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SOUTH LONDON ENERGY FROM WASTE, VILLIERS ROAD SITE

GEOTECHNICAL INTERPRETATIVE REPORT

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GEOTECHNICAL INTERPRETATIVE REPORT

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C O N T E N T S

Page

1. INTRODUCTION

Gifford has been appointed by Lagan Construction Ltd to advise on the civil and structural design for a new Energy from Waste plant to be constructed in South London. Lagan Construction Ltd is working with the Energy from Waste developer Viridor in the latter stages of a bidding process for the contract to establish the new Energy from Waste facility. Two sites in South London are being considered for development, Beddington and Villiers Road. This report concerns the geotechnical interpretation of the Villiers Road site, a separate report has been written concerning the Beddington scheme. This report makes use of the findings of a ground investigation designed and supervised by Gifford together with background information made available about the site.

The site location is shown on **Figure 1.**

The ground investigation was designed by Gifford and undertaken by Geotechnical Engineering Limited. The site work commenced on 9th May 2011 and was completed on 27th May 2011.

This report details the findings of the ground investigation and includes a series of conclusions and recommendations relating to the potential Geotechnical risks and constraints on development at the site. These conclusions and recommendations have been used to develop the foundation, substructure, and buried structure design for the purposes of producing final bid proposals.

2. SITE LOCATION AND SURROUNDING AREA

The site is located in South London, approximately 1 kilometre south east of Kingston-upon-Thames at Ordnance Survey National Grid Reference 518869E, 168581N.

The site is approximately 1.9 hectares in area and is currently occupied by a public recycling facility receiving waste suitable for recycling. Current site facilities include a number of buildings as well as designated storage areas and access routes.

A sewerage works lies to east and the south of the site. This extends 600 metres to the east and 150 metres to the south. There is a cemetery beyond this facility to the south. There is housing to the south west of the site. A business park containing twenty units is located to the west of the site with a recreation ground beyond this. The northern boundary of the site is formed by a large Royal Mail facility along with a cemetery and a crematorium.

Geological mapping for the site [Solid and Drift Geology Map, Sheet E270 South London, 1: 50 000 Series Geological Maps, England & Wales] indicates that the new development is underlain by Kempton Park Gravel, part of the Thames River Terrace Deposits, described to be gravel, sandy and clayey in part. This is underlain by the London Clay Formation. The underlying bedrock is Chalk.

3. STANDARDS AND LIMITATIONS

This report has been prepared for Lagan Construction Ltd and shall not be relied upon by any third party, unless that party has been granted a contractual right to rely on this report for the purpose for which it was prepared.

The overall approach to the investigation was undertaken in accordance with the guidelines presented in BS5930 (1999) Code of Practice for Ground investigation, BS10175 (2011) Code of Practice for the Investigation of Potentially Contaminated Sites and CLR11 Model Procedures for the Management of Contaminated Land (DEFRA / Environment Agency, 2004).

The ground investigation has been undertaken utilising a number of exploratory holes located at positions determined from the site history, current site layout, proposed location of development works, the likely ground conditions and the current standards at the time the work was undertaken. Since the ground conditions are only known in detail at each exploratory hole location the ground conditions between holes have been interpolated. Therefore, the actual nature of the ground may differ from our interpretation. Similarly, chemical analysis has been undertaken on samples recovered only at the exploratory hole positions, and it is possible that areas of contamination are present between the exploratory holes.

The extent of the ground investigation carried out was to provide a general overview of ground conditions likely to be encountered on site. The investigations carried out at site will be suitable for outline and scheme design development. However, it should be understood that, as an aid to construction and scheme specific design, further investigation may be required following final construction proposals. Further investigations would target specific site features to provide a more detailed ground model of the development area.

Conditions on site can vary with time and it is recommended that if development is delayed, then consideration should be given to reviewing the findings of this report, to ensure they are still valid and to assess whether additional sampling and testing would be required.

The risk assessment presented in this report has been undertaken on the basis of the proposed land use. If this is altered then it would be necessary to revisit the risk assessment and potentially the conclusions and recommendations within this report. It is possible that additional remedial works might be needed to meet the requirements of a more sensitive land use.

Observations relating to asbestos are intended to assist the Client and do not constitute an asbestos survey. Gifford are not asbestos specialists and we recommend that you appoint an asbestos consultant to advise you. Gifford can offer recommendations regarding suitable asbestos consultants, but cannot provide specific asbestos risk assessment advice.

