Victoria Station Upgrade

Supplementary Environmental Statement:

Technical Appendix F – Geotechnical Report

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List of Contents

Page

Summary

The Victoria Station Upgrade project is a major infrastructure enhancement to the existing Victoria Line Underground Station. It comprises an enlargement to the existing underground station ticket hall, the construction of a new ticket hall to the north of the existing underground station beneath Bressenden Place and the construction of a link between the two ticket halls. Additional ancillary works include connections to the existing District and Circle Line Underground Station, as well as tunnels and shafts for station infrastructure. The VSU scheme is part of the wider Victoria Transport Interchange project, a masterplan for the upgrade of the Victoria area of Central London.

This report is an interpretation of the geological, geotechnical and hydrogeological conditions in relation to the proposed works and follows the completion on site of the VSU Additional Ground Investigation in March 2008. This report forms the basis for the derivation of the geotechnical design parameters required for detailed design.

Geo-environmental issues associated with the potential for contamination of the materials to be encountered on site during the proposed works are addressed separately and reported as part of the Environmental Impact Assessment

The main conclusions of the report are as follows:-

The overall sequence at the VSU site comprises Made Ground overlying Superficial Deposits (Alluvium and River Terrace Deposits), which in turn are underlain by the London Clay Formation. The London Clay overlies deposits of the Lambeth Group. The Thanet Sand Formation and chalk is encountered at depth. The River Terrace Deposits can be further sub-divided into modified, i.e. subject to historical ground treatment, and unmodified deposits. The proposed underground works are to be constrained to the London Clay Formation and overlying deposits. This stratigraphic sequence has been presented graphically for several representative sections at the site.

The Made Ground at the site is of variable thickness with its heterogeneous composition reflecting historic development and redevelopment in the vicinity. The Alluvium is a predominantly clay material with localised organic pockets, silt and sand partings, and peat bands up to one metre thick. The Alluvium was absent in a number of boreholes within the site forming a north-east south-west trending tract. This apparent absence is likely to be the result of development in the area.

The River Terrace Deposits were encountered throughout the site and are typically between five and seven metres thick. There is considerable variation in the composition of these deposits throughout the footprint, with a trend of increasing sand and fines content towards the south; this variation is reflected in the corresponding permeabilities, the permeability of the River Terrace Deposits being considerably higher in the northern part of the site..

Geostatistical analyses have been carried out to predict the elevations of the surfaces of the River Terrace Deposits and London Clay together with their corresponding confidence limits. The surface of the River Terrace Deposits varies within a relatively narrow band between 98.5mTD and 100mTD. The surface of the London Clay varies within a narrow range within the footprint of the proposed underground works, generally varying between 93.2mTD and 93.6mTD, locally 93.8mTD. On the basis of the geostatistical analyses the presence any significant depressions or channels in the surface of the London Clay in the vicinity of the VSU site are considered unlikely.

At the VSU site, in common with observations for the Central London generally, only the lower part of the London Clay Formation, i.e. units A2, A3 and B, were encountered. The proposed works are to be undertaken largely within units A3 and B.

The report summarises the insitu and laboratory testing undertaken during the recent and historical ground investigations in the vicinity of the proposed underground works. Parameters for both routine analysis employing linear elastic-perfectly plastic soil models and more sophisticated soil-structure interaction analysis using non-linear elastic soil models have been presented for the materials likely to be encountered at the VSU site. Comparisons have been made with published data and good agreement is observed.

In terms of ground treatment, the trend of decreasing groutability, i.e. D15 values, from north to south across the site has been confirmed by the additional ground investigation; the proportion of sands and fines increasing in a southerly direction. On this basis permeation grouting within the River Terrace Deposits close to the STH is likely to be difficult and probably impractical. With regards to the preferred method of treating the River Terrace Deposits and overlying Superficial Deposits, jet grouting, this variation in the nature of the material will result in variations in the application of the process. As with permeation grouting, jet grouting adjacent to the STH is likely to be more challenging than elsewhere on the site.

The VSU site is located within the course of the former River Tyburn. A minor perched aquifer (the Upper Aquifer) is evident within the River Terrace Deposits. The perched water in this aquifer controls the pore pressure within the upper part of the London Clay. The water level data are consistent with the simple model of flow in the area being controlled by drainage of the District & Circle Line tunnels.

A relatively simple hydrogeological model is proposed which reflects local changes in geology and is suitably cautious. A hydrostatic water table is assumed in the Upper Aquifer, a hydrostatic variation of groundwater pressure with depth in the Upper London Clay and, due to the effects of under-drainage, a sub-hydrostatic increase in water pressure with depth in the Lower London Clay. Following current best practice, both ULS and SLS pore pressure profiles are presented. For the ULS condition, the water table is assumed to be at the ground surface as a result of flooding or a burst high level water main.

A reduced quantity of water being abstracted from the Lower (Chalk) aquifer during the last 20 to 30 years has led to a rise in water table level in the Chalk with associated increases in pore water pressure in overlying strata. Future increases in pore water pressure, perhaps to a full hydrostatic condition, need to be taken into account in the station box design.

1 Introduction

1.1 Background

Mott MacDonald were commissioned by London Underground Limited in December 2006 as MDC2 to develop the design for the Victoria Station Upgrade Project. The proposed upgrading of Victoria Underground Station involves the construction of a new ticket hall to the north of Victoria Street, extensions to the existing ticket hall to the south and connecting passageways to alleviate overcrowding of the existing station and to facilitate provision of step-free access. The VSU scheme is part of the wider Victoria Transport Interchange project, a masterplan for the upgrade of the Victoria area of Central London.

As part of this commission a comprehensive geotechnical desk study was undertaken in order to progress the works to RIBA Stage D Scheme Design.

The main geotechnical issues associated with the proposed works are:

- the construction of approximately 15m deep box structures through the superficial deposits (Made Ground, Alluvium and River Terrace Deposits) and underlying London Clay;
- the construction of a number of tunnels, shafts and escalator barrels at the interface between the London Clay and the overlying River Terrace Deposits;
- the feasibility of treating the River Terrace Deposits (RTD) in order to facilitate construction of the shallow tunnels. The RTD have been subject to relatively unsuccessful ground treatment (permeation grouting) in the past during earlier underground works in the vicinity.
- the construction of a number of tunnels, shafts and escalator barrels through the London Clay.

Proposals for supplementary ground investigations to reduce the project risks associated with uncertainties in the ground and groundwater conditions, and facilitate the detailed design of the temporary and permanent works and the development of robust construction techniques, were also included in the desk study report. These proposals included:

- The formation of boreholes to verify strata thickness and elevations;
- Detailed logging, insitu testing, high quality sampling and laboratory testing to facilitate more informed decisions regarding appropriate ground improvement methods, more cost-effective design and more realistic ground movement predictions.

An additional ground investigation was subsequently procured under the Mott MacDonald Framework Contract for Ground Investigation, with the fieldwork being undertaken between October and December 2007 and February 2008.

Geo-environmental issues associated with the potential for contamination of the materials to be encountered on site during the proposed works are addressed separately and reported as part of the Environmental Impact Assessment .

1.2 Scope of Report

This interpretative report summarises the geological, geotechnical and hydrogeological information relating to the VSU project available to MDC2 at the beginning of May 2008. It has been compiled following the completion of the additional ground investigation at the site in March 2008 and incorporates information from earlier ground investigations undertaken in the vicinity of the proposed underground works in addition to those ground investigations conducted by Norwest Holst Soil Engineering Limited in 2006/2007 for the VSU project.

Specialist laboratory testing to determine the long term deformation characteristics of the London Clay and the potential to carry out ground freezing within the River Terrace Deposits have been scheduled. At the time of writing, these tests were not complete.

This report has been prepared as part of the VSU project by MDC2 for London Underground Limited. It is not intended for and should not be relied upon by any Third Party; no responsibility is undertaken to any Third Party. In preparing this report it has been assumed that the information obtained from previous investigations and studies is accurate and can be relied upon. Whilst MDC2 has carefully reviewed the various information consulted in the preparation of this report, they cannot accept responsibility for any shortcomings that result from data deficiency (except in so far as such deficiencies should have been apparent to an experienced Engineer).

Unless indicated otherwise all levels quoted in this report are to Tunnel Datum. Tunnel Datum is located 100m below Ordnance Datum.

2 Project Description

The proposed upgrading of Victoria Underground Station involves the construction of a new ticket hall to the north of Victoria Street, the Northern Ticket Hall, extensions to the existing ticket hall to the south, the Southern Ticket Hall. and connecting passageways, the Paid Area Link, to alleviate overcrowding of the existing underground station and facilitate the provision of step-free access.

(i) Northern Ticket Hall

Situated beneath Bressenden Place; the NTH is to comprise an approximately 12m deep, two-storey subsurface structure, of maximum length and width of approximately 76m and 33m respectively. Formation level is to be approximately 93.0mTD. The box structure is to be constructed employing top down construction methods following the installation of hard/firm secant piled walls. Large diameter reinforced concrete bored piles are to be constructed beneath the footprint of the box.

(ii) Southern Ticket Hall Extension

The extension to the existing Southern Ticket Hall is to consist of an approximately 11.1m deep, 2-storey subsurface structure located beneath the 'beach' area next to the front entrance to Victoria Mainline Station. The STH extension is approximately triangular in plan with a plan area of approximately 1000m^2. Formation level is to be approximately 93.4mTD.

The STH extension is to be constructed through a combination of bottom-up and top-down techniques and will incorporate construction of the cut-and-cover passageway connection to the westbound District and Circle Line. Secant piled wall, and tension piles and plunge columns where accessible will be installed.

(iii) Paid Area Link tunnels

Construction of the Paid Area Link (PAL) tunnels will primarily provide a passenger route between Northern and Southern Ticket Halls but also an Interchange Concourse enabling access to the Victoria and District & Circle underground lines.

The typically 6.4m outer diameter PAL tunnels are to be constructed at relatively shallow depth at the interface between the River Terrace Deposits and the Underlying London. The tunnels are to be constructed using a phased, compartmentalised approach.

To ensure face stability and groundwater control, tunnel construction is to be undertaken in 10m long watertight compartments. The compartments are to be formed from a two metre thick jet grout canopy around the perimeter of the tunnel extrados within the River Terrace Deposits/Alluvium/Made Ground; full-face jet grouted panels are to be formed within the RTDs at each end of the compartment. The jet grouted canopy and full-face panels are to be toed-in to the underlying London Clay. The watertight compartments are then dewatered before tunnelling commences.

The tunnel sections are to be formed employing the Sprayed Concrete Lining technique with an excavation sequence consisting of a top heading and bench/invert to complete the full excavation section with the final ring closure within two metres, i.e. the face will be advance in one metre increments. Immediately after excavation all exposed ground at the face and around the perimeter will be sealed with shotcrete. In addition pre-support measures, comprising a grouted pipe arch, will be installed ahead of any tunnel excavation.

Probe holes / drain holes will also be employed whereby systematic forward probing will be implemented as part of the tunnelling process. These holes act as drain holes and if required specific drain pipes could be installed.

Although jet grouting appears the most probable ground treatment solution various techniques are being considered, including ground freezing, which may be locally required in certain areas where access problems make jet grouting impractical. The majority of the grouting works are expected to be undertaken from ground level, but may also be conducted from within the basements of selected buildings, where necessary.

(iv) Shafts

Various shafts including the D&C East Bound Personally Restricted Mobility (EB PRM) shaft, the Fire Fighting Shaft, the North Escalator Shaft, and the STH Escalator Shaft are to be constructed. Many of these shafts are inclined and/or have connecting tunnels. The ground treatment, pre-support measures, and the excavation and support sequence to be used are similar to those described for the PAL tunnels above. The shafts range in external diameter from 5.8m to 8.0m and depths from 12.2m to 20.4m.

