BUROHAPPOLD ENGINEERING

Folkestone Harbour

Geotechnical Investigation and Interpretative Report

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1 The Site

1.1 General

On behalf of Folkestone Harbour (CP) Ltd, BuroHappold Engineering (BuroHappold) were commissioned to undertake an outline geotechnical assessment of Folkestone Harbour, Folkestone, Kent to assist with the redevelopment of the harbour into an iconic residential and retail development.

The extent of the area investigated is shown with a red line boundary on Figure 1-1 (see also drawing GSK-01 in Appendix) and is hereafter referred to as 'the Site'. A geoenvironmental investigation was also undertaken, the findings of which are presented in an separate report [1].

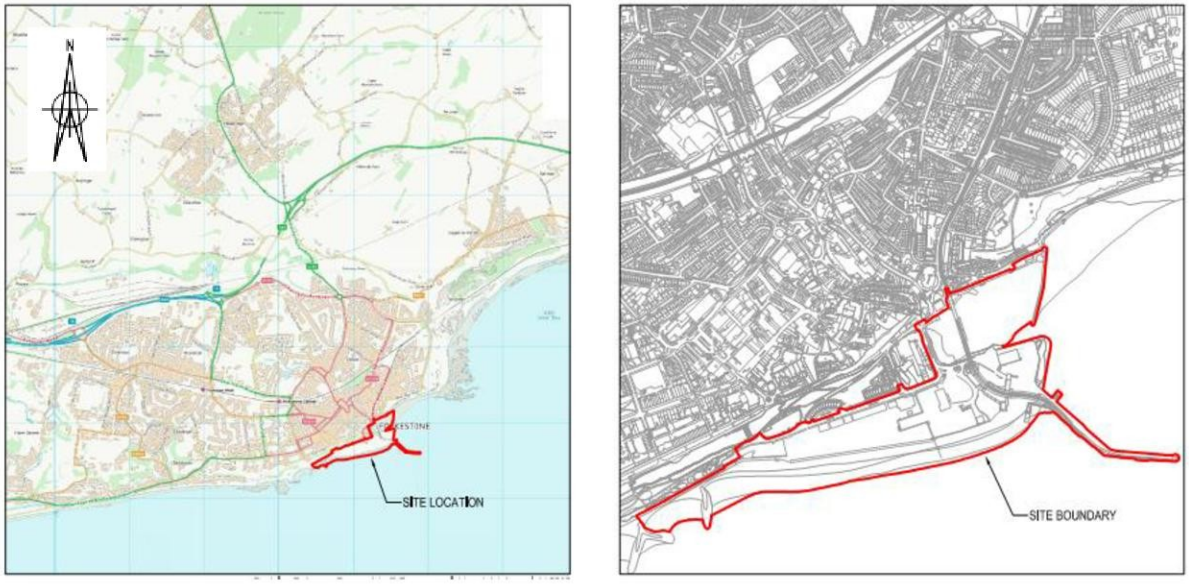


Figure 1 Site location and red line boundary

The aims, objectives and information sources used as part of this outline geotechnical assessment for the Site are summarised below.

1.2 Aims and objectives

The aims of the ground investigation were to assess the outline geotechnical and geoenvironmental conditions for the Folkestone Harbour redevelopment. This report deals with only the geotechnical aspects of the investigation.

To achieve the above aims, the objectives of the geotechnical investigation at the Site are to provide an outline assessment of the following conditions:

- Shallow deposits (Made Ground and Beach Deposits) ability to act as a bearing strata for a possible piling mat or shallow foundations for lightly loaded columns;
- Define geotechnical behaviour of the Beach Deposits, Sandgate Formation and Hythe beds;
- Define the long term strengths of the materials;
- Obtain the parameters necessary to allow modelling the impact of the proposed development

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- Outline Foundation Design
- Preliminary concrete design class assessment

1.3 Information Sources

The principle sources of historic information were taken from the previous desk study by Arup [2] and historic photographs available from online sources. This report is based upon information obtained these third party sources, together with observations from the Site walkover survey and data from the recent intrusive ground investigation (undertaken between 4th April to 6th May 2016) [3].