This report addresses geotechnical issues only.

4. SCOPE AND OBJECTIVES

The ground investigation obtained geotechnical and contaminated land information. The purpose of this report is to interpret the ground investigation information to identify potential constraints and risks to the development, to provide design and construction recommendations, and to support the bid proposals. It was not possible to investigate the whole site due to the location of existing buildings and constraints imposed by activities on site during the course of the investigation. Further investigation will be required to finalise the design and support any planning applications.

4.1 Scope

The scope of the ground investigation comprised:

- The drilling of cable percussion and window sampling boreholes;
- Excavation of trial pits;
- In-situ testing and recovery of soil and groundwater samples;
- Geotechnical and geochemical laboratory analysis;
- Gas and groundwater monitoring; and
- Interpretative reporting.

4.2 Objectives

The objectives of the ground investigation are as follows:

- To establish the ground conditions and the distribution of strata types beneath the site;
- To comment on the relevant engineering properties of the materials encountered;
- To assess the presence and extent of made ground at the site;
- To inform the foundation design process; and
- To provide recommendations for future works, where appropriate.

5. PROPOSED DEVELOPMENT

The proposed development is an Energy from Waste Plant, designed to facilitate the receipt of waste and its processing to generate energy. The scheme includes:

- The main facility building containing a waste bunker, a feed hopper leading into a boiler, a series of process areas, followed by storage tanks for by-product materials including an ash bunker, a chimney, a turbo generator and an area for air cooling before release;
- Designated areas of the building for water treatment an electrical room and a diesel generator;
- Separate water storage tanks, fuel tanks and an electricity substation;
- Hard standing for access, delivery and maintenance routes, and car parking along with two weighbridges; and
- A publicly accessible household waste recycling centre with storage and contractor access.

6. GROUND INVESTIGATION

6.1 Site Works Undertaken

Geotechnical Engineering Limited undertook the ground investigation between 9th May 2011 and 27th May 2011. Part-time site presence was provided by Gifford during the investigation.

The ground investigation comprised the following:

- Fourteen cable percussive boreholes to depths of 20.0 metres to 30.0 metres below ground level (mbgl);
- Four window samples to a maximum depth of 8.45 mbgl; and
- Four machine excavated trial pits to a maximum depth of 3.0 mbgl.

Exploratory hole locations are shown in **Figure 1** of the Geotechnical Engineering Limited Factual Report in **Appendix A.**

Laboratory testing was carried out on samples recovered during the investigation. A summary of geotechnical testing carried out as a part of the investigation has been provided below.

Gas and groundwater monitoring wells were installed within eight exploratory holes. Data from four rounds of groundwater monitoring visits have been requested. Details of the construction of the wells are shown in the Geotechnical Engineering Limited exploratory hole records, enclosed in Appendix A.

Detailed information on the ground investigation can be found within the Geotechnical Engineering Limited Factual Report in **Appendix A.**

6.2 Geotechnical Testing

Geotechnical laboratory testing was undertaken on samples recovered during the investigation that represented the strata encountered. Test results are contained within the Geotechnical Engineering Limited factual report in **Appendix A.** Data from the laboratory test results have been reviewed and graphical plots have been produced to aid interpretation, the graphical plots can be found in **Appendix B.**

The geotechnical testing of soil samples recovered from the site comprised:

- 36 natural moisture content tests;
- 24 Atterberg limit tests;
- 12 particle size distribution tests;
- 20 sulphate tests for analysis in accordance with BRE Special Digest 1 “Concrete in Aggressive Ground”;
- 20 pH tests for analysis in accordance with BRE Special Digest 1 “Concrete in Aggressive Ground”;
- 2 California bearing ratio tests; and
- 10 undrained triaxial tests.

7. GROUND AND GROUNDWATER CONDITIONS

7.1 Ground Conditions

The exploratory hole records and location plan are presented in the Geotechnical Engineering Limited Factual Ground Investigation Report, enclosed as **Appendix A**. A summary of the ground conditions and average depths of strata encountered during the investigation is provided in **Table 7.1.**

Material	Depth to Base of stratum (mbgl)	Thickness Range (m)
Made Ground	3.5 – 6.0	3.5 – 6.0
London Clay Formation	30.0 (base not proved)	Greater than 14.5

Table 7.1 Summary of Ground Investigation Information

Made Ground
The ground surface varies across the site, but investigation locations encountered block paving, concrete, gravel, brick paving over tarmacadam and vegetation.