(v) Other Works

Other works include the D&C West Bound Paid Area Links (WB PAL) Interchange Stair, District & Central (D&C) Underpass, the District & Central (D&C) WB Link Passage and the Victoria Line Overpass. Electrical and Mechanical (E&M) Services Tunnel North and the Fire Fighting Access Tunnel.

3 Site Location and Description of Existing Urban Environment

Victoria Underground Station is situated in Victoria, Central London (grid reference TQ289791) and forms part of a complex transport interchange comprising mainline and underground railway, and local bus stations. Two underground lines pass through Victoria Underground Station: the District & Circle (D&C), and Victoria Lines. The site boundaries to the west, north and east are defined by the following public roads; Buckingham Palace Road, Bressenden Place and Wilton Road. The southern boundary of the site is formed by Victoria mainline railway station. Street level typically varies between 103.7mTD and 104.6mTD in the general vicinity of the proposed works.

The site is located within a congested urban environment. The roads surrounding Victoria mainline and underground stations are busy and although the site lies within the Central London Congestion Charge Zone, the roads around Victoria mainline station are part of a north-west to south-east link principally formed by Grosvenor Place and Vauxhall Bridge Road, which is excluded from congestion charging. Pedestrian traffic is heavy, especially from the Victoria mainline station and along the pavements on the southern side of Victoria Street. Buildings in the area are a combination of both old and new, with and without architectural merit. Two theatres and several public houses are situated in the vicinity of the site. Many small shops are located around the north side of Terminus Place, along Victoria Street and on the western sides of both Bressenden Place and Allington Street.

4 Site History and Regional Geology

4.1 Summary of Site History

Details of the historic development of the site date back to the 1600s when the site was mainly marsh land surrounding a shallow lake. The marsh was subsequently drained and development of the site commenced from the 1640s with the construction of a brewery. The site subsequently underwent progressive development and redevelopment including the construction of a wharf beneath the current District and Central Line running tunnels and station platforms. Development culminated with the construction of Victoria Mainline Station and the District and Circle Underground Line during the nineteenth century. The Victoria underground line in the Victoria area was completed in the late 1960s, with much of the existing road network well established and the area became almost entirely built-up with retail, commercial and residential properties. Some buildings have since been demolished and a small number of prestigious new developments have been constructed.

Various buried infrastructure and utilities traverse the site. This includes the Kings Scholars Pond Sewer which flows approximately from north to south, from Bressenden Place in the north and crossing above the District & Circle Lines in Victoria Street. There are also wells within the VSU site and surrounding area.

4.2 Regional Geology

The regional geology of the Victoria area is shown on the 1:50,000 geological map Sheet 270 South London. The map indicates that the ground conditions at the site comprise superficial deposits (Alluvium and River Terrace Deposits) underlain by the London Clay Formation. Made Ground is present overlying the superficial deposits and the Lambeth Group underlying the London Clay Formation. The sequence of materials shown on the geological map has been confirmed by past ground investigations. The following sections describe the salient features of the Made Ground and geological formations present in the area around Victoria.

4.2.1 Recent Deposits

As noted the site has been subject to urban development since the seventeenth century with much well-established surface and subsurface infrastructure. The majority of the site is underlain by Made Ground, the nature of which will reflect the historic development and redevelopment of the site and surrounding area. The Made Ground mainly comprises reworked sandy clay. The presence of perched groundwater within the Made Ground has also been reported. The entire site is covered with hard surfacing, paving slabs, asphalt and/or concrete (often reinforced). Further details on the site specific distribution and properties of the Made Ground are described in Section 6.

4.2.2 Quaternary deposits

Alluvial deposits were formed on the flood plains and channels of both the River Thames and River Tyburn. The BGS Special Memoir on the Geology of London (Ellison et al., 2004) suggests that in the Central London area, Alluvium typically consists of silty clay and clayey silt, with locally developed beds of fine to coarse grained sand generally less than one metre thick but locally up to four metres thick.

The River Terrace Deposits were deposited between the Anglian and the Devensian stage by the River Thames. The thickness of the River Terrace Deposits at the VSU site is highly variable, and can be divided into two general groups: unmodified and modified. The latter group representing those River Terrace Deposits subject to ground treatment during previous development of the site.

For the unmodified River Terrace Deposits, Thomas (1969) noted that during the formation of the underground cofferdam at Victoria Station, *"a large proportion of the station works were constructed within water bearing granular strata varying over the area between very coarse open gravel and fine silty sand. Generally the fine material predominates in the upper level and the coarse in the lower, but both are extremely variable and pockets of fine material occur at all levels."*

With regards to the modified River Terrace Deposits, it was produced primarily during the construction of the Victoria Line in the 1960s and the Passenger Congestion Relief Works in the early 1990s where permeation grouting of the River Terrace Deposits was undertaken. The grouting was carried out to improve these materials by enhancing their stability and reducing their susceptibility to collapse whist excavation was undertaken within and adjacent to these deposits.

4.2.3 Thames Group

The published geological map indicates that the London Clay formation of the Thames Group forms the top of the solid geology in the area under consideration. The formation consists of dark bluish to brownish grey fissured clay, containing variable amounts of fine-grained sand and silt; with clays weathering to a chocolate brown colour and more sandy beds to orange. The BGS memoir for London and the Thames Valley describes minor constituents of the London Clay to include calcareous and phosphatic nodules, barite, siderite, glauconite and pyrite. Beds of calcareous 'cementstone' concretions, up to 0.4m in diameter occur throughout the London Clay.

Five major transgressive-regressive cycles are recognised within the London Clay Formation (King 1981) and are used to broadly define five main divisions in the London Clay (based on a combination of lithological and bio-stratigraphical data).

The units of the London Clay can be divided into the following successive divisions: A1, A2, A3, B, C, D and E. The divisions are characterised by changes in the proportions of clay, sand and silt. In the Victoria area, only the lower part of the sequence (units A1, A2, A3 and B are generally preserved). These classifications of the London Clay help correlate field descriptions with engineering properties.

Standing and Burland (1999, 2006) proposed further sub-divisions of division A3, namely A3(i) and A3(ii), following investigations into the higher than anticipated volume losses observed during excavation of the Jubilee Line Extension running tunnels at St James Park. These sub-divisions were postulated to delineate the upper region of this division, which was seen to contain distinct silt and sand partings that are of significance for tunnelling projects.

Local deep drift-filled hollows can exist in the surface of the London Clay (Berry, 1979). This type of feature is believed to have formed in the late Quarternary under the prevailing periglacial climatic conditions. On the basis of the available information, there is no evidence of a significant depression in the surface of the London Clay in the vicinity of the VSU site.

4.2.4 Palaeocene Deposits

The Lambeth Group is known to underlie the London Clay over the majority of the London Basin. In the west of London, Ellision et al (2004) indicate that the London Clay is underlain by the Reading Formation, the dominant lithology of which is red, blue- grey and brown mottled clay. Sand beds and lenses occur within the formation and the proportion of these varies considerably and may constitute more than half the formation, dominantly in the basal part. Typically, the Reading Formation is around 12m to 16m thick.

The Upnor Formation is known everywhere to underlie the Reading Formation. Ellison et al (2004) report that the formation consists of variably glaucontic fine to medium grained sand with beds and stringers of well rounded, black flint pebbles. When fresh the sands are dark grey brown to dark green and weather to a pale grey-brown although the glauconite remains dark green. The thickness of the Upnor sands is given as 5 to 6m in central London thinning in a westerly direction.

Engineering works within the Lambeth Group are not anticipated as part of the VSU project.

4.2.5 Underlying Strata

The Lambeth Group is overlies the Thanet Sand Formation, which in turn rests upon the Upper Chalk Formation.

Engineering works within these underlying strata are not anticipated as part of the VSU project.

5 Ground Investigations

5.1 Previous Ground Investigation Reports

Several site investigations have been carried out in the vicinity of the VSU site. These can be summarised as follows:

(a) 1992 Congestion Relief Work, Terresearch sunk three cable tool boreholes (T1 – T3) in 1992 together with a suite of general classification tests as well as strength and consolidation testing. Permeability and chemical testing were also undertaken as part of the site investigation.

(b) Allington House Development, LBH Wembley. In 1995 sunk three cable tool boreholes (AH-BH1 to AH-BH3) and excavated three trial pits (TP1 – TP3) as part of the Allington House Development scheme, located at the junction of Victoria Street and Allington Street. The exploratory work also consisted of penetration resistance tests in the boreholes.

(c) Cardinal Place Development; Foundation & Exploration Services Ltd (FES) formed boreholes (BH1, BH3, BH4, BH7 to BH9, BH11 to BH 14, BH11A and BH13A) up to depths below ground level of 85m using light cable percussion boring technique and twenty trial pits as part of the ground investigation for the Cardinal Place Development in 2001.

(d) The Abford House Re-development; a phased approach was adopted for this ground investigation; the first phase of investigative work was carried out in February 2007and the second phase was carried out in September 2007. Both ground investigations were carried out by Concept Site Investigations. The scope of work included three cable percussion boreholes (ABF-BH1 to ABF-BH3) to a maximum depth of 50.5m, three windowless sampling boreholes (to a maximum depth of about 3.6 metres), nine trial pits (to a maximum depth of approximately 2.2 metres) and eleven diamond rotary concrete cores. Standard penetration resistance testing was undertaken in the boreholes. 50 mm diameter groundwater monitoring piezometers were installed in two of the boreholes with a slotted response zone between 5.5 and 12.0 metres below ground level through the London Clay.

(e) VSU Initial Ground Investigation. As part of the Victoria Station Upgrade Project, Norwest Holst Soil Engineering Limited undertook a ground investigation at the site under the supervision of MDC1 in 2006. The investigation comprised the drilling of seven cable tool boreholes (BH05/01 – BH05/07), selectively extended by rotary drilling to a maximum depth of 40m, as well as nine trial pits. Self-boring pressuremeter tests were undertaken in three of these boreholes. Electric piezocones were carried out in probehole locations P1, P1A, P2, P3A and P6 to a maximum depth of approximately 10m below ground level. Seismic Cone Penetration Tests were also carried out in probe hole P6.

5.2 VSU Recent Ground Investigation Report (2007-2008)

The most recent ground investigation (GI) for the VSU project was carried out by Norwest Holst Soil Engineering Limited between 29th October and 21st December 2007 11th of February 2008 and 3rd March 2008 under the supervision of MDC2. The purpose of this investigation was to further determine the ground and groundwater conditions.