The third party data has been accepted at face value and has not been independently verified. BuroHappold can therefore give no warranty, representation or assurance as to the accuracy or completeness of such information.

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2 Site Setting

2.1 Location and Description

The Site is located in Folkestone, a coastal town situated along the English Channel in Kent, south east England (see Figure 2-1). The British National Grid Reference of the approximate centre of the Site is TR23129 35702. The majority of the Site is located to the west and south of Folkestone Harbour and includes the shingle beach foreshore and the formerly developed hardstanding area immediately behind the foreshore. The Site comprises an area of approximately 10.8 hectares (inclusive of the existing pier).

Based on the ground conditions encountered during the intrusive ground investigation, the Site has been divided into four main areas (A, B, C and D) and each area will be considered separately throughout this report, see below.

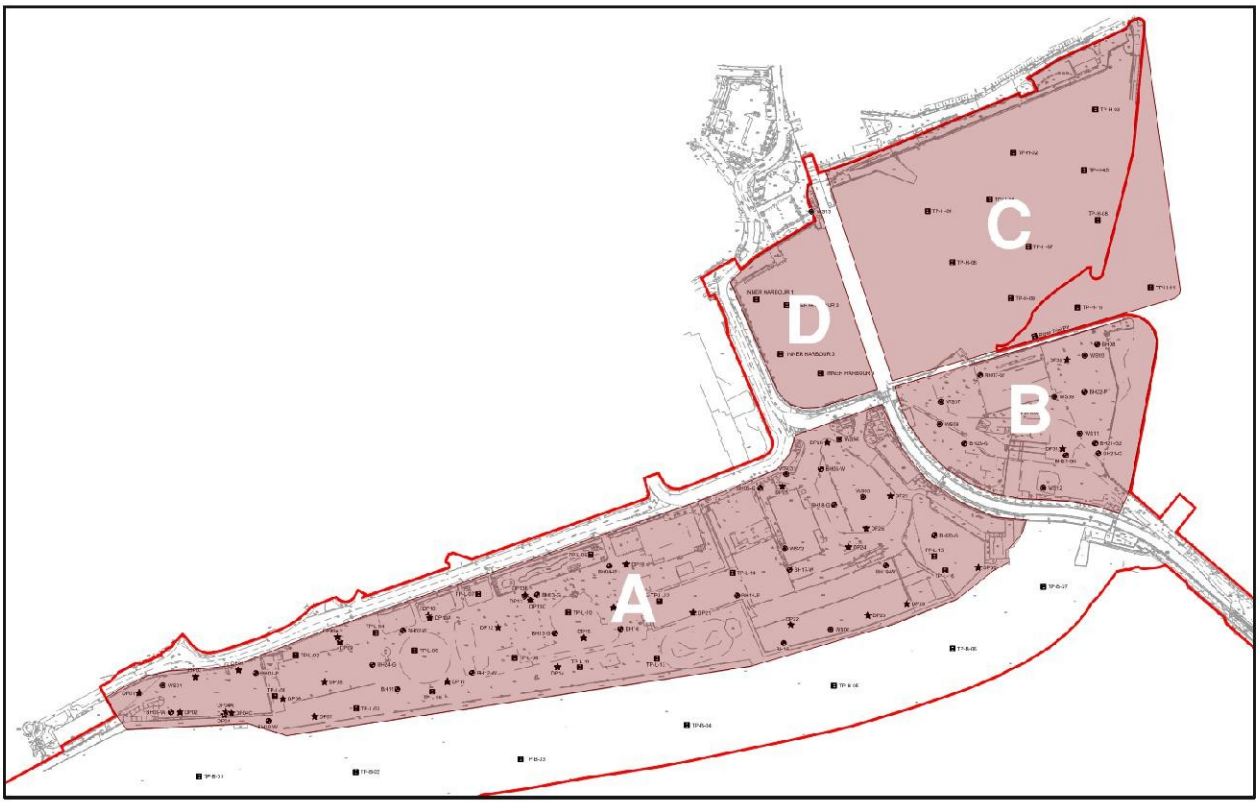


Figure 2 Site plan showing areas A, B, C and D

The area to the south of Area A includes the beach foreshore, a footpath intersects Area A and the beach foreshore, see Photograph 3. The thin section of land to the north of Area A is bounded by Marine Parade, see Photograph 1.