Made Ground was encountered in all the exploratory holes and trial pits. Typically the thickness of Made Ground varies across the site with no particular relationship between depth and location. Obstructions were encountered in three locations at 0.9 mbgl to 4.5 mbgl, terminating these holes. The nature of these obstructions was such that the holes in question could not be drilled beneath this level therefore the constituents of the obstruction could not be confirmed.

The Made Ground is reported to contain alternating layers of thickness 1.0 metres to 2.0 metres of clayey gravelly sand and gravelly clay within the north west quarter of the site. Generally gravel within the Made Ground is described as brick, siliceous, plastic and concrete. Lumps of metal, glass, wood, string and pottery are also recorded.

Made Ground was found to contain much less clay within investigation locations across the south east of the site. A clay layer, up to 1.5 metres thick, was encountered in some of the exploratory holes in the Made Ground. The clay layer, where identified, was encountered at the base of the Made Ground. The gravel content of the Made Ground on this part of the site is also brick, siliceous, plastic and concrete.

London Clay Formation
The predominant superficial deposit on this site was identified to be the London Clay Formation, extending from below the Made Ground to beyond the extent of the deepest borehole at 30.4 mbgl.

Immediately below Made Ground the borehole records describe the London Clay Formation to be sandy silty clay containing some fine selenite gravel in places. Below 8.0 mbgl the London Clay Formation becomes stiff, sub horizontally laminated and is described to be much sandier, with sand lenses recorded in some holes. From approximately 9.0 mbgl to 10 mbgl the clay is described as silty and contains silt partings or silt pockets in places. At around 12.0 mbgl and below, the clay becomes sandy again and in two boreholes is said to contain hard thin bands of claystone. The stratum is very stiff below 14.0 mbgl and sand lenses become more frequent. At

15.0 mbgl the clay becomes friable and below 18.0 mbgl fossils, fossil fragments and silt partings are recovered.

Mudstone laminations of around a thickness of 0.1 to 0.3 metres were recorded within the London Clay Formation, on the north of the siteat depths of 9.6 mbgl to 11.0 mbgl.

7.2 Groundwater

Groundwater strikes noted during the drilling/excavation of the exploratory holes are summarised in **Table 7.4.**

Exploratory Hole	Strata	Depth (mbgl)	Depth Following Standing Period (mbgl)
BH01	Made Ground	4.5	Not Recorded
BH02	London Clay Formation	11	10.2 (20 minutes)
BH03	London Clay Formation	9.4	7.9 (20 minutes)
BH04	London Clay Formation	6.3	6.0 (20 minutes)
BH04	London Clay Formation	11	10.8 (20 minutes)
BH06	Made Ground	4	3.8 (20 minutes)
BH10	Made Ground	4.4	4.0 (20 minutes)
BH11	Made Ground	3.5	Not Recorded
WS01	London Clay Formation	4.6	4.6 (20 minutes)
WS03	London Clay Formation	6.6	6.7 (20 minutes)

Table 7.4 Summary of Groundwater Strikes

Groundwater was encountered within Made Ground within a gravel band or as a localised seepage, flow rates were not measured.

Three of the groundwater strikes in the London Clay Formation were associated with bands of mudstone. Another groundwater strike was encountered in a shallow very gravelly section of the London Clay Formation. These were the most significant water strikes in the London Clay Formation. Two other water strikes in silty bands of the London Clay Formation recorded no substantive inflow.

Groundwater level monitoring is to be undertaken by Geotechnical Engineering Limited in order to provide further details on the groundwater regime beneath the site.

8. GEOTECHNICAL RESULTS AND INTERPRETATION

The geotechnical assessment is based on the in-situ and laboratory testing described in the Geotechnical Engineering Limited Factual Report enclosed in **Appendix A** together with the soil descriptions on the exploratory hole records.