This ground investigation comprised the following:

a) Four cable percussion boreholes (MM05, MM07, MM08, MM09) ranging from 11.50mbgl to 20.0mbgl with associated sampling and in-situ testing. Within MM09, continuous thin wall sampling was undertaken between 12.0 and 19.8mbgl. MM07 and MM08 were formed using conventional light cable percussion techniques, using 150mm diameter temporary steel casings. MM05 was formed with "cut down" Dando 100 cable percussion rig using 150mm temporary steel casings.

b) Five cable percussion and triple tube, Geobore-S wireline rotary cored holes (MM02A, MM02B, MM03, MM04, MM06). The cable percussion technique was adopted from ground level to within the London Clay with associated sampling and in-situ testing. Typically this method was terminated ~1.5m below the surface of the London Clay corresponding to 11.0mbgl to 14.0mbgl. Continuous thin wall sampling was carried out in MM03, MM04 and MM06 from the base of the cable percussion holes. Rotary coring was used to advance the holes thereafter to final depths ranging from 50.0mbgl to 87.0mbgl.

c) Three triple tube rotary cored drill holes (MM10, MM11, MM12) to depths ranging from 4.5mbgl to 6.0mbgl to prove the foundations of Elliot House.

d) Six cable percussion boreholes (CPT01, CPT01A, CPT02, CPT04, CPT07, CPT08, CPT09). Five of the holes were advanced to between 11.3mbgl and 13.0mbgl, while CPT01A was terminated at 1.8mbgl owing to a brick footing. These were starter holes for subsequent CPTs in the London Clay.

e) Eight electric Piezocone Penetration Tests (CPTU's) to a maximum depth of 26.2mbgl or refusal within pre bored holes; CPT01, CPT01A, CPT02, CPT04, CPT07, CPT07A, CPT08 and CPT09. The pre bored holes were drilled to between 11.3 and 13.1mbgl using 200mm and 150mm temporary steel casings. Tests within CPT01 and CPT07 reached unsatisfactory depths, therefore CPT01A and CPT07A were undertaken at a similar position to achieve the specified depth. Excluding CPT01 and CPT07 the minimum termination depth of the cone penetration tests was 20.0mbgl.

f) Self boring pressuremeter testing was carried out in boreholes MM02a and MM02b, within the London Clay in the former and both the London Clay and the Lambeth Group in the latter. After each test the boreholes were advanced using Geobore-S wireline rotary coring.

g) Hand dug observation pits were excavated to a depth of 1.2m at each exploratory hole location.

h) Geotechnical testing on samples recovered

i) Standpipe Piezometer installations within holes MM02A, MM02B, MM03, MM04, MM05, MM06, MM07, MM08 and MM09. The response zone was located within the River Terrace Deposits.

5.3 Laboratory Testing

Both routine index and classification tests and advanced laboratory testing have been carried out on soil samples recovered from the vicinity of the VSU site. The following section describes the geotechnical laboratory testing undertaken during the recent ground investigation:

5.3.1 Victoria Station Upgrade (2007/2008) undertaken by Norwest Holst Soil Engineering Limited:-

a) general index classification tests which included moisture content and Atterberg Limit determinations, and particle size analyses.

b) 28 no. unconsolidated undrained triaxial compression tests without the measurement of pore water pressure using 100mm diameter samples.

c) 20 no. advanced triaxial tests. At the time of writing these tests were not complete. These include 9 no. anisotropically consolidated undrained triaxial tests with small strain and shear wave velocity measurements; and 11 no. anisotropically consolidated drained triaxial tests with small strain and shear wave velocity measurements.

d) 16 suction tests and shear wave velocity determinations on unconfined specimens.

e) Contamination, sulphate and organic testing.

f) Specialist tests to determine the potential to carry out ground freezing within the River Terrace Deposits. At the time of writing these tests were not complete.

6 Ground Conditions

6.1 General

The results of the ground investigations in and around the VSU site generally confirm the regional stratigraphy. The ground conditions comprise Made Ground and superficial deposits (Alluvium and River Terrace Deposits) underlain by the weathered and unweathered London Clay Formation. The surface of London Clay is approximately 93mTD. The interface between Divisions B and A3 of the London Clay Formation is approximately 17m below the surface of the London Clay Formation. Deeper within the London Clay Formation the interface between Divisions A3 and A2 is approximately 30m below the surface of the London Clay. Deep boreholes obtained from various site investigations have also proved the thickness of the London Clay to be approximately 40m. Beneath the London Clay, a Lambeth Group layer (dominated by clay but with some sand and silt towards the base) of approximately 10m – 11.2m thick was proven. The Lambeth Group is underlain by the Chalk.

6.2 Made Ground

All exploratory holes encountered Made Ground of varying thickness. The constituents that make up the Made Ground typically reflect the historic development and redevelopment of the VSU site. Currently, the entire site is covered with hard surfacing, paving slabs, asphalt and/or concrete. The thickness of this hard surfacing varies between 0.15 and 1.0m.

Typical description:	The Made Ground encountered was very heterogeneous. Descriptions range from grey to dark grey; grey-brown, red-brown, orange-brown; very light to dark brown, black, as well as occasionally yellow sandy GRAVEL (sometimes silty) and gravelly (sometimes silty/slightly clayey) SAND to soft to firm brown to dark brown and dark grey/black slightly to very sandy gravely (occasionally slightly gravely) often organic rich CLAY The sand is described as fine to coarse. The gravel is described as angular to sub-rounded fine to coarse of yellow and red brick, concrete, asphalt, sandstone, shale, flint, bitumen, metal, wood, clay pipe fragments, leather, clinker, coal and ash and occasional angular cobbles of concrete and brick. The 1992 Victoria Station Passenger Congestion Relief boreholes describe the granular Made Ground as loose. In the recent and previous GIs, the Made Ground is typically more granular closer to the surface, with organic rich, predominantly cohesive material occasionally occurring towards the base of the unit. As such, the interface between the Made Ground and the underlying Alluvium is not certain across the entire site.

	Cone penetration testing was undertaken at the junction of Wilton Road and Victoria Street during the 2006 VSU ground investigation. One CPT (P2) located immediately adjacent to the District & Circle Line retaining walls, encountered 2 metres of un-reinforced concrete near the surface, which had to be cored prior to completing the test.
	During the recent GI, concrete was first encountered near the ground surface no deeper than 0.75mbgl and no thicker than 0.3m, within exploration holes including: MM10 and MM11 within Elliot House; MM06 just north of STH, and CPT01 southwest of The Stag Public House.
Thickness:	0.4m (FES-BH4) – 5.4m (MM04)
	Typically thicknesses within the footprint of the site works are between 3.0m to 5.0m thick, with the base of the Made Ground occurring at 99.5mTD to 101.5mTD. East of the proposed works, the ground level is generally at a lower elevation and the associated thickness of Made Ground also tends to be lower with thicknesses commonly less than 1.0m.
	The thickest layer of Made Ground was encountered immediately south of the Victoria Palace Theatre (MM04). However based on other borehole data, the thicknesses of Made Ground are anticipated to be greatest immediately south of the proposed Southern Ticket Hall.
	In the vicinity of the former Grosvenor Canal and Pimlico Wharf, boreholes MM07 and BH05/07 indicate thicknesses of 3.4m and 2.2m respectively.
	Recorded cohesive deposits are 0.45m to 0.6m thick. The thickest cohesive deposits, 2.65m and 2.8m thick, were recorded within MM08 and MM04 at relatively low elevations and may incorporate underlying Alluvium.
Extent of Stratum:	Encountered in all boreholes
Stratigraphical Relationship:	Overlying Alluvium and/or River Terrace Deposits

6.3 Alluvium

Typical description:	Materials recovered from the boreholes have been described as dark or very dark grey, blueish grey, brown or dark brown, red-brown, black, green and blue; black mottled grey, grey mottled dark brown or grey mottled black. The material type is dominantly clay with variable proportions of sand from “slightly” to “very”. The sand is described as fine to medium grained. Most typically the deposit is “slightly” sandy clay, although there is a trend whereby the “Alluvium” becomes more granular with depth. An example is in MM06. From 1.5mbgl to 2.5mbgl the material is “silty very peaty CLAY”, from 2.5mbgl to 3.0mbgl it becomes sandy and from 3.50mbgl it becomes very sandy. Localised silty sandy and organic pockets and partings are also occasionally recorded. The presence of rare or occasional flint gravel of fine size, which can be rounded to angular are sometimes recorded. The consistency of the Alluvium is recorded as soft, firm, or soft to firm. Traces of decayed organic matter including possible wood, roots and reed stems were also noted, in addition to occasional organic odours. Notably layers of spongy fibrous PEAT of thicknesses between 0.8m and 1.0m are encountered between 101.3mTD and 100.5mTD in BH05/03, and 100.4mTD and 96.6mTD in MM09. These boreholes are located in the northern part of the site. Rare limonite staining was recorded between 99.2mTD and 101.5mTD within MM02B.
Thickness:	Where present: 0.5m (AH-BH2) – 7.0m (FES-BH12).
Extent of Stratum:	Encountered in the majority of boreholes. Those boreholes whereby Alluvium was not recorded were part of the Cardinal Place Site Investigation and lie within the north-western part of the site (north-west of the proposed NTH). However there are several boreholes from the most recent VSU GI; MM02B, MM04, MM05 and MM07, which indicate a north-east to south-west tract crossing the site where Alluvium was not identified.
Stratigraphical Relationship:	Underlies Made Ground and overlies modified/unmodified River Terrace Deposits.

6.4 River Terrace Deposits

6.4.1 Unmodified River Terrace Deposits

Typical description: Colour descriptions range from brown, light brown, brown and red-brown, orange-brown or yellow. The unmodified River Terrace Deposits are typically medium dense, occasionally dense and very dense, although loose deposits were encountered in BH05/01, BH05/06 and BH05/07 at the northern and southern ends of the site. The materials were variously described as silty and slightly clayey slightly gravelly SAND; gravelly fine to coarse SAND; and sandy to very sandy GRAVEL.

Thickness: The material's thickness may vary between 1.5m (CPT08) and 7.7m (T3), although the majority of Terrace Deposits are between 5m to 7m thick.

Extent of Stratum: Encountered in all boreholes.

Stratigraphical Relationship: Underlies Made Ground and/or Alluvium and overlies the London Clay formation.

6.4.2 Modified River Terrace Deposits

The properties of the River Terrace Deposits have been altered by historic activities, in particular by the District and Circle Underground works. Such works are comprehensively covered in the Geotechnical Desk Study. Terrace Deposits which have been affected by permeation grouting are discussed below.

Typical description: In exploratory hole T3, sunk during the site investigation for the 1992 Passenger Congestion Relief Works, layers of weakly cemented granular material were encountered between 5.9 and 11.0 metres below ground level (98.4 to 93.1mTD). It is noted that exploratory hole T3 was located in the area grouted during construction of the passenger tunnels between the Victoria Line Interchange concourse and the District & Circle Line platforms.

All three CPTs that were undertaken for the VSU project at the junction of Wilton Road and Victoria Street encountered obstructions within the River Terrace Deposits. Two of the probes were located in ground previously treated by permeation grouting during the construction of the Victoria Line Interchange concourse. In addition, probe P3 was located directly above the Track Drain Diversion Tunnel that runs through the River Terrace Deposits. It is considered likely that these CPTs terminated in the modified River Terrace Deposits.

Thickness:	The thickness of modified Terrace Deposits is likely to be the whole layer in treated areas, but may be quite variable around the boundaries of these areas due to local grout migration.
Stratigraphical Relationship:	Underlies Made Ground and/or Alluvium and overlies the London Clay formation, but may be quite variable around the boundaries of these areas due to local grout migration.

6.5 London Clay Formation

Most boreholes that were installed in the site investigations have reached the London Clay Formation. According to King (1981) the lithology and depositional sequences of the London Clay Formation in the London area are divided into divisions: A1, A2, A3, B, C, D and E. These divisions (characterised by changes in the proportions of clay, sand and silt) can be identified by, amongst other techniques, careful visual logging of the material or from the analysis of natural moisture content profiles. In Central London, only the lower part of the sequence, units A1, A2, A3 and B are generally preserved. The VSU site investigation (1996/1997) and the most recent VSU site investigation (2007/2008) were the only investigations, of those summarised in Section 5 of this report, to use the soil descriptions for London Clay as defined by King (1981). For other investigations, the occurrence and lithology unit for the London Clay have been inferred from the available material descriptions on the borehole logs.