The majority of the Site (Areas A and B) is generally flat at approximately 6 m above Ordnance Datum (OD) with the beach sloping downwards to the south towards the English Channel. The inner and outer harbour (Areas C and D) areas range from approximately 0.6 m OD to 2 m OD.

The southern and eastern areas of the Site are bounded by the beach up to the Old Folkestone Pier. The northern parts of the Site are bounded by Lower Sandgate and Marine Parade roads. The inner and outer harbour areas are bounded to the north by a concrete jetty.

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The western extent of the Site (Area A) is bounded by an area of vegetation that lies between the shingle beach foreshore and Lower Sandgate Road. The eastern site boundary (Area B) is marked by the New Harbour Arm and the outer harbour wall which separates the outer harbour from Sunny Sands Beach.

Excluding the harbour, the Site is roughly rectangular in shape, spanning approximately 900 m east to west. It is narrowest at its western end (approximately 25 m), and widens to the east to a maximum of approximately 180 m. The eastern extent of the Site is intersected by a derelict rail line that joins the Old Folkestone Pier to the southeast of the Site (it is this intersection that divides Area A and B).

The Site includes active public car parks (Area A contains a carpark in each of its western and eastern extents, see Photograph 2 and Photograph 4) and a concrete surfaced lorry park in the south-east extent of Area A. Sections of the lorry park are hired to local waste companies to store skips and other equipment. Multiple spoil heaps are scattered across the eastern extents of Area A, these mainly comprise demolition debris. The central part of Area A is predominantly open, asphalt covered land (frequently occupied by passing trade e.g. circus, fun fairs). The northern extent of Area A comprises four areas of gravel that. Area B comprises a large temporary car park which provides access to the newly renovated harbour arm.

Marine Parade and Lower Sandgate Road and their adjoining footpaths are public rights of way. The land to be developed at Folkestone Harbour along the beach front is privately owned; however, it is openly used by the public for pedestrian beach access.

A number of buried utilities are present on parts of the Site including, but not limited to telecommunications, electricity, water, foul and storm drainage, and gas. The majority of the utilities are understood run along Marine Parade and Lower Sandgate Road. Spurs run to into the Site to the location of disused or demolished structures that may be cut off or removed. Some drainage remains, with a number of drain covers present.

The outer harbour (Area C, Photograph 5 and Photograph 6) has been dredged multiple times in the past and at low tide a relatively competent sandy base is exposed, whereas the inner harbour (Area D) generally has a soft silty base.



Photograph 1 View looking east across the Site (Area A) from the far W car park. Note Marine Parade in the left of the photograph (cars parked along it).



Photograph 2 View looking west across the 'Pay and Display' car park. Note there is a change in level within the car park (highlighted in the photograph). Cliffs within the right of the photograph contain rock from the Hythe Beds.

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Photograph 3 View looking west across the foreshore. in the foreground the public footpath that runs along the southern part of Area A can be identified. Note groynes can be seen along the beach.



Photograph 4 View looking south across the lorry and car parks (eastern section of Area A).



Photograph 5 Outer harbour (Area C) at low tide whilst trial pitting works were being carried out. View looking north-east.



Photograph 6 Outer harbour including the old railway bridge (Area C) at high tide. View looking north.

2.2 Proposed Development

The current proposed development (at the time of writing the final masterplan has not been confirmed) comprises multiple 4/5 storey residential town houses within Area A (see Figure 2). South of these town houses will be a boardwalk connecting the harbour arm in the east to the public coastal gardens in the west. Beyond this will be a number of beach houses that it is assumed will have piled foundations.

In later phases of the development of the area commercial buildings and residential apartments (approximately 12 storeys) have been proposed within Area B.