The results of the geotechnical testing are summarised in **Table 8.1.** Graphs used for the geotechnical interpretation can be found in **Appendix B.**

Strata type	Plasticity Index PI (%)	Liquid Limit WL (%)	Plastic Limit WP (%)	Natural Moisture Content MC (%)	Standard Penetration Test SPT N	Quick Undrained Triaxial c_u (kPa)
Made Ground	14 – 32	33 – 77	16 – 52	14 – 56	1 – 21	109
London Clay Formation	18 – 53	42 – 82	17 – 31	13 – 37	4 – 66	43 – 408

Table 8.1 Summary of Geotechnical Test Results

8.1 Made Ground

Classification

Moisture content in Made Ground varied from 14% to 56%, with no correlation with depth. Atterburg Limits were calculated for eight samples of Made Ground. Based upon this data samples with dominant cohesive content are classified to be mostly a clay of low plasticity, silt of intermediate plasticity or clay of intermediate plasticity. However one sample was found to be a clay of high plasticity and one was silt of very high plasticity. This reflects the variable nature of Made Ground, however overall a characteristic plasticity index of 16% can be adopted in Made Ground.

Eighteen particle size distribution tests were carried out on samples of the Made Ground. Approximately 85% of the samples were composed of sand or gravel sized particles. Samples from five investigation locations, all on the south west of the site, contained cobbles or boulders up to a maximum of 21% of the tested sample by weight. Clay and silt particles accounted for 5% to 20% of the mass of the samples tested.

Strength

SPT N values in Made Ground range from 1 to 21, however there is no appreciable increase with depth. Due to this variability in Made Ground, a characteristic SPT N value of 4 should be used for design at a founding depth of greater than 1.0 mbgl.

Where Made Ground is cohesive , undrained shear strength can be approximated by;

$c_u = f_1 \times N$,
[Stroud and Butler, 1975]

where; f_1 = 6 based on the characteristic plasticity index of 16%.

Therefore an undrained shear strength of 24 kN/m^2 can be derived from SPTs within the cohesive layers within the Made Ground.

One undrained triaxial compression test was carried out on a sample of Made Ground from a depth of 5.0 mbgl. The resulting undrained shear strength is 109 kPa. While this value for undrained shear strength is in keeping with the higher values calculated from SPT results, it is significantly higher than the characteristic value derived from SPTs for the Made Ground.

Stiffness
Where Made Ground is cohesive, the coefficient of volume compressibility (m_v) can be approximated by;

$m_v = 1 / (f_2 \times N)$,
[Stroud and Butler, 1975]

where f_2 can be taken as 0.65 MN/m^2 based on a plasticity index of 16%.

Therefore a coefficient of volume compressibility (m_v) is estimated as 0.38 m^2/MN.

For granular Made Ground a value for drained Young's Modulus can be approximated by;

$E'/N = 1.25$ MPa,

[Clayton, CIRIA Report 143, Standard Penetration Test (SPT): Methods and Use, 1995]

Therefore E' is 5 MPa for shallow foundations based on SPT results.

California Bearing Ratio (CBR) tests were carried out on two remoulded Made Ground samples. These laboratory test results are 6.9% and 5.7% at the top of the samples and 12% and 10% at the bottom of the samples.

8.2 London Clay Formation

Classification
Natural moisture content measurements of samples from the London Clay Formation varied from 13% to 37%. Ntural Moisture content can be seen to decrease with depth across the London Clay Formation, with the minimum value being reached at approximately 14.0 mbgl. Atterburg Limits were calculated for forty one samples of the London Clay Formation. Most samples were classified to be clay of either high or very high plasticity. An average plasticity index of 43% can be adopted for design in the London Clay Formation.

Strength
SPT N values in measured within the London Clay Formation are from 4 to 273. The largest value corresponds to a test carried out where mudstone was encountered with in the London Clay Formation. Therefore the largest result is not characteristic for the stratum and a more representative maximum value is 66. There is a distinct increase in SPT N with depth, therefore design values can be expressed as a linear equation;

$N = 2z$,

Where z is depth in metres below ground level and is greater than 4.5 metres.

Undrained shear strength in the London Clay Formation can be approximated from SPT results using the equation;

$c_u = f_1 \times N$,
[Clayton, CIRIA Report 143, Standard Penetration Test (SPT): Methods and Use, 1995].

Where f_1 can be taken as 5 for the London Clay Formation.

This gives undrained shear strength in the range of 20 kN/m^2 to 330 kN/m^2, when eliminating uncharacteristically high values for SPT N.