Typical description:	Typically described as stiff to very stiff fissured thinly to very thinly laminated dark brown mottled grey/dark grey slightly sandy clay. The sand is generally fine grained and often present as partings or laminations. Fissures are described as random and sub-horizontally orientated, closely to extremely closely spaced, planar and stepped, smooth (occasionally polished), rough and undulating, sometimes planar and infilled with fine sand and rare black staining. The formation contains occasional beds of mudstone and moderately weak to strong grey claystone. Calcite veins may also be present within the formation.
Thickness (all grades):	44.2 m (FES-BH11A) – 46.0m (FES-BH1)
Elevation of base:	47.8 mTD (FES-BH1) – 51.3 mTD (ABF-BH3)
Stratigraphical Relationship:	Underlies River Terrace Deposits and overlies the Lambeth Group formation.

6.5.1 Sub–Divisions B1 & B2

London Clay Formation Sub-Division B typically forms the surface level of the London Clay, which varies between 92.6 mTD and 93.7 mTD.

Typical description:	Sub-division B2 was described as very stiff indistinctly thinly locally thickly laminated fissured grey clay, occasional pyrite nodules; London Clay Formation Sub-Division B1 was described as very stiff fissured clay with occasional partings of silt.

Extent of Stratum:	Some borehole logs not detailed enough to allow positive identification. However, geotechnical data generally indicates layer is present throughout the area.
Stratigraphical Relationship:	Underlies River Terrace Deposits and overlies London Clay A3 Sub-division. Sub-division B2 overlies B1.

6.5.2 Sub-Divisions A2 & A3

Typical description:	London Clay Formation Sub-Division A3 was described as very stiff thinly and thickly laminated indistinctly fissured grey clay with occasional grey silt partings; sub-division A2 was described as very stiff, indistinctly locally thickly laminated indistinctly fissured grey clay with occasional silt partings, becoming glauconitic towards the base.
Extent of Stratum:	Some borehole logs not detailed enough to allow positive identification. However, geotechnical data generally indicates this layer is present throughout the area.
Stratigraphical Relationship:	Underlies London Clay sub-division B1 and overlies Lambeth Group Formation. Sub-division A3 overlies A2.

6.5.3 Claystones

Claystones occur as sub-horizontal bands within the London Clay. It should be noted that the identification of claystones in the holes is dependent on the type of drilling technique used. The continuously sampled triple tube rotary drill holes give the most accurate indication of the depths, thicknesses and nature of the claystone layers. In cable percussion boreholes claystone bands can be identified if they are strong enough to require the use of chiselling to advance the hole. Weathered, weak or intermittent bands will only be identified if sampled and due to the non continuous nature of sampling in these holes, some layers may not have been detected. In addition, due to the method of advancing the holes, the thickness and nature of the bands may also not be accurately determined in the cable percussion boreholes even when sampled.

In the recent GI, claystone layers were encountered in the majority of boreholes; MM02A, MM02B, MM03 and MM05. These vary in thickness from 7cm to 44cm with an average of about 20cm, and occur between 62mTD to 90mTD. The strength is reported as weak to strong, although most commonly moderately strong. Where there is more than one claystone in hole there is a tendency for them to be closely spaced together; for example in MM03, two claystones were encountered between 90.6mTD and 89.8mTD. There does not appear to be an area, or specific elevation, where claystones are more prone to occur. These results do not suggest anything surprising compared with the previous GIs.

6.5.4 Relic Periglacial Features

A review of irregularities of relic periglacial features demonstrates that “pingos” are considered very unlikely to occur.

6.6 Lambeth Group

Typical description:	The Reading Formation beds typically comprise stiff mottled silty CLAY. Colours range from grey blues, brown to red. Layers of thinly laminated clay up to 0.5m are reported in MM02A with fine brown sand lenses and shells. Closely spaced, randomly orientated fissures are occasionally reported, as is calcrete. The Upnor Sands Formation is commonly described as green mottled orange brown slightly silty, gravelley fine to medium sand. The gravel is subrounded to round, fine to coarse, brown and black flint.
Extent of Stratum:	Expected to underlie London Clay across the site although only encountered in a limited number of boreholes which were advanced deep enough.
Thickness:	10.3m (FES-BH1) to 11.3m (MM02A)

Stratigraphical Relationship: Overlying the Upper Chalk Formation and underlying the London Clay Formation.

6.7 Interface Levels

Statistical analysis of the elevation levels for the surfaces of the River Terrace Deposits and the London Clay were conducted using Geostokos software. The results will be discussed within Section 9 of this report.

The main principle of the Geostokos analysis was to fit a statistically derived contour plot for the chosen interface, using elevation data from existing exploration holes. Unlike conventional kriging techniques, where a distance relationship is arbitrally selected for estimating values, Geostokos is used to derive a distance relationship using actual values of the data set. Such a technique enables a statistical confidence to be given with the final contour plot. It must be noted that the interface values do not necessarily plot on the fitted contour surface.

7 Groundwater Conditions

7.1 Existing Conditions

The VSU site is located within the course of the former River Tyburn. A layer of soft alluvial clay overlying the River Terrace Deposits (likely to be associated with the former river) is overlain by Made Ground. Although it is possible that water could exist within the Made Ground, perched on the relatively less permeable Alluvium, the ground investigation works have suggested this is unlikely. The River Terrace Deposit layers contain a few metres depth of water perched on the underlying London Clay, with the upper part of the gravels typically unsaturated. The perched water in the RTDs forms a minor perched aquifer, which is often referred to as the Upper Aquifer (as opposed to the Lower Aquifer at depth within the Chalk).

Ground investigation works have typically comprised standpipe monitoring wells and piezometers (standpipe, pneumatic and electronic types). For the previous GIs these were generally installed in the London Clay at a maximum depth of 48m below ground level. For the recent GI, slotted standpipe piezometers were installed to the base of the River Terrace Deposits and occasionally just into the top of the London Clay Formation. The observed water level varies between 96mTD and 97mTD. These groundwater data cover a limited monitoring period and therefore may not provide sufficient information on any seasonal fluctuations. In addition, the monitoring instruments may not have reached equilibrium with the water pressures in the ground by the end of the monitoring period.

Critically the water level data are consistent with the simple model of flow in the area being controlled by drainage of District and Circle Line tunnels in the Victoria area.

The data also indicate that there is no significant flow to the River Thames and there is only limited influence from rainfall recharge. CIRIA Report 129 suggests that long term data trends in the upper aquifer show no great changes in the past and are consistent with the modelling for the lower Tyburn catchment presented in the report. The main inflow is leakage from water mains and the main outflow is pumping from the existing pumping station at Victoria Underground Station. As such, any change in leakage is likely to result in a change in the pumping quantity rather than having a significant change in water level.

The head of water (or equivalent) measured at a piezometer/standpipe versus piezometer tip elevation or base of standpipe response zone this indicates a relatively complex variation of pore water pressure with depth, as follows:

i. Between 65mTD and 97mTD, hydrostatic variation of pore water pressure with depth;

ii. Below 65mTD sub-hydrostatic variation of pore water pressure with depth exists, due to the influence of underdrainage into the underlying chalk.

The perched water in the Upper Aquifer controls the pore pressure within the upper part of the London Clay. Typically, this is a hydrostatic pressure distribution. Downward drainage of water through the lower London Clay would reduce the water pressures to below hydrostatic. This is evident from deep piezometer data taken from the Cardinal Place site investigation. Comparing the test data obtained at 66mTD and 51mTD, the pressure head difference was only 0.8m.

7.2 Hydrogeological Model

In the absence of vertical flow, the pore water pressure increase with depth will be hydrostatic. In the London Basin, the pore pressure distribution is more complex due to the influence of the upper (River Terrace Deposits/Made Ground) and lower (Chalk and/or Thanet Sands) aquifers, and the relative permeability changes with depth, particularly towards the base of the London Clay.

For design purposes, a relatively simple hydrogeological model is required which can reflect local changes in geology and is suitably cautious. The following model is proposed for the existing groundwater regime, and with some adjustment the groundwater regime to be assumed during construction.

i. A hydrostatic water table in the Upper Aquifer of the River Terrace Deposits.

ii. Upper London Clay – hydrostatic variation of groundwater pressure with depth.

iii. Lower London Clay – due to the effects of under drainage the increase in water pressure with depth is below hydrostatic.

7.3 Potential Long Term Changes in Groundwater Regime

Reductions in abstractions of water from the Chalk aquifer by industry during the last 20 to 30 years have led to a rise in water table level in the Chalk. This has caused associated increases in pore water pressure in overlying strata. Future increases in pore water pressure, perhaps to a full hydrostatic condition need to be taken into account in the station box design, especially for the following:

- long term reductions in shaft and base resistance of piles
- heave displacements due to reductions in effective stress associated with an increase in pore water pressure

8 Geotechnical Properties

8.1 Classification Tests

In this section the available classification test data, including the results of bulk density, natural moisture content, Atterberg limits and particle size distributions, are summarised and briefly discussed for the superficial deposits, River Terrace Deposits and the London Clay Formation. The superficial deposits include the Made Ground and Alluvium. The data has been sourced from ground investigations carried out at the site and those recent investigations undertaken in the near vicinity. The data obtained from the most recent (March 2008) Norwest Holst ground investigation have been indicated in the figures where appropriate. No obvious differences between the results of the previous and most recent ground investigations are observed.

8.1.1 Bulk Density

Fifteen samples of the Alluvium were tested and three samples of Made Ground were tested. There is a significant variation in the bulk density reported for the Alluvium. This variation is attributed to localised sandy and organic pockets. Traces of decayed organic matter including possible wood, roots and reed stems have also been noted in previous ground investigations.

Only a limited number of bulk density determinations were made on samples of the River Terrace Deposits. On the basis of these test results the bulk density is relatively constant with depth and ranges from 1.83 Mg/m^3 to1.93Mg/m^3, however they may be significantly affected by sampling disturbance.

This indicates that the bulk density of the London Clay is relatively constant with depth, with a mean value of approximately 2.0Mg/m^3.

8.1.2 Natural Moisture Content

There is widespread scatter in the results of the natural moisture content for the Made Ground and Alluvium from 18% to 60% (average 38%) and 20% to 100% (average 50%) respectively. The higher moisture contents recorded in the Alluvium can be attributed to increased proportion of organic material. The results of the moisture content tests undertaken on the London Clay samples are much more closely grouped; there are well-defined trends throughout the depth of the London Clay indicating the range within which most of the data lies. As described by Hight (2003) the variation in natural moisture content can be used to identify the boundaries between the various lithological units of the London Clay. As a result of erosion, only the lower units of the London Clay Formation, Sub-divisions B, A3 and A2 are present. Unit B extends from the surface of the London Clay to a level of approximately 76mTD and is underlain by unit A3 which extends between approximately 76mTD and 62mTD. Unit A3 is underlain by unit A2 which extends from approximately 62mTD to the base of the London Clay at approximately 50mTD.

8.1.3 Atterberg Limits

It is observed that the plastic and liquid limits of Made Ground are scattered between 18% to 37% (average 26%) and 28% to 86% (average 54%) respectively. The liquid and plastic limits of Alluvium are range between 31% to 134% (average 75%) and 14% to 102% (average 32%) respectively, showing a wider range and greater average values than Made Ground.

The observed trends within the London Clay are consistent with the lithological sub-units identified from the natural moisture content data.

The plasticity indices for the Made Ground and Alluvium tend to vary between 18% and 60% although Alluvium typically has a higher average (42%) than Made Ground (29%). Five Alluvium samples exceed 60%, probably indicating the presence of significant organic material.

The plasticity index of the London Clay is consistent with published data and the proposed lithological boundaries identified in this report.

The variability of the Made Ground is reflected by a large variation for the plotted values ranging from low to very high plasticity and plotting above and below the M-line (behaving as clay or silt). The results for the Alluvium are also variable although a clay of intermediate to high plasticity is suggested by the data. The London Clay is generally classified as a highly to very highly plastic clay.