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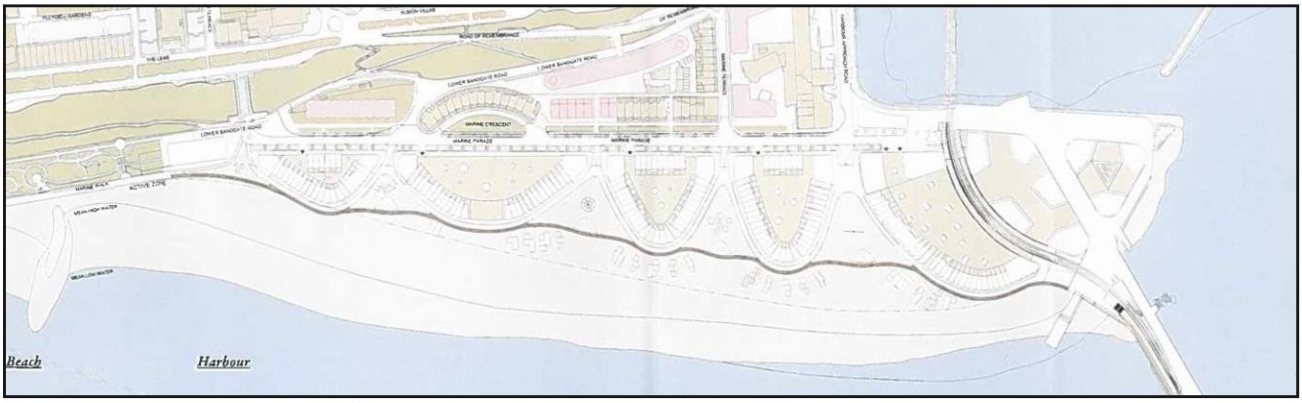


Figure 3 Architectural plan across the site of the proposed development (subject to change)

2.3 Site history

The historical use of the Site is outlined in Table 2-1, a full discussion of the Site history is provided in the Arup desk study [2].

Table 2-1 Summary of Site History

Dates	Site History
Pre 1872	The 1794 historical harbour masterplan shows an 'L' shaped rock pier in approximately the same location as the present day harbour approach road, suggesting that the historical pier may be incorporated into the current structure. Pent Stream is located just to the west of the pier; it is this stream that now flows into the inner harbour. The 1829 drawings provide detail on the then proposed construction works. These drawings include some detail of the existing construction at the time, which comprised masonry construction with piles (likely sheeted wooden piles), at depth.
1872-1873	The eastern portion of the Site was dominated by the port facilities associated with the harbour and pier. The Site was accessed by both road and rail. Rail access was via a swing bridge that separated the smaller inner harbour (to west) and larger outer harbour (to east). The railway sidings serviced the wharfs of the outer harbour and also the pier. Buildings within the Site area included harbour station, custom house and a goods warehouse. Maps from 1873-1875 show a timber yard located to the immediate north of Marine Parade and to the immediate east of the present day Marine Crescent. To the north of the Site were various developments on a reclaimed area at the foot of the old cliff line. These developments included: Pavilion Hotel (with formal gardens), Harbour House, terraced houses (including Marine Parade and Marine Crescent), a timber yard and baths.
1898	Between 1873 and 1898 the harbour facilities had been enlarged. The harbour station was expanded and new tracks/sidings were created. The pier was remodelled (extended and widened). To the west of the railway sidings a formal garden with a band stand was built. To the south of these gardens small structures and a ramp (possibly boat huts and an accompanying boat ramp) are shown to be present. The west side of the Site contained small structures including boat houses, Camera Obscura and a lifeboat house (with a ramp). Victoria Pier is shown to be present (1888).
1907	The harbour station was altered and additional railway tracks and sidings present. The main pier was widened and extended further, whilst the short pier was shortened. To the north of the Site the Royal Pavilion Hotel was enlarged. The former Harbour House building was demolished. The timber yard is no longer shown. A number of groynes were shown on the foreshore at the east end of the Site.
First World War	An air raid occurred on 1st February 1917 at various localities in and around Folkestone. It was reported that 163 bombs fell, killing more than 60 people, the majority at Tontine Street in the centre of Folkestone.