A limited number of shear vane tests were carried out in the London Clay Formation, the results of which showed an increase with depth. At less than 7.0 mbgl undrained shear strength can be taken as 30 kN/m^2 based on hand vane results. While there are few hand vane results taken from a depth of 7.0 mbgl and below, the results show a sharp increase. The minimum result for undrained shear strength is 106 kN/m^2. This may indicate the boundary between the unweathered and weathered London Clay Formation.

Twenty undrained triaxial compression tests were carried out on samples of the London Clay Formation. The undrained shear strength values were 43 kN/m^2 to 408 kN/m^2. Once again these results increase with depth. Considering results of triaxial compression tests, SPTs and shear vane tests, undrained shear strength of the London Clay Formation can be represented by a linear equation depending on depth. The characteristic undrained strength is represented by the following formula;

$$c_u = 12.5z - 18 \text{ kN/m}^2$$

where z is equal to depth in metres below ground level and is greater than 4.5 metres.

Consolidated undrained triaxial shear strength tests were conducted on samples of the London Clay Formation. This testing can be used to derive effective stress shear strength parameters for the London Clay Formation. The peak principal effective stresses measured in each test can be combined to provide the mean principal stress and peak shear stress for each test, this data is then plotted to provide an effective stress chart which can then be transformed to a Mohr Coulomb failure envelope. Based upon this approach the following Mohr Coulomb failure parameters were derived:

$$\varphi' = 31°;\ c' = 5\text{kN/m}^2$$

Effective stress shear strength parameters can also be derived from correlation with plasticity index. The embedded retaining walls design guide, CIRIA C580, provides recommendations for this correlation which indicates a critical effective angle of shearing resistance (φ'_{crit}) of 21°. This is a critical angle of shearing resistance whereas the figure derived from the testing represents a peak angle which provides some explanation of the discrepancy in these figures. However based upon past experience in similar materials the following characteristic peak values are proposed for design purposes:

$$\varphi' = 25°;\ c' = 2\text{kN/m}^2$$

Stiffness for use in the design of Shallow Foundations
A value for undrained Young's Modulus (E_u) can be found from;

$$E_u = 360 \times c_u$$

[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore, undrained Young's Modulus is represented by the following equation;

$E_u = 4500z - 6480$ kN/m².
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Effective Young's Modulus is found from the equation;

$E' = 0.6 \times E_u$.
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore;

$E' = 2700z - 3888$ kN/m².

The coefficient of volume compressibility can be approximated by;

$m_v = 1 / E'$,
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore; $m_v = 1000 / (2700z - 3888)$ m²/MN.

Stiffness for use in the design of Retaining Structures
A value for undrained Young's Modulus (E_u) for the design of retaining structures can be found from;

$E_u = 800 \times c_u$.
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore, undrained Young's Modulus is represented by the equation;

$E_u = 10000z - 14400$ kN/m².

Effective Young's Modulus is found from the equation;

$E' = 0.6 \times E_u$.
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore;
$E' = 6000z – 8640$ kN/m².

The coefficient of volume compressibility can be approximated by;

$m_v = 1 / E'$,
[Padfield and Sharrock, CIRIA Special Publication 27, Settlement of Structures on Clay Soils, 1983]

Therefore;

$m_v = 1000 / (6000z – 8640)\ m^2/MN$.

8.3 Geotechnical Parameters

Geotechnical parameters for the purposes of design have been derived from both the in-situ and laboratory testing. These are summarised in **Table 8.3.**

Strata type	**Standard Penetration Test SPT N**	**Undrained Shear Strength (kN/m^2)**	**Coefficient of Volume Compressibility for shallow foundations m_v (m^2/MN)**	**Coefficient of Volume Compressibility for retaining structures m_v (m^2/MN)**	**Effective Young's Modulus E' (MPa)**
Made Ground	4	24	0.38[1]	NR	5[2]
London Clay Formation[3]	2z	12.5z - 18	1000/(2700z–3888)	1000/(6000z–8640)	8 – 171[4]

NR no result

[1] where Made Ground is cohesive

[2] where Made Ground is granular

[3] where z = depth in metres below ground level and is greater than 4.5 metres

[4] based on depths of 4.5 mbgl and 30 mbgl, for shallow foundations and retaining structures

Table 8.3 Summary of Geotechnical Soil Properties

9. GEOTECHNICAL ASSESSMENT

The geotechnical assessment has been based on in-situ testing, soil descriptions recorded on the exploratory hole records, the laboratory test results and the interpreted ground model described in Section 8.