8.1.4 Particle Size Distribution

(i) Made Ground

Particle size distribution for the Made Ground is observed to be quite heterogenous although predominantly granular. Most samples comprise approximately equal proportions of sand and gravel (35% or more) with a minor proportion of fines (less than 30%). Samples retrieved close to the ground surface tend to give higher proportions of sands and gravels.

(ii) Alluvium

Four PSD test results are available in the Alluvium, showing it to contain variable proportion of sands (15% to 80%) and fines (20 to 85%), of which the proportions of clay exceeds that of silt (Figure 8.12). The typical test result is generally consistent with the common Alluvium logging description of “slightly sandy clay”.

(iii) River Terrace Deposits

PSD results were recorded in the vicinity of the North Ticket Hall (NTH), Paid Area Links (PAL) and South Ticket Hall (STH) respectively. There is notable variation between the three areas, with a trend of increasing sand and fines content towards the south;

- NTH; Results show a varying proportion of sands (15% to 60%) and gravels (40% to 85%), with some outlying tests showing a greater percentage sand. There are typically less than 7% fines, with the majority of the data grouped around a mean of 35% sand and 65% gravel. However the majority of PSD test results assigned as part of the NTH were from the Cardinal Place Site Investigation, which was undertaken a considerable distance (up to 200m) from the perimeter of the proposed NTH.

- PAL; Results generally show a greater proportion of sand than observed in the vicinity of the NTH (20% to 100%). The data shows two distinct trends, the first with a mean similar to the NTH (35% sand, 65% gravel) and a second, more poorly (uniformly) graded, showing 55% to 95% medium sand, about 5% course sand and less than 40% gravel. There are typically less than about 7% fines, although 6no. tests show a greater fines content, up to 30%.

- STH; Results are similar to the PAL plot, with two distinct trends observed. 7no. tests show a well graded trend with around 35% sand, 65% gravel whilst the remainder indicate a more poorly graded medium sand with less than 20% gravel. As observed at other locations there are typically less than 7% fines, although 9no. tests show a greater fines content up to around 50%.

D60 values for the NTH are typically around 5mm to 20mm, whereas for the PAL and STH the values of D60 are grouped either around 10mm or 0.4mm. These observations are consistent with an increasing proportion of poorly graded medium sand south of the vicinity of the NTH. It is also apparent that the River Terrace Deposit samples retrieved and tested from within 4m below the ground surface are from the NTH vicinity. This is because the ground surface level is generally lower in this area of the site.

D10 values indicate an increasing fine sand content towards the south, with the typical diameter in the smallest 10% reducing from around 0.5mm to 1mm in the vicinity of the NTH to 0.15mm to 0.35mm in the vicinity of the STH. There are no obvious trends observed with decreasing elevation, although the lower bound of D10 values is seen to increase below about 95mTD (9.5mBGL). It is possible that this is due to a loss of fines below the water table during sampling or previous construction works.

Results of percentages of fines (<0.063mm) against elevation and depth respectively. The Figures demonstrate a trend of increasing fines content in the River Terrace Deposits towards the south. The fines content in the vicinity of the NTH is typically less than 2%, whereas in the vicinity of the STH the content increases up to around 3% to 5%. Shallower samples show occasional high fines content (10% to 60%) in all areas, although more often in the vicinity of the STH and PAL. This may be owing to the interpretation of Alluvium and/or Made Ground as Terrace Deposits. Alternatively the reduced occurrence of high fines content at depth could be owing to factors which cannot be discounted; may include;

- natural stratigraphic grading variations of the Terrace Deposits

- loss of fines during sampling below the water table;

- or

- loss of fines below the water table during previous construction works.

From the results of particle size distribution testing performed on samples of the River Terrace Deposits recovered from the VSU site, the grading is expected to vary both vertically and horizontally over a relatively short distance. The borehole logs have revealed that these terraces comprise variable proportions of sand and gravel. The water bearing River Terrace Deposits and existing groundwater control measures may have resulted in fines being eroded from the River Terrace Deposits, particularly at lower elevations, thus leaving locally more permeable deposits.

8.2 Permeability

Estimates of permeability values were derived from PSD data for the River Terrace Deposits using Hazen's formula; $0.001 \times D10^2$, where D10 is the effective size (mm), meaning it is the largest size of the smallest 10% of the soil sample passing through the sieve. Each permeability value was ascribed a confidence category according to the uniformity coefficient (C =D60/D10); C values less than 4 indicating uniformly graded soils were deemed to give lower confidence results, whilst C greater than 4 indicating well graded or gap graded were thought to give higher confidence results.

The results of analysis show that permeability does not vary significantly with elevation or depth for permeabilities derived from PSD results. The average value is 1×1.0E-03m/s and the range is typically between 2.0E-04m/s and 1.0E-02m/s corresponding to medium to high permeability. As expected there is higher confidence associated with the relatively lower permeability values, and lower confidence associated with relatively higher permeability results.

Four permeability values were recorded from in situ falling head tests undertaken in the recent GI between 93mATD and 94mATD. The values plot between 1E-05m/s and 1.0E-6m/s, corresponding to low permeability, and are significantly lower than those derived from PSD tests. Permeability results from falling head tests achieved in the previous GIs are relatively close to the PSD derived permeabilities, apart from one low value.

It must be stated that owing to the loss of fines during sampling, there is a possibility that the subsequently derived permeability values could be overestimated; however it appears more likely that the falling head tests undertaken during the recent GI may not be representative. The River Terrace Deposits generally contain limited quantities of fines in the zones tested. Factors such as smearing and caking of clay onto the surface of the gravels from overlying alluvium during drilling may be one reason to explain these low permeability values, which should be treated with caution.

Permeability values have been derived using Hazen's formula, corresponding to different regions of the site, the permeability values derived for the STH and the PAL links are around 6.0E-04m/s. In contrast, for the NTH, values are considerably higher around 2.0E-03m/s, which shows consistency with the cleaner deposits in the north of the site as previously discussed in Section 8.1.4(iii).

8.3 Liquidity Index, Void Index, Intrinsic Compression Line and Sedimentary Compression Line

The Made Ground and Alluvium liquidity values are typically between +0.05 and +0.65, which reflects the relatively low overconsolidation ratio of the Alluvium.

The liquidity indices for the London Clay unit B are principally clustered around 0.0 with a minority of indices around -0.5. Within units A3 and A2, there is much greater scatter and a trend is less obvious. The void indices for the London Clay sub-units B, A3 and A2, display an overall decrease (i.e. increasing negative values) at greater depth. Step changes in void index are observed at the boundaries between sub-units.

Vaughan (1997) postulated that for geologically old soils, such as the London Clay, the pre-consolidation pressure is close to the intercept of the *in situ* swelling line with the ICL. The data indicates a pre-consolidation pressure in the order of 4000kPa for all three identified London Clay units. This is consistent with geological reports of 200 – 300m of erosion from above the present London basin.

8.4 Penetration Resistance Tests

8.4.1 Standard Penetration Tests

The SPT-N values recorded within the Made Ground typically fall between N=0 and N=25 with an average value of N=14. Most of the outlying values are derived from incomplete tests which probably reflect the presence of an obstruction such as a brick or concrete fragment, which may have been encountered. The variability within the Made Ground is understandable given the complex nature of historical development and redevelopment in and around the VSU site.

Eleven SPTs were undertaken within the Alluvium, giving a large range in values from N=1 to N=40, although they are generally less than N=16 and have an average of N=15. The variation of N values with elevation suggests a general increase of stiffness/strength with decreasing elevation, although there is also a high variation for a given level interval. As described previously alluvium contains localised sand and organic pockets, which are likely to affect SPT-N results. There is also the issue of interpretation of overlying Made Ground and underlying River Terrace Deposits which could potentially be mistaken as Alluvium.

The SPT ‘N’ values are highly variable with similar scatter observed when plotted with either depth or elevation. The lower bound ‘N’ values are approximately constant with depth (or decreasing elevation), with a value of about 10. The upper bound ‘N’ values tend to increase with depth (or decreasing with elevation) from a value of about 20 close to its surface, towards about 60 close to the base of the RTD. The lower bound values may reflect disturbance at the base of the borehole, especially below the water table, or the presence of more cohesive layers. The upper bound values may reflect the influence of increasing effective stress with depth, and/or coarse gravel particles.

There is no discernable difference between the split spoon and the cone test results.

In the London Clay there is a trend of increasing SPT ‘N’ with depth. Through unit B ‘N’ values are clustered around a linear trend, with a mean value increasing from approximately 20 at the surface of the London Clay to approximately 50 at 76mTD, the interface with unit A3. Through units A3 and A2 there is a notable increase in the scatter of the SPT ‘N’ values recorded and a reduction in the rate of increase with depth. The general trend continues to be approximately linear, with a mean value of about 50 at the surface of unit A3 increasing to around 80 at 50mTD, the interface with the Lambeth Group.

8.4.2 Cone Penetration Testing

The static cone penetration tests where conducted from the base of a pre-drilled hole, approximately 1m into the London Clay, a $10cm^2$ cone was used for all tests. The q_c and F_R profiles plotted have not been smoothed or averaged. Seismic cone tests where attempted on site but the results where unreliable.

There is negligible variation in q_c profile across the site, with the typical variation about a mean profile of around +/- 10%. An overall trend of increasing q_c and reducing F_R is observed with increasing depth. The three tests which penetrate below 84mTD show a reduction in q_c from about 4.5MPa at 84mTD to 3.75MPa at 82.5 mTD, followed by an increase back to around 5MPa at 81mTD. A similar opposing trend is observed in the friction ratio, with values increasing between 84mTD and 82.5mTD and reducing again below.

9 Geotechnical Design Parameters

The Geotechnical Design Parameters have been selected based upon all the available ground investigation data and Mott MacDonald's previous design experience. These parameters have also been chosen solely for multi propped retaining wall and shotcrete concrete lining tunnel design.

9.1 Made Ground

9.1.1 Unit Weight

Characteristic unit weight for Made Ground can be taken as 19.0 kN/m^3.

9.1.2 Undrained Shear Strength

Within the Made Ground only two undrained shear strength values of 30kPa and 38kPa were obtained from unconsolidated undrained triaxial tests on 100mm diameter samples. Low confidence is associated with the Made Ground characteristics owing to its variability. A moderately conservative characteristic value of 24kPa can be assumed for design purposes, with c' =0.

9.1.3 Effective Stress Shear Strength Parameters

The upper bound SPT 'N' value of N=25 suggests an internal friction angle of 34 degrees (Stroud, 1989), whereas the lower bound N value of 0 at the surface, increasing to 5 at 100mTD, infers a very low strength material where a friction angle cannot be correlated. Given the variability of the Made Ground a moderately conservative characteristic peak angle of effective friction of 25° is recommended for design purposes.

9.1.4 Deformation Moduli

A constant drained Young's Modulus of 5MPa is assumed for Made Ground.

9.1.5 Coefficient of Earth Pressure at rest (K_o)

A coefficient of earth pressure at rest (K_o) of 0.5 can taken as the characteristic design value.

9.2 Alluvium

9.2.1 Unit Weight

A characteristic unit weight for Alluvium can be taken as 19.0 kN/m^3.

9.2.2 Undrained Shear Strength

The undrained shear strength values have been obtained directly from quick undrained tests on undisturbed samples, piston samples, in situ vane tests, and from pressuremeter tests using the Gibson and Anderson Method and the curve fitting method. Undrained shear strengths have also been derived from reported SPT "N" values. For this correlation Su= f_1N was used where f_1 was taken to be 4.5 (Stroud, 1989).