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Dates	Site History
1938	The beach area was landscaped and utilised for leisure facilities including the construction of the 'Marine Promenade' (open air Bathing Pool, Rotunda, Boating Lake, and Marine Gardens Pavilion). Some changes occurred to the harbour and station, including enlargement of the goods warehouse. The railway sidings were extended westwards. On the beach/foreshore the former groynes were no longer shown.
Second World War	Due to Folkestone's proximity to France, the area was subjected to considerable attack and damage, including bombs dropped by air and also long range shelling from the French coastline. Both town and harbour were hit on numerous occasions. Given the strategic importance of the harbour (potential landing site for invasion) the harbour had numerous defences including mines. The mines were placed on land, on the beach and in the water. 1941 aerial photography shows bomb damage to the inner harbour. Further information is in the BAE detailed UXO risk assessment report [4].
1967-1978	The eastern portion of the Site underwent a number of changes, including a new lorry park that extended onto part of the former beach area.
2008	Different phases of coastal protection measures were created (shingle and sand beach, no original wave cut rock platform is exposed). Between 1999 and 2008 new beach defences were created (curved groynes/moles).

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3 Site Investigation

3.1 2016 Ground Investigation

The scope of the geotechnical portion of the intrusive ground investigation was designed to meet the aims and objectives stated in Section 1.2 above. Geotechnical investigation contractor services were provided by Structural Soils on behalf of BuroHappold, with part-time supervision provided by a BuroHappold Ground Engineer. The site investigation took place between 4th April to 6th May 2016 and a factual report was produced by Structural Soils [3].

The works undertaken by Structural Soils included both geotechnical and geoenvironmental aspects; however, only the scope of the geotechnical portion of the intrusive ground investigation is considered in this report and comprised:

- 27 No. boreholes (cable-percussive with 8 rotary follow-on) to depths of between 15m and 30m bgl;
- 16 No. window sample holes to a maximum depth of 3.4m bgl;
- 31 No. dynamic probe penetrometer tests to a maximum depth of 1.2m bgl;
- 35 No. machine dug trial pits (11 No. in the outer harbour) to a maximum depth of 3m bgl;
- 4 No. hand dug (inner harbour) trial pits to a maximum depth of 2m bgl;
- Installation of groundwater monitoring wells in a selection of nine boreholes (BH02-W, BH04-W, BH06-W, BH07-W, BH09-W, BH10-W, BH12-W, BH17-W, BH19-W), with groundwater level loggers installed in four groundwater monitoring wells and one barometric logger installed in a fifth groundwater monitoring well;
- One round of groundwater monitoring;
- Programme of groundwater level monitoring with data loggers; and
- Programme of geotechnical laboratory testing of various strata.

3.2 Cable Percussive Holes

A total of 27 boreholes were drilled using a cable-percussive method. This method was used to obtain samples from the Made Ground, Beach Deposits and some of the Sandgate Formation. Where harder ground was encountered – siltstones and sandstones of the Sandgate Formation or Limestones from the Hythe Beds, the cable-percussive method was aborted and rotary core follow on was used. The average depth reached with the cable-percussive method of drilling was approximately 15m bgl.

3.3 Rotary Bored Holes

Rotary core follow on was used for 8 of the boreholes to reach greater depths and obtain samples of the material from the Sandgate Formation and Hythe Beds. This also allowed the thickness of the Sandgate Formation to be proved across the site.

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3.4 Trial Pits

A total of 11 trial pits were carried out within the outer harbour and a further 4 small trial pits within the inner harbour. The outer harbour pits were approximately 2m in depth, however, due to unstable conditions (collapsing sides/water encountered) some pits did terminate short of this.

A total of 24 trial pits were carried out across the remainder of the Site. These pits allowed samples to be retrieved from the Made Ground (if present) and the beach gravels for both geoenvironmental and geotechnical purposes.