9.1 Foundation Assessment

9.1.1 Piled Foundations

Tension piles will be required to resist the large uplift loads in braced bays and to prevent chimney overturning.

A value for the peak adhesion factor (α) of 0.5 is recommended for use with appropriate factors of safety for ULS pile design taken from Eurocode 7.

9.1.2 Ground Improvement

Ground improvement at site is proposed for the construction of the tipping hall floor slabs, site hard-standing, and access roads. Ground improvement is proposed to consist of the excavation of Made Ground to a depth of approximately 1 metre below the proposed formation level, this material will then be replaced with compacted granular fill. Areas of adequate formation beneath the access roads and hardstanding will only be treated with surface compaction.

9.2 Floor Slabs

Considering the variable nature of Made Ground, the use of ground bearing floor slabs is only recommended once appropriate ground improvements, as detailed in section 9.2.1, have been carried out so as the impact of any differential settlement between the piled foundations and ground bearing slabs can be mitigated. Alternatively ground floor slabs should be constructed as a suspended floor.

9.3 Buried Concrete

Twenty soil samples were tested to assess the risk to buried concrete from sulphate and acid attack. The results indicate a pH range of 7.4 to 9.3. Testing of sulphate 2:1 soil/water extract gave results of 20 mg/l to 1900 mg/l.

The results of the sulphate testing were assessed following the guidance outlined in BRE Special Digest 1, Concrete in Aggressive Ground (2005). Based on the model for mobile water the Sulphate Class for this site is DS-3 and the mix design for buried concrete corresponds to Aggressive Chemical Environment for Concrete (ACEC) class AC-3.

9.4 Excavations and Retaining Walls

9.4.1 Waste Bunker

This is the larger of the two proposed bunkers and the internal dimension of the proposed structure is to be 15 metres wide and 29.5 metres long. The final excavation level is expected to be 13.5 metres below ground level which includes an anticipated thickness of the base slab of 1.5 metres.

As the excavation will be carried out below the water table, an ‘impermeable’ wall system should be used to cut off water flows from retained soil into the bunker excavation.

In order to reduce settlement of the ground surrounding the bunker due to side wall deflection, during and post construction, an anchored secant piled wall scheme is recommended. The use of ground anchors or other supports can reduce wall movements from a magnitude of hundreds of millimetres to no more than 30 millimetres movement in total. The use of anchors will also limit wall moments, hence allowing the use of a thinner wall section and shorter piles.

The formation of the bunker excavation will induce immediate and long term heave which will impose load on the base. Therefore casting of base slab should be delayed as long as possible to minimise the amount of long term heave. In addition a layer to prevent heave at the base of the structure is recommended.

There is a risk of uplift of the base slab due to water pressure. Therefore the scheme should use either a deep base slab spanning between bunker walls or tension piles in order to reduce the depth of the base slab.

9.4.2 Ash Bunker

This smaller storage bunker will also require excavation in ground with high water table. The final excavation level is expected to be 7 metres below ground level including 1 metre thick base slab. A similar secant piled wall to the waste bunker is recommended to form the ash bunker with a shorter embedment depth and without propping.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 Geotechnical

Foundations

Tension piles will be required to restrain the base slab for the bunkers.

Buried Concrete

The recommended concrete sulphate class for this site is DS-3 and the mix design for buried concrete corresponds to Aggressive Chemical Environment for Concrete (ACEC) class AC-3.

Excavations

Two significant excavations, the waste bunker and the ash bunker are proposed below the water table.

During construction the bunkers will require water ingress into the excavations to be prevented using an impermeable wall system. The use of sump pumping or similar to collect water from excavation during construction is recommended. An anti-heave layer on the base of the larger structure is recommended.

The use of a hard-firm secant pile walls will help to minimise water flow from seepages. The use of ground anchors and other supports for the larger bunker will reduce wall movements and allow a thinner wall section and shorter piles. In addition an anchored secant piled wall scheme will reduce settlement of the ground surrounding the bunker.