The undrained strength of Alluvium shows a wide spread in values from Su=14kPa to Su =57kPa, with three outlying higher values of 81kPa, 148kPa, 162kPa derived from SPTs in addition to a value of 70kPa obtained from a shear vane test. The median of all the results is 32kPa and the mean excluding the high values (>60kPa) is 33kPa. There appears to be a general trend of increasing strength with depth. The lower bound values are observed to increase from 4.5kPa at 102.2mTD to 18kPa at 99.0mTD. Differences of undrained shear strengths given by different methods are not significantly discernible. A moderately conservative characteristic value of 24kPa should be taken for design purposes, regardless of the elevation level.

9.2.3 Effective Stress Shear Strength Parameters

A correlation between effective friction angles and plasticity indices for normally consolidated marine clays has been shown by Bowles ,J.E (1987). Using this and an average plasticity index for Alluvium of 42% suggests effective friction angle of 26° for undisturbed clays.

The fully softened effective friction angle can be estimated using a relationship with liquid limit, clay fraction and effective stress (after Stark and Eid, 1997), where less scatter occurs compared with correlations using plasticity index. The effective normal stress of Alluvium is taken to be 100kPa and a clay fraction of between 25% and 40% is assumed based on the limited PSD test results and the logging descriptions. Using the average liquid limit for Alluvium of 75%, a fully softened friction angle of 28° is indicated from the Stark and Eid plot, with c'=0.

However a moderately conservative value of peak friction angle of 20° and c'=0 is recommended for design purposes to allow for the very high upper bound values of liquid limit which have been observed in the Alluvium.

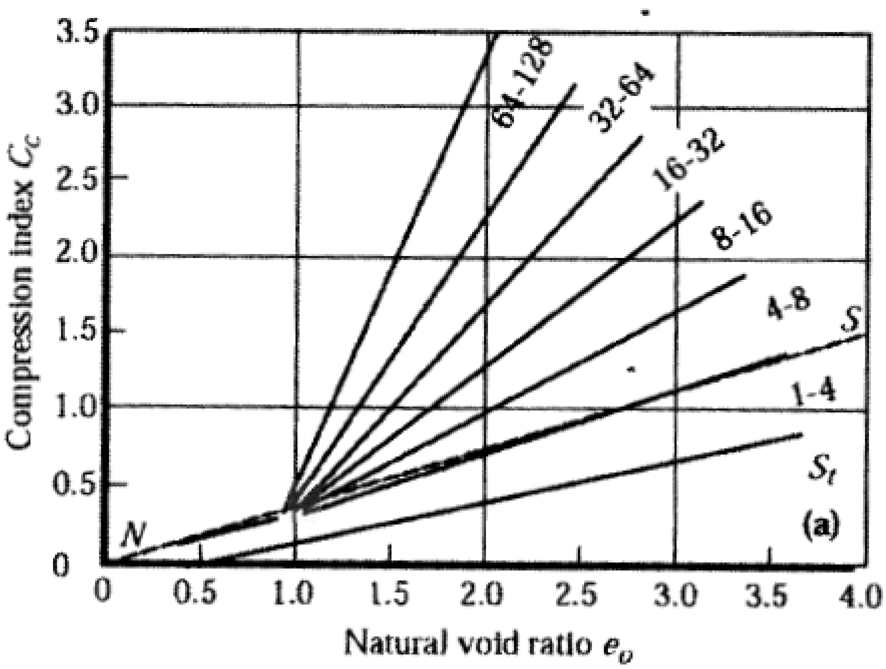
9.2.4 Deformation Moduli

Cc from Empirical Relationships

The empirical relationship below has been used to estimate Cc from the Liquid Limit (LL) data for Alluvium shown in Figure 8.6. The results of this correlation are plotted with reduced level in **Error! Reference source not found.**.

$$Cc = 0.009(LL - 10)$$

The compression index, Cc, can also be linked to the natural void ratio and sensitivity using the chart reproduced below (after Nagaraj and Srinivasa Murthy (1986)).



The sensitivity of the soil layers can be estimated from the undrained strength, Su, and liquidity index, I_L, using the following relationship;

$$S_t = Su(I_L - 0.21)^2$$

Using the above equation to evaluate sensitivity and the data for moisture content (e = MC x Gs, where Gs is assumed to be 2.65)

The table below summarises predicted values of Cc from the Nagaraj and Srinivasa Murthy chart. The values show reasonable agreement with the results of the Correlation with LL and the oedometer test results. The majority of results fall between 0.2 and 0.6 with higher values (up to 1.75) representing samples of more sensitive material with higher moisture content.

Strata	Typical Su (kN/m^2)	I_L	S_t	e_0	Cc
Alluvium	24	0.15 – 0.6 (0.4)	0.1 – 3.7 (0.9)	0.7 – 3.6 (1.0)	0.2 – 1.3 (0.37)

The use of compression indices are recommended if settlement or heave estimates need to be made due to changes, for example, in pore water pressure in the Alluvium.

For retaining wall analysis, a constant drained Young's Modulus of 5MPa should be assumed for Alluvium for design purposes.

Three pressuremeter tests were conducted in the Alluvium, giving the following range of undrained Young's Modulus values of 35MPa to 93MPa (0.01% strain), 18MPa to 45MPa (0.1% strain), 9MPa to 22MPa (1% strain). Although for thin layers of Alluvium, the drained modulus is more relevant for design purposes.

9.2.5 Coefficient of Earth Pressure at rest (K_o)

Coefficent of Earth Pressures values were derived from three pressuremeter tests conducted in the Alluvium, using the Marsland and Randolph method and the curve matching technique. The values ranged from K_o =~0.3 to K_o=~1.5, with four out of the six values being less than K_o =0.7, and the curve matching procedure typically producing the smaller K_o values. A constant coefficient of earth pressure at rest (K_o) of 0.5 can taken as the characteristic design value.

9.3 River Terrace Deposits

9.3.1 Interface Levels

The modelled surface elevation of the River Terrace Gravels ranges from 97.0mTD to 102.0mTD. Generally the higher elevations (>100.0mTD) are indicated east of the proposed works whereas the lower elevations occur (less than 98.5mTD) to the west of the proposed works. The footprint of the works is shown to cover River Terrace Deposits with associated elevations of between 98.5mTD and 100.0mTD. Within the vicinity of the crossover of the D & C line and the Victoria Line a relative topographic high of 101.5mTD is observed, whilst the elevations of gravels below the proposed works to the north are noticeably lower.

There is a 95% chance that for a given point the true surface of the Terrace Gravels will be encountered 2.0m above or below the contoured surface.

9.3.2 Unit Weight

The unit weight of River Terrace Deposits may be taken as 19kN/m^3 for design purposes.

9.3.3 Relative Density

The SPT-N values have been corrected (N_1) for vertical effective stress.

The River Terrace Deposits are taken to be normally consolidated. Therefore, assuming a lower bound value of N_1=5 to be constant with changing depth/elevation, the lower bound relative density (D_r %) is about 20%.i.e. “loose” (Stroud, 1989). Similarly, an upper bound value of N_1=40 is assumed not to change with depth/elevation giving an upper bound relative density of about 85% .i.e. “dense/very dense”. A constant characteristic value of N=15 is selected giving a relative density of 40%., i.e medium dense.

9.3.4 Effective Stress Shear Strength Parameters

To derive the peak friction angle (φ’), the critical friction angle (φ'_{cv}) is estimated based on the material sorting and particle shapes. The River Terrace Deposits are generally either well graded or uniformly graded soils. Consequently, the lower bound φ'_{cv} is selected to be 30° corresponding to a uniformly graded, rounded quartz. For the upper bound φ'_{cv} , a value of 38° was identified corresponding to a well graded angular quartz, and for the moderately conservative characteristic φ'_{cv} value, 32° is appropriate corresponding to a sub-angular uniformly graded sand.

Referring to the plane strain configuration in Figure 5 from Stroud (1989), and using a lower bound relative density of 20% and φ'_{cv} of 30°, the difference between peak effective friction angle and critical effective state friction angle (i.e. φ'-φ'_{cv}) is reported to be 2°. Therefore the lower bound φ' is estimated as 32°. Adopting the same approach, the upper bound φ' is estimated as 49° (φ'-φ'_{cv} = 11°). The characteristic effective friction angle, which can be used for design purposes is 38° (based on N=15, φ'-φ'_{cv} =6° and φ'_{cv} =32°).

9.3.5 Deformation Moduli

Stroud (1989) proposed a correlation between stiffness (Young's modulus) and SPT 'N' values for coarse soils for different levels of strain.

The stiffness/SPT 'N' values correlations are based on uncorrected 'N' values. For soil/structure and ground movement analyses it is usually more appropriate to assume an increase in Young's modulus with depth. Considering the SPT data, the SPT 'N' value can be assumed to increase from 15 at the RTD surface to 35 at the base. For retaining wall analyses which assume a linear elastic Young's modulus, and assuming the RTD as normally consolidated, then E' is usually assumed to equal 'N'. Hence: E' =15MN/m^2 at the surface, increasing to 35MN/m^2 at the base of the RTD, can be assumed as characteristic design values.

For non-linear numerical modelling. the following recommended decay curve has been derived from the test data and published data on the behaviour of River Terrace Gravels and recent experience on similar deposits. The data are presented as shear modulus normalised by mean effective stress (G/p'). Drained moduli may be derived from the following relationship;

$E' = 2(1+ \nu')G$

Where ν' the drained Poisson's ratio is assumed to be 0.3 for the River Terrace Gravels.

To avoid erroneously low values of deformation modulus as mean effective stress approaches zero, i.e. near the ground surface at excavation boundaries absolute minimum value to be adopted for the River Terrace Gravel is G_{min} = 5MPa.

Values of normalised shear modulus with axial/shear strain amplitude are presented below for the River Terrace Deposits.

Strain	G/p' (Isotropic)
0.005	813
0.01	500
0.10	187
0.20	133

9.3.6 Coefficient of Earth Pressure at rest (K_o)

A constant coefficient of earth pressure at rest (K_o) of 0.5 can taken as the characteristic design value.

9.3.7 Groutabililty

The degree of groutability of soils was defined by Mitchell in the ICSMFE conference in Stockholm in 1981 (Soil Improvement State of the Art report, pages 509-565). The definition states that the ratio '(D15) soil'/'(D85) grout' should be greater than 24 for good grouting with the ease of grouting decreasing as this ratio reduces. For an OP cement the D85 is around 30-50 microns giving a required (D15) soil of around 0.7mm to 1.2mm. For a microfine cement, the D85 is 7-12 microns giving a D15 of the soil of 0.15mm to 0.3mm.

It is observed that the measured D15 value of the River Terrace Deposits typically exceeds 0.15mm. Where D15 is less than an effective size of 0.15mm, both the OP and microfine cement grouts will be unsuccessful. For application of an OP cement grout, where D15 should exceed 0.7mm, only a small proportion of the cleaner gravels in the NTH would be able to be grouted by adopting this method. Because of the tendency for fines to be lost during conventional sampling of sands and gravels, the insitu D15 is likely to be lower than indicated by laboratory measurements. Hence, allowing for this factor some problems with grouting using micro-fine cements should be anticipated once the lab D15 values drop below, say 0.3mm to 0.4mm. Threrefore grouting is likely to be difficult and probably impractical in the RTD, close to the STH and across much of the PAL. Grouting in the vicinity of the NTH is likely to be more successful but thin seams of silt/clay within the RTD (based on borehole logging) mean that even adjacent to the NTH the RTD may not be fully grouted. An important consideration in addition to the PSD's is the macrofabirc of the deposit. Thin seams of silt/clay have been recorded in the RTD, between zones of "cleaner" gravel. A consequence of this, is that grouting is likely to result in discontinuous grouted "blocks" of material separated by thin seams of silt/clay. Hence the treated material may still be unstable if tunnelling is attempted through these materials.