3.5 Encountered Geology

The information presented in Table 3-1, Table 3-2 and Table 3-3 summarises the ground conditions encountered within each area (A, B and C respectively) across the Site during the intrusive ground investigation. A cross section illustrating the geology can be found in Appendix

Table 3-1 Ground conditions of the western section of the Site (Area A)

Geology	**Geological Description**	**Bottom Depth** Min, Max (m bgl)	**Top Level** Min, Max (mOD)	**Thickness** Min, Max **[Av.]** (m)	**Notes**
Asphalt	-	0.04, 0.23	+2.8, +7.09	0.04, 0.23 **[0.1]**	Asphalt not always present, coach park is predominantly reinforced concrete.
Concrete	Occasionally reinforced.	0.08, 0.4		0.1, 0.33 **[0.22]**	
Made Ground	Clayey, very sandy GRAVEL, with low cobble content. Sand is fine to coarse. Gravel is angular to rounded fine to coarse brick, concrete, flint and chert. Cobbles are sub-angular concrete.	0.25, 2.2		0.1, 2.1 **[0.5]**	
Beach Deposits	Multi-coloured [yellow, brown, orange, white] sandy GRAVEL. Gravel is sub-rounded to rounded flint. Sand is medium to coarse. Beach deposits become more sandy with depth.	3.8, 11	+2.74, +6.12	3.46, 10.3 **[7.77]**	Water Strike ranges from 0.3-7.5 m bgl, predominantly within the Beach Deposits.
Sandgate Beds	Alternating layers of dense to very dense dark greenish grey, slightly clayey SAND with firm to stiff dark grey sandy silty CLAY. Sand is fine to medium. Occasionally slightly gravelly (shell fragments). With depth, very weak thinly laminated MUDSTONE and occasional SANDSTONE layers. Bedding within the mudstone is horizontal. The bedding fractures are medium to widely spaced undulating rough infilled with greenish grey fine to medium sand.	25.6, 29.2	-6.91, -1.11	18.2, 19.9 **[18.8]**	
Hythe Formation	Weak light grey silty fine grained distinctly weathered LIMESTONE with occasional inclusions of unweathered light grey medium grained SANDSTONE.	>40.4	-25.87, -19.01	>11.3	Located within BH03G, BH12W, BH01P.

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	Weak dark grey partially weathered MUDSTONE is also present. Fractures are medium to widely spaced stepped rough infilled with silty clay and fragments of sandstone Medium strong thinly laminated sandy SILTSTONE containing halite crystals and rare shells.				

Table 3-2 Ground conditions of the eastern section of the Site (Area B)

Geology	**Geological Description**	**Bottom Depth;** Min, Max (m bgl)	**Top Level** Min, Max (mOD)	**Thickness** Min, Max **[Av.]** (m bgl)	**Notes**
Asphalt	-	0.03, 0.35	+6.06, +6.32	0.03, 0.35 **[0.1]**	Asphalt not always present, coach park is predominantly reinforced concrete.
Concrete	Occasionally reinforced.	0.14, 0.4		0.06, 0.35 **[0.23]**	
Made Ground	Slightly clayey, sandy GRAVEL. Gravel is sub-rounded to angular brick, glass, flint, ceramic, concrete, chalk fragments. Sand is fine to coarse.	0.23, 4		0.03, 3.85 **[0.96]**	
Beach Deposits	Multi-coloured [yellow, brown, orange, white] sandy GRAVEL. Gravel is sub-rounded to rounded flint. Sand is medium to coarse. Beach deposits become more sandy with depth.	9.5, 14	+2.25, +5.29	7.7, 11.6 **[9.77]**	Water Strike ranges from 1.1-7.1 m bgl, predominantly within the Beach Deposits.
Sandgate Beds	Alternating layers of dense dark greenish grey silty SAND and firm (becoming stiff with depth) dark grey CLAY. Sand is fine to medium containing frequent pyrite grains. Clay occasionally silty. Some layers of extremely fissured distinctly weathered dark grey sandy SILTSTONE and extremely weak thinly laminated greyish black silty MUDSTONE. Bedding horizontal. Occasional pyrite nodules up to 10mm present.	27.26, 39.9	-6.31, -4.68	14.76, 27.4 **[20.93]**	
Hythe Formation	Medium strong, distinctly weathered, light grey LIMESTONE. **Fracture Set 1**: Dipping 10-20°, very close to closely spaced undulating rough moderately wide aperture infilled with dark grey gravelly clay. Gravel is angular fine limestone. **Fracture Set 2**: Dipping 70-90°, planar rough, tight to partly open aperture, no filling/clean surfaces.	-	-31.19, -30.44	>16.6	Located within BH05G, BH07, BH15, BH22P.