Roads and Hardstanding

Ground improvement will be required for the construction of the tipping hall floor slabs, site hard-standing, and access roads. This will include varying degrees of excavation and replacement with compacted granular fill, and surface compaction.

Further Ground Investigation

The scope of ground investigation carried out at this stage was curtailed by the presence of buildings and the operations on the site at present. Once the site is cleared it is proposed that further ground investigation be carried out to supplement the findings of this phase of investigation and provide more detail on the substructure works for the key structures to be built and other key aspects of infrastructure.

Based upon our current review the additional investigation will comprise the following:

3 no. cable percussion boreholes 2 in the vicinity of the bunker, 1 to be located within the turbine hall all to be drilled to 30m depth.

A range of soil and groundwater samples will be taken from each exploratory holes based upon the following strategy:

Borehole samples	Granular soils – one bulk sample at 1m depth intervals Cohesive soils – one UT100 sample at 1.5m depth intervals disturbed samples to be taken from the UT100 cutting shoe Groundwater – take a groundwater sample at every water strike
In-situ Testing	Granular soils – one SPT at 1m depth intervals Cohesive soils – one SPT at 1.5m depth intervals

Groundwater monitoring	Install standpipe piezometers within the London Clay Formation in both boreholes.
Laboratory testing	Range of testing to be carried out on recovered samples including soil classification, CBR, undrained shear strength, effective stress, and oedometer tests plus pH value and water soluble sulphate.

