessary licenses for temporary works supporting adjacent highway structures.

9.2.5 Groundwater flow

Although the depth of groundwater is unknown at the present time while a specific Site Investigation is pending, the water strikes in near-by borehole records suggest a perched water table within the gravels approximately 3.40m to 4.60m below ground level. Thus the retaining walls will be designed accordingly taking into account the appropriate hydrostatic load as well as accidental loading due to the unlikely event of a burst water main.

It is likely that the London Clay stratum below will also have a hydrostatic profile and this will be interpreted following the site investigation. Once the basement is formed, over time the hydrostatic profile will find its equilibrium and the building as a whole will need to resist any associated up-lift loads. Without site data, we have assumed at this stage that a drainage blanket is not required. It should be noted that on the basis of statutory searches no major aquifer has been identified on the site. The groundwater flows are therefore unlikely to be affected by the proposed development.

9.2.6 Surface water flow

The majority of the site in its current condition is paved or features hardstanding. The area of hardstanding is not expected to increase as a result of the current proposals.

Effects on the surface water flow due to construction of the proposed basement will be assessed in further detail and provided in the independent Flood Risk Assessment report.

9.2.7 Land/slope stability

Based on the provided topographical survey, the eastern side of the site lies at approximately +4.30m AOD with a gentle slope across to the western side to approximately +3.00m AOD. This slope occurs between the furthest ends of the site, over 98.0m. North to south the site remains level with a slope less than 1.0 degree.

Information obtained through statutory searches from the Landmark Information Group have not indicated any known historic land instabilities or geological faults beneath the site.

9.2.8 Local water features

The River Thames is located approximately 400m away from the Vinegar Yard site. There are also no anticipated ponds in the local proximity of the development. Further detail can be found in the Flood Risk Assessment Report.

9.2.9 Flood risk

The development site of Vinegar Yard has been identified as being located in Flood risk zone 3A.

Further detail can be found in the independent Floor Risk Assessment report.

9.2.10 Archaeology

Museum of London Archaeology (MOLA) will be afforded the opportunity to review proposed trial pit locations and propose additional investigations to satisfy their requirements as part of the future site investigation works.

Further targeted investigation (based on the findings of the first round of trial pits) remains an option post-planning. It is also noted that there is the potential for all or part of the demolition, groundworks, and excavation to be undertaken as part of an enabling works package that would afford the opportunity for advanced archaeological investigation or a watching brief during a less programme critical period of the works.

Reference should be made to the independent site specific archaeological documentation included in the relevant submittal for further information.

9.2.11 Construction management plan

Reference is drawn to the draft Construction Management Plan (CMP) included with the submission and the notes related to phasing contained within this report. This will be further developed during subsequent stages of design and planning of the works following discussions with and the appointment of a main contractor.

10 Conclusion

The information presented previously in this document provides an overview of the proposed development of the Vinegar Yard scheme with emphasis on the substructure and the basement impact assessment.

As noted in the previous sections of this report, the framework, design philosophy, and procedures set out above will form the basis for the detailed analysis and assessment works that will subsequently be required to secure the necessary third party approvals prior to commencing works on site.