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Table 3-3 Ground conditions of the outer harbour (Area C)

Geology	Geological Description	Bottom Depth (m bgl)	Top Level Min, Max (mOD)	Notes
Beach Deposits	Light grey to black slightly silty, slightly gravelly SAND with some organic odours. Sands are fine to medium. In some locations this grades to a slightly sandy, slightly gravelly CLAY. In other locations black to brown silty, very sandy gravels are present. Gravels are angular to sub-angular, fine to coarse. Sand is fine to medium. Concrete, brick and flint fragments are occasionally present. Clayey sandy silt lenses are also present.	1.5 - 2.2	-1.11, +0.79	There are no definitive bands of one material.

3.6 Groundwater

During the intrusive ground investigation, nine boreholes were installed as groundwater monitoring wells. Within the harbour, observation pits were carried out during low tide and water ingress was noted at approximately 1.5-2.2 m bgl; at high tide the water within the pits was noted to rise up to ground surface.

Monitoring - Groundwater levels were monitored on five occasions between the end of May to the end of June 2016. The monitoring was undertaken using a hand-held dip-meter. A summary of the groundwater monitoring results is presented in Table 3-4 below.

Table 3-4 Groundwater levels as dipped during manual monitoring round

Sample	No. of occasions water level recorded	Response zone (m bgl)	Level (mOD)	Water level range as dipped during manual monitoring round (m bgl)
BH02-W	5	1 to 8	+2.7 to +2.58	3.56 to 3.68
BH04-W	5	1 to 6	+3.44 to +1.43	2.66 to 4.67
BH06-W	5	1 to 8	+1.51 to +0.63	4.57 to 5.45
BH07-W	5	1 to 8	+2.79 to +0.56	3.40 to 5.63 (dry on one occasion)
BH09-W	5	1 to 8	+3.14 to +2.14	2.82 to 3.82
BH10-W	5	0.5 to 8.5	+2.47 to +2.1	3.33 to 3.70
BH12-W	5	0.5 to 8.5	+2.83 to +2.32	3.44 to 3.95
BH17-W	5	2 to 6	+1.91 to +1.8	4.29 to 4.40
BH19-W	5	1 to 8	+1.55 to +1.41	4.87 to 5.01

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Dataloggers - Groundwater dataloggers were installed within four groundwater monitoring wells (BH02-W, BH07-W, BH10-W, BH17-W) and a barometric logger in BH05-G between 2 June to 9 June 2016 to monitor groundwater fluctuations.

The data logger located in Area B, adjacent to the harbour wall (BH07-W) appears to be strongly tidally influenced with fluctuations in level within tidal cycles of up to approximately 3 metres at depths of between approximately 2.5 and 5.5 m bgl. This range indicates that the top of groundwater is present within the beach gravel deposits.

One location to the southwest of Area A (BH10-W) appears to be less tidally influenced with the fluctuations in level within tidal cycles of approximately 0.1m at depths between approximately 3 and 3.5. Two locations in the central area of Area A (BH02-W and BH17-W) do not appear to be tidally influenced with groundwater depths at approximately 3.5 and 4.5 m bgl respectively. The data from these locations also indicates that the top of groundwater is present within the beach gravel deposits.

3.7 Laboratory Testing

All geotechnical and contamination testing was carried out in order to determine geotechnical parameters for the various strata encountered and for potential site contamination levels to be established. All testing was carried out by Structural Soils Limited.

The geotechnical testing listed below was carried out in accordance with BS1377(1990), BRE SD1 and ISRM (1981 and 1985).