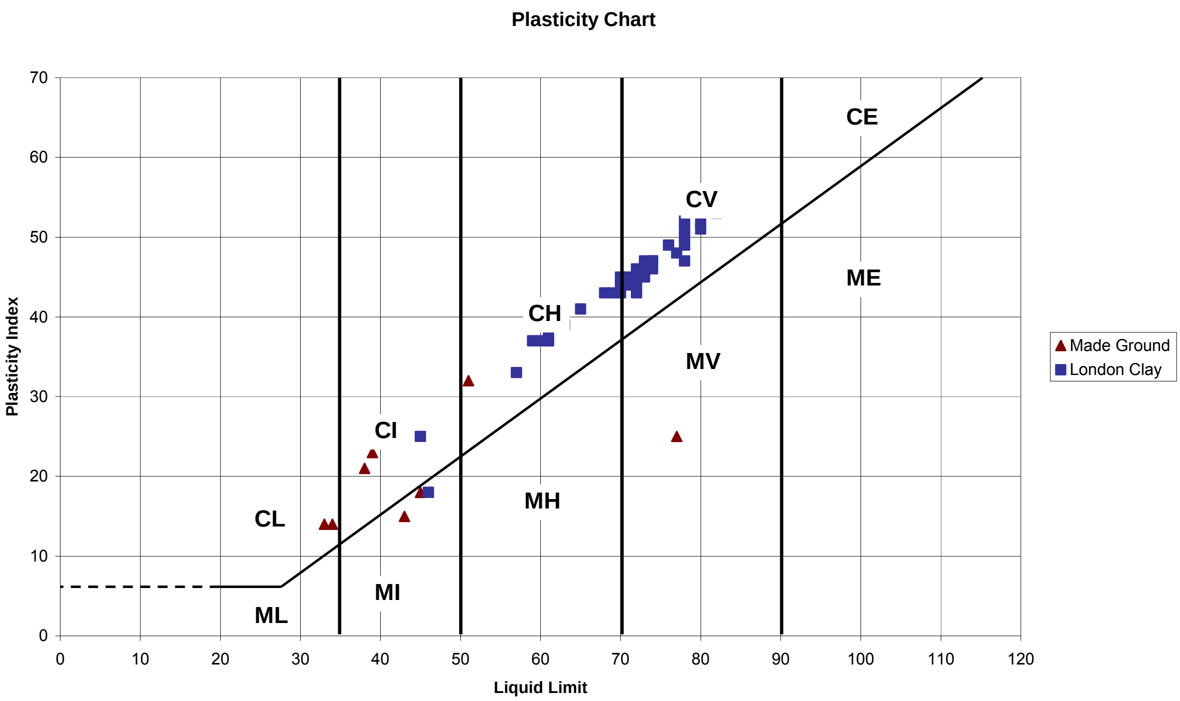
FIGURES

Appendix A

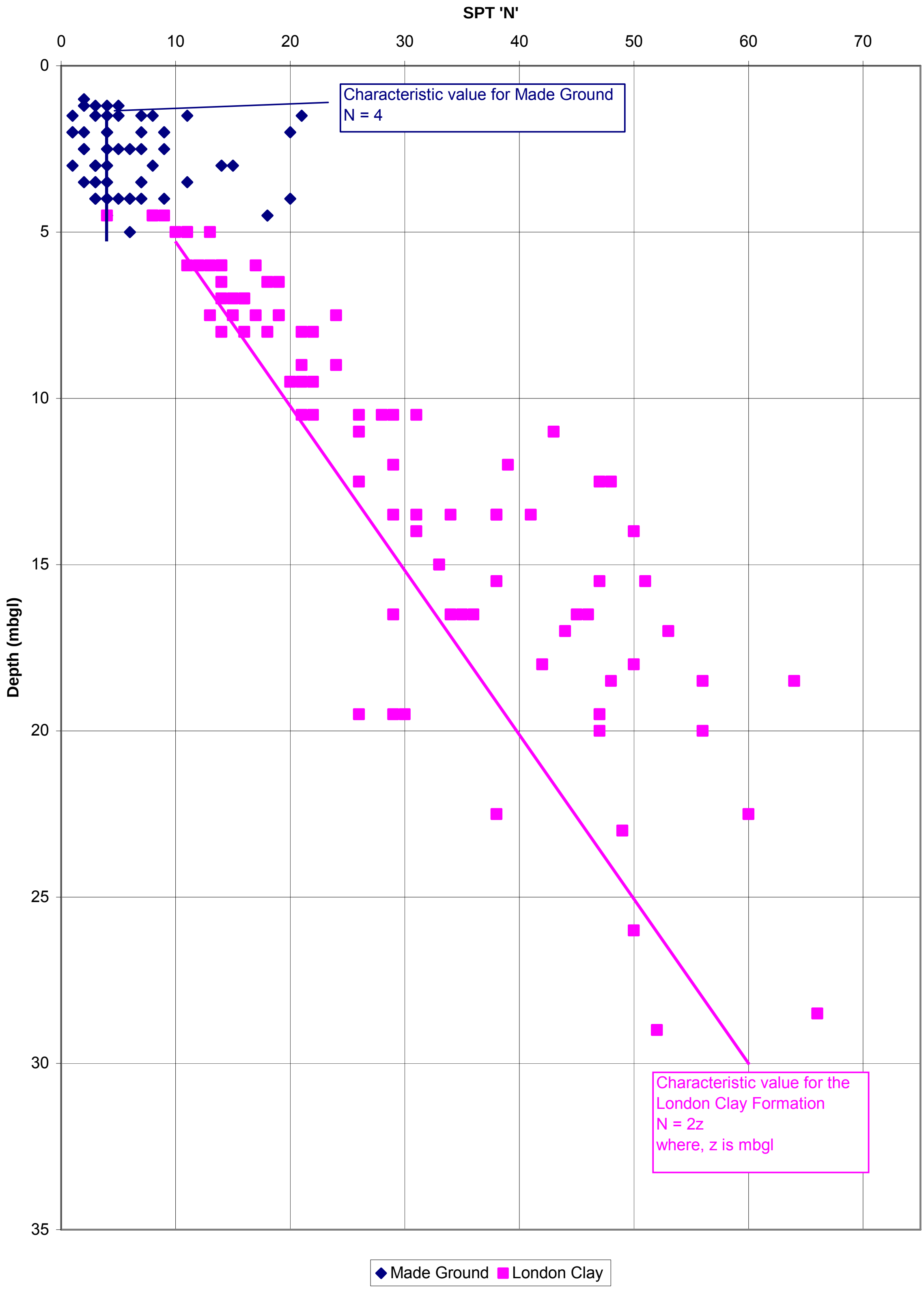
Geotechnical Engineering Limited Factual Report

Appendix B

Graphical Geotechnical Data


Plasticity Chart
Plasticity Index
Liquid Limit
0
10
20
30
40
50
60
70
0
10
20
30
40
50
60
70
80
90
100
110
120
CL
ML
CI
MI
CH
MH
CV
MV
CE
ME
Made Ground
London Clay

SPT 'Nn with Depth



Undrained Shear Strenth with Depth, Tested by Various Methods