It is observed that D15 values tend to decrease from north to south (i.e. NTH to the PAL to the STH area). Therefore if permeation grouting was selected, groutability is expected to decrease from north to south. This is consistent with the PSD results which suggest cleaner sands and gravels in the north (NTH area) with an increased proportion of sands and fines further south (iii) and also the permeability results. The use of permeation grouting in the past adjacent to the STH area has been relatively unsuccessful which is consistent with the above analysis.

Jet grouting is less sensitive to the material type, sizes and grading than conventional grouting methods. The soil is essentially eroded with the fabric destroyed, and then mixed with grout to form a column. Currently jet grouting is the favoured technique for grouting of the superficial deposits within the VSU site. Nevertheless jet grouting will be influenced by variations in soil type; in more silty or clayey soils it is more difficult to erode the soil. Hence, more energy is required to form a coherent block of jet grouted material in silty/clayey soils than in clean cohesionless sands or gravels. This means that the costs and time required for jet grouting will vary across the site, and will also be more difficult when jet grouting in the overlying Alluvium/Made Ground, and, particularly into the underlying London Clay. Jet grouting adjacent to the STH is likely to be more challenging than areas close to the NTH.

For superficial deposits subjected to ground treatment (i.e. jet grouting) the following parameters are recommended based on past experience.

Material	Unconfined Compressive Strength (MPa)	Drained Elastic Modulus (MPa)	Poisson's Ratio	Undrained Shear Strength (kPa)
Alluvium	0.5	100	0.2	125
RTD's silty or clayey horizons	1.0	300	0.2	250
"Clean" RTD's well graded sand/gravel	3.0	1000-1500	0.2	750

Following completion of the jet grouting trial these parameters may need to be reassessed once the data from the trial has been interpreted.

9.4 London Clay

9.4.1 Interface Levels

The estimated surface elevation of the London Clay derived from Geostokos modelling, indicates that the London Clay surface varies across the site from 92.6mTD to 94.2mTD. Within the footprint of the proposed works, the variation is typically between 93.2mTD and 93.6mTD. There are relative local depressions where the surface of the London Clay is modelled as low as 92.8mTD proximal to the proposed works, including; the north eastern end of the NTH; immediately east of STH; and immediately west of the D&C link passage.

There is a 95% chance that for any given point on the contour plot the true surface of the London Clay will be encountered 1.1m above or below the modelled surface, this reflects the statistical confidence limits of the analysis, based on the amount of input data. It should be noted that this analysis is only likely to reflect "natural" surface level elevations. Local variations due to historic construction activities would not be identified by this analysis.

9.4.2 Unit Weight

The assumed unit weight of London Clay may be taken as 20kN/m^2 for design purposes.

9.4.3 Stress History and Stress State

Best estimate profiles have been derived for vertical, horizontal and mean effective stresses (p_0' = $1/3(\sigma_v'+2\sigma_h')$) in the London Clay, based on the SLS groundwater profile. For the purpose of this plot the ground level has been assumed constant at an average level of 104.5mTD. To derive the best estimate profile for *in situ* horizontal effective stress a judgement was made based on the results of the techniques describe below.

(i) Stress History

The over consolidation ratio (OCR) of the London Clay at the VSU site before the deposition of the River Terrace Deposits and superficial deposits, and at the present day has been calculated using a pre-consolidation pressure of 4000kPa and assuming that prior to the deposition of the overlying strata the London Clay surface was at its current level and groundwater pressures were hydrostatic. The pre-consolidation pressure has been derived from the void index and intrinsic compression line as described by Vaughan (1997).

An estimate of K_0 and hence horizontal effective stress prior to deposition has been made using the relationship between OCR and K_0 proposed by Mayne and Kulhawy (1982). The recent changes in stress history, due to the deposition of the River Terrace Deposits and superficial deposits and changes in ground water pressures, have been evaluated using the elastic solution for one-dimensional loading/unloading described by Burland *et al.* (1979) ($\Delta\sigma_h' = \Delta\sigma_v'.\nu'/(1-\nu')$). Estimates of horizontal and vertical effective stress in the London Clay before and after the recent changes in stress state have been made together with an estimate of the current *in situ* K_0 profile based on the stress history approach compared to that derived from self boring pressuremeter tests.

(ii) Self Boring Pressuremeter Tests

The horizontal effective stress with depth measured during self boring pressuremeter (SBP) tests through the London Clay compared to the stress history estimate has been derived together with the corresponding values of K_0. Total stress has been converted to effective stress. Two different techniques have been used to measure the *in situ* horizontal stress;

- Inspection of the cavity pressure versus strain curve for the average of all SBP arms
- A curve fitting technique which assumes soil stress-strain response is non-linear (power law) and undrained (after Bolton and Whittle, 1999).

The inspection of cavity 'lift-off' pressures typically gives a lower estimate of horizontal stress than the curve fitting approach. The lift off pressures are generally considered more reliable. However, the 'lift-off' pressure can be affected by disturbance of the cavity wall during installation.

(iii) Correlations with Undrained Shear Strength

Skempton (1969) demonstrated that the undrained shear strength of a stiff over consolidated clay can be related to the mean effective stress using the effective stress strength parameters and the pore pressure coefficient A_f. A more simplistic relationship, p_o' = kSu, can be a reasonable assumption in some stiff clays (with k=2.0 when Su is based on high quality samples, thin wall or rotary cored or block, and with shearing at relatively slow strain rates, say 4% to 5% per clay). For comparative purposes the estimated *in situ* mean effective stress using the mean undrained shear strength is compared with the mean effective stress estimated from stress history and SBP tests. When Su is based on conventional triaxial testing of U100 samples, due to the varying influence of sampling and testing effects, the value of k will tend to be lower at shallow depth, and higher at greater depth.

As some sampling disturbance is inevitable the *in situ* effective stress would be expected to form an upper bound to the suction test data, (particularly those derived from rotary coring, and/or with silt/sand seams in the sample) with the more disturbed samples falling well below the *in situ* value.

9.4.4 Undrained Shear Strength

Data has been obtained from a combination of laboratory and *in situ* testing using the following techniques;

- Standard Penetration Tests (SPT 'N')
- Static Cone Penetration Tests (CPT q_c)
- Self Boring Pressuremeter tests (SBP)
- Quick undrained triaxial tests (U100)

The field measurement profiles have been derived from the test results in the following ways:

(a) SPT 'N': Su values are based on empirical correlation with SPT 'N' value and plasticity index. In the London Clay Su = fi.N, where 'fi' is equal to 4.5. The value of 'fi' is based on the relationship between 'fi' and plasticity index described by Stroud (1989) and a mean plasticity index from Figure 8.9.

(b) CPT q_c: Su values have been calculated as follows, and are presented and compared with SPT 'N' derived values;

$$Su = \frac{(q_c - \sigma_{v0})}{N_k},$$

where σ_{v0} is the total vertical stress, and q_c is based on the mean of the profiles. The N_k value is dependent on both the plasticity index and the 'structure' of the soil, in particular fissure spacing. Two profiles have been plotted, corresponding to N_k values of 26 and 20 (based on research by the BRE, Marsland and Powell, 1989). An N_k value of 26 is reasonable for stiff clays with "intermediate" spaced fissures; whereas N_k = 20 is more reasonable where fissures are closely spaced. For VSU, an N_k of 26 seems most appropriate.

(c) SBP: The Su values have been calculated from the gradient of the graph of cavity pressure versus change in logarithm of volumetric strain from the latter part of the pressuremeter test, as described by Gibson and Anderson (1961). As described by Yeung and Carter (1990), the Gibson and Anderson method assumes expansion of an infinitely long cavity which over estimates shear strength. For finite L/D values (length/ diameter) a correction factor is necessary, which for the London Clay is 0.75. In addition, as discussed by Clark (1996), the pressuremeter needs to expand to strains in excess of 6%; otherwise strength tends to be overestimated.

CPT data is only available through the top 10-12m of the London Clay, down to 80mTD. The CPT prediction of Su assuming $N_k = 26$ is an approximate mean to the SPT results above about 85mTD.

The 'Quick' undrained triaxial tests on U100 samples plot higher than the general *in situ* trends through the top 10 – 15m of the London Clay. Below this the U100 results tend to fall below the *in situ* data. This is likely to be due to an increase in mean effective stress in shallower samples due to the sampling procedure and high strain rates used during conventional tests, both factors leading to increases in measured strengths. At greater depth the U100 results tend to fall below the *in situ* data, most probably due to pore pressure migration in the sandier London Clay units, and remoulding (due to sampling disturbance) of the more structured Unit A London Clay, these factors would lead to decreases in measured strengths.

Linear design profiles for worst credible, moderately conservative (EC7 – "characteristic") and mean undrained shear strength, Su, are described below with respect to elevation in mTD (z):

- Mean Su (kN/m^2); 959 – 9.43z
- Moderately Conservative Su (kN/m^2); 762 – 7.55z
- Worst Credible Su (kN/m^2); 567 – 5.65z

9.4.5 Effective Stress Shear Strength Parameters

Examination has been undertaken of the peak s' – t values from undrained triaxial tests with pore water pressure measurement, undertaken as part of the 2006/7 VSU ground investigation. Samples taken as part of the VSU Additional Ground Investigation are currently being tested and the results are not yet available. Considering the relationship with liquid limit proposed by Stark and Eid (1997) suggests a fully softened friction angle of between 22^o and 26^o, with the higher values corresponding to the deeper, sandier A2 unit.

Based on the factors noted above and previous experience in comparable ground conditions a judgement has been made on the appropriate moderately conservative (EC7 – "characteristic") effective stress strength parameters in the London Clay. For unit B, c' = 10kN/m^2 and phi' = 22^o. For units A3 and A2, c' = 15kN/m^2 and phi' = 26^o. These design lines are comparable to the Hight and Jardine profiles, for the shallower depths of London Clay. These design lines are likely to underestimate the peak strength of the basal units of London Clay, however it is the shallower units of London Clay which will be most relevant for the VSU project.

9.4.6 Undrained Deformation Moduli

(i) Self-Boring Pressuremeter Tests

The secant shear modulus as been considered against shear strain for selected SBP tests in the London Clay. Secant shear modulus has been calculated along the re-loading path of the first un-load re-load loop, with the lowest value of stress and lowest value of strain for that loop being used as a datum for the subsequent increases in cavity pressure and strain. Cavity strain has been converted to shear strain using the transformed strain method proposed by Jardine et al.(1991). The data shows that the stress strain behaviour is non-linear, with shear modulus sensitive to shear strain amplitude, and that shear modulus increases with depth. There is a marked increase in shear modulus from the A3 to the A2 London Clay unit.

The secant shear modulus has been normalised by the secant shear modulus at 0.01% shear strain amplitude for the London Clay units B, A3 and A2 respectively. The co-efficient of non-linearity, L = $G_{0.1\%}/G_{0.01\%}$, can be used to describe the non-linearity of a given test response. Plotting the data in this way demonstrates that the observed non-linearity is relatively consistent between tests in the same unit, with non-linearity increasing (L reducing) in the deeper more sandy units. L values of 0.43 (+/- 0.07), 0.40 (+/- 0.07) and 0.36 are recorded in the London Clay units B, A3 and A2 respectively. Some scatter is introduced, in part due to the reduced reliability of the pressuremeter at low strains, the range could not been quantified for unit A2 as only 2 tests reach 0.01% strain.

In unit B there is significant scatter in the G/p_0' values at 0.01% strain, with the majority of data falling between 300 and 450. There does not appear to be any trend in the scatter with test elevation, but it is clear that SBP tests conducted during the more recent GI lie above those carried out previously. The difference could be due to changes in the installation or test technique or proximity to existing infrastructure leading to variations from the assumed best estimate p_0' profile. At 0.1% strain the majority of data lies between 110 and 180.

In unit A3 only one test was conducted prior to the most recent GI. As observed in unit B this test falls below those made more recently. When normalised with p_0' the secant moduli in unit A3 are comparable to those recorded in unit B. The recorded G/p_0' values lie between 350 and 450 at 0.01% strain, and between 120 and 150 at 0.1% strain.

The results of four SBP tests are available for unit A2, all from the most recent GI. The recorded values of G/p_0' in the sandier A2 unit are notably higher than the values recorded in the overlying London Clay (units B and A3). At 0.01% strain G/p_0' values lie between 450 and 570 with 3 of the 4 tests between 520 and 570. Data only extends to 0.1% strain for 2 of the tests, extrapolating the data trends suggests G/p_0' values between 160 and 250.

. The observed trend of undrained stiffness ($E_u = 3.G_s$) with level is consistent with the trend Values measured in SBP tests with in undrained strength through the London Clay. The relationship E_u = 800Su shows a close correlation to the pressuremeter data at 0.1% strain. The relationship E_u =2000Su correlates reasonably well with the SBP data at 0.01% strain, especially above 65mTD (i.e. upper 25m of London Clay). At greater depth the SBP data implies E_u at 0.01% strain tends towards 2500Su.

(ii) Undrained Triaxial Test Data

The results of 12 anisotropically consolidated undrained compression and extension tests are available from the 2006/7 VSU ground investigation. The envelopes observed by Hight et al. (2003), for tests on high quality samples of London Clay units B, A3 and A2 from Terminal 5, Heathrow have been for comparative purposes. The VSU extension tests fall within the Hight envelopes, although tend towards the lower bound (possibly due to sample disturbance). It is noted that the shearing phase involves a sharp change in stress path direction relative to the consolidation phase, see inset, this will facilitate engagement of the ‘Y1’ yield surface and more realistic measurements of small strain stiffness. The available compression tests for the VSU site fall well below the envelope given by Hight. It is postulated that this is a result of the initial ‘Y1’ yield surface not being observed due to the continuation of the consolidation stress path at the start of the shearing phase, see inset. The VSU extension tests, which underwent a change in stress path direction at the start of the shearing phase (i.e. a stress reversal), show a stiffer initial response. These data plot towards the centre of Hight’s envelope, but tending to the lower bound and below at larger strains.

Undrained triaxial tests have been specified as part of the 2007/8 VSU Additional Ground Investigation but are as yet incomplete.

9.4.7 Drained Deformation Moduli

Drained deformation moduli for swelling beneath excavations have initially been derived based on previous experience.

(i) Drained Triaxial Tests

Drained triaxial tests have been specified as part of the 2007/8 VSU Additional Ground Investigation but are as yet incomplete.

9.4.8 Bulk Modulus

Reliable measurements of the non-linear bulk modulus of London Clay are difficult to obtain. The envelopes of a recent set of high quality measurements by Yimsiri (2002) have been reviewed as bulk modulus normalised by the current value of mean effective stress (K/p’). The normalised bulk modulus decay curves used for analysis at Terminal 5, Heathrow have also been included (from Hight et al., 2003), along with the design profiles, derived envelope of results from some high quality testing undertaken as part of the ground investigation for the New Wembley Stadium by Mott MacDonald.

9.5 Recommended Design Parameters for Non-Linear Undrained and Drained Deformation Moduli

9.5.1 Recommended Parameters

For non-linear numerical modelling of ground deformation and soil-structure interaction, the following recommended deformation decay curves have been derived from the data discussed in above, published data on the behaviour of London Clay and recent experience of similar London Clay sites. The data are presented as shear modulus normalised by mean effective stress (G/p'). Undrained and drained Young's moduli (E_u and E') and bulk moduli (K) may be derived using the following relationships;

$$E_u = 3G$$
$$E' = 2(1+\nu')G$$
$$K = 2G\frac{(1+\nu')}{3(1-2\nu)}$$

Where ν' is the drained Poisson's ratio, assumed to be 0.1 for clays and 0.2 for Lambeth Group sands

For suitable parameters for calculating heave below an excavation, e.g. using NLS spreadsheet (O'Brien and Sharp, 2001), refer below for drained soil swelling parameters.

To avoid erroneously low values of deformation modulus as mean effective stress approaches zero, i.e. near the ground surface or at excavation boundaries, the following absolute minimum values should be adopted;

- All London Clay, G_{min} = 3MPa
- All Lambeth Group Clay, G_{min} = 5MPa
- All Lambeth Group Sands, G_{min} = 10MPa

Values of normalised shear modulus with axial/shear strain amplitude are presented below for the London Clay and Lambeth Group clay and sand. Due to the anisotropic nature of the London Clay and Lambeth Group, three different moduli have been presented for these strata. Horizontal moduli are appropriate for situations where the principal stress change is horizontal, e.g. behind retaining walls. Vertical moduli are appropriate for situations where the principal stress change is in the vertical direction, e.g. unloading in front of a retaining wall. For situations where the change in stress is more complex, e.g. around a tunnel, approximate isotropic parameters have been provided.

- London Clay, Unit B

Strain	G/p' (Horz.)	G/p' (Vert.)	G/p' (Iso.)
0.005	554	401	478
0.01	467	333	400
0.10	177	110	160
0.20	120	75	109

- London Clay, Units A3 and A2

Strain	G/p' (Horz.)	G/p' (Vert.)	G/p' (Iso.)
0.005	593	484	539
0.01	500	400	450
0.10	190	120	175
0.20	119	75	109

- Lambeth Group Clay

Strain	G/p' (Horz.)	G/p' (Vert.)	G/p' (Iso.)
0.005	700	614	657
0.01	580	500	540
0.10	180	120	170
0.20	125	83	118

- Lambeth Group Sands

Strain	G/p' (Horz.)	G/p' (Vert.)	G/p' (Iso.)
0.005	1522	1312	1417
0.01	870	750	810
0.10	218	188	230
0.20	155	134	164

9.5.2 Recommended Parameters, Comparison with Previous Published Data

The recommended parameters for vertical undrained stiffness for unloading stress paths, normalised with mean effective stress, are compared with the envelopes derived from high quality triaxial extension tests (Hight, 2003). The recommended design line is an approximate mean of the combined envelopes, at strains less than 0.02%, tending towards the upper bound at strains of the order of 0.1%. When compared to the available test data for VSU, the design lines form an upper bound. Kovacevic *et al.* (1996) demonstrated that in order to obtain a reasonable estimate of the observations made during a NATM trial at Redcross Way (for the Jubilee Line Extension), the assumed soil stiffness needed to be close to the upper bound of laboratory shear stiffness values. This is consistent with recent MM numerical modelling experience. The profile used by Kovacevic has been compared to the recommended isotropic (“tunnelling”) shear moduli. Kovacevic’s curve is similar to the mean of the unit A and B design lines at the small and large strain cut-offs (0.005% and 0.2%), with Kovacevic’s line falling slightly below the assumed degradation curve at intermediate strains.

Shear moduli from SBP tests tend to be higher than those observed from triaxial tests due to the intrinsic anisotropy of the London Clay (G_{hh} being greater than G_{vh}). Comparison with triaxial test envelopes from Hight et al (2003), for units A and B at heathrow T5, shows that the SBP data from the most recent GI lie between the mean and upper bounds, with data from the shallower unit B plotting below that from the deeper unit A2. As the SBP does not measure a purely horizontal response, an upper bound has been considered when deriving the recommended horizontal moduli. For units A3 and A2 the same design line has been used, giving an upper bound to the A3 test data but closer to a mean value in unit A2. The normalised shear moduli derived from a back analysis of the observed retaining wall horizontal deflections at the Heathrow cofferdam (Chang et al, 2001) has been reviewed. This non-linear profile is close to the proposed design line across the intermediate and large strain range (0.01% and greater), which are most relevant for retaining wall design.

The recommended parameters for the Lambeth Group have been based on experience from investigations in comparable ground conditions for the Crossrail project.

Recommended values of undrained shear stiffness at small strain (<0.005%) compared to small strain predictions using p_0’, based on the approach outlined by Atkinson (2000) have been reviewed. Atkinson’s relationship for G_{vh} with p_0’ shows a similar trend to the recommended parameters, with broadly similar values being predicted by the suggested relationships to plasticity index. Measurements of small strain G_{hh} made at Terminal 5, Heathrow using in situ seismic techniques have been reviewed for comparison. Similar lithologies of London Clay are observed at the two sites, however there is around 10m less overburden at the T5 site. The data has been plotted so that the London Clay surfaces coincide, i.e. London clay surface at T5 is plotted at 93mTD (i.e .the same as the London Clay surface at VSU). The T5 data shows a similar trend to the Atkinson relationship and the recommended parameters. However, the values of G_{hh} at VSU are likely to be slightly higher than those observed at the same depth in the London Clay at T5 due to the increased overburden and hence higher p_o’. Experience from T5 and the Crossrail Project has shown 1200Su to be a reasonable estimate of G_{hh} *in situ*.

9.5.3 Drained Swelling Curve

The drained swelling curve for calculating heave beneath excavations, e.g. using the NLS method (O'Brien and Sharp, 2001), has been derived based on previous experience of similar sites in the London Clay. Young's moduli, E', are defined by values of E' at 0.1% strain amplitude, and ratios of $E'/E'_{0.1\%}$ to describe the variations in moduli with strain.

The values of drained Young's modulus at 0.1% strain amplitude (i.e. at 0.1% vertical strain) assumed for each strata are described as a function of mean effective stress p' as follows;

- London Clay, Unit B; E'/p' = 200
- London Clay, Unit A3; E'/p' = 220
- London Clay, Unit A2; E'/p' = 220
- Lambeth Group; E'/p' = 240
- Upnor Formation, E/p' = 450

The variation of drained deformation modulus with shear strain for all strata is assumed to be as follows;

$E'/E'_{0.1\%}$	Shear strain (%)
0.15	3
0.35	1
0.50	0.5
0.80	0.2
1.00	0.1
1.50	0.05
3.60	0.01

Soil swelling curves, volumetric (or vertical) strain against normalised change in vertical effective stress, derived from the parameters described above at various depths in the London Clay and Lambeth Group have been derived and considered in relation to the results from high quality oedometer testing on thin wall, rotary core and block samples, and case history data from Bell Common (London Clay) and Lion Yard (Gault clay). The data show that curves from thin wall and rotary core samples demonstrate a softer response, plotting above, than those from higher quality hand cut block samples, and that the site observations are notably stiffer than the laboratory results. The recommended parameters for VSU, conservatively, form an upper bound (i.e. slightly softer response) to the available case history data, and give a slightly stiffer response than that observed in oedeometer testing on London Clay block samples from the New Wembley stadium.

9.6 SLS and ULS Pore Water Pressure Profiles

To follow current best practice (CIRIA C580 and EC7), two design lines are needed for serviceability limit state (SLS) and ultimate limit state (ULS) analyses of the proposed permanent works.

For the SLS condition, the water table is taken at 97mTD due to pumping from the District and Circle Lines. Possible localised perched water tables in Made Ground/Alluvium/River Terrace Deposits are considered minor. Water pressure is assumed hydrostatic with depth through the upper part of the London Clay down to 83mTD, below this level water pressure increases at 70% hydrostatic with depth down to 65mTD below which water pressure increases at 50% hydrostatic.

For the ULS condition, the water table is assumed to be at the ground surface as a result of flooding or a burst high level water main. Hydrostatic pressures are assumed within the Made Ground/Alluvium/River Terrace Deposits. As the extreme condition is anticipated to be transient and short lived, water pressure in the Upper London Clay remains at the ‘Normal’ condition and conservatively is assumed to vary hydrostatically with depth.

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