Type	Remarks
Classification Tests	
Moisture Content Determination	28no
Atterberg Limit Determination (Liquid and Plastic Limits)	24no
Particle Size Distribution Analysis	187no
Shear Strength – Total Stress	
Consolidated Drained Direct Shear Tests (set of three 60mm x 60mm x 20mm thick samples)	2no
Chemical Tests	
pH and Water Soluble Sulphate Content of Soils	14no
Rock Tests	
Point Load Index	33no
Uniaxial Compressive Strength Testing	16no

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4 Geotechnical Parameters

4.1 Design Ground Model

The ground investigation at the Site has revealed the geological sequence to comprise, Made Ground, Beach Deposits, Sandgate Beds, Hythe Formation as discussed in detail in Table 3-1, Table 3-2 and Table 3-3 for various areas across the Site. Table 4-1 below summarises the stratigraphy.

Table 4-1 - Stratigraphy

Stratum		Geological Description
Pleistocene and Recent	Storm Gravel Beach Deposits	Dense fine to coarse sandy flint GRAVEL.
Cretaceous -Lower Greensand	Sandgate Beds	Alternating layers of dense to very dense, slightly clayey SAND with firm to stiff sandy silty CLAY. With depth, very weak thinly laminated MUDSTONE and occasional calcareous weakly cemented SANDSTONE.
	Hythe Formation	Weak, silty, distinctly weathered sandy LIMESTONE with occasional glauconitic sandy MUDSTONE.

A design stratigraphy has been determined for the Site from all the available information. This is presented below in Table 4-2. A design water level has been determined using the highest tide level. The information gathered from the monitoring site visits were slightly limited and did not target particularly high tide times. A design level of 3.85mOD has been used in design.

Table 4-2 Design Ground Model

Stratum	Elevation (top of strata, mOD)	Metres below ground level	Proven Range of Thickness
Capping Layer (overlying asphalt and concrete)	+6	0	1
Made Ground			
Upper Beach Deposits	+5	1	5
Lower Beach Deposits	0	5	-
Sandgate Beds	-5	10	19.5
Hythe Formation	-24.5	29.5	Not proven

Table 4-3 Summary of Geotechnical Parameters

Strata	Bulk Unit Weight (kN/m^3)	PL, LL, PI (%)	ϕ'_{peak} (°)	$\phi'_{critical}$ (°)	Undrained Shear Strength, C_u (kPa)	Drained Young's modulus, E_v' (MPa) *(Stiffness)*	Undrained Young's modulus, E_{uv} (MPa)	Uniaxial Compressive Strength (MPa)
Made Ground	20	-	34	33	-	13	-	-
Upper Beach Deposits	20	-	35	33	-	14 – 27	-	-
Lower Beach Deposits	20	-	39	35	-	32 – 45	-	-

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Sandgate Formation	Sandy clay	20	20, 50, 30	37	33	34	27 – 60	-	-
	Clay and Mudstone	20	-	-	-	175+12z	60 – 96	78 - 128	-
Hythe Beds		-	-	-	-	-	-	-	60

Made Ground

Hardstanding covers approximately 90% of the Site. Hardstanding is underlain typically by a clayey, very sandy gravel, with low cobble content. The gravel is typically brick, concrete, flint and chert with occasional cobbles of concrete.

No bulk density measurements were taken within the Made ground. A bulk unit weight of 20kN/m^3 is recommended for design.

Standard penetration tests (SPT N Values) were taken and typically these varied from 7 to 29 (21 tests undertaken), see Figure 5. The slightly higher SPT N Values are taken from the Made ground within Area B (eastern area of the Site, not considered for the first phase of design work). This area had much greater thicknesses of Made Ground (up to 4m), and contained multiple obstructions on boulders of concrete and cobbles of brick.

4.1.1 Strength

For strength calculation the SPT N value has been concerted to $N_{1(60)}$. A moderately conservative choice of design SPT $N_{1(60)}$ value has been selected. $N_{1(60)}$ = 17, i.e. a loose material taking into account the variable nature of the entire area across the Site, see Figure 4.

For granular soils Table 3 from BS8002 recommends the following value for ϕ, note the SPT N value has been calculated as $N_{1(60)}$;

Where A is a measure of angularity

B is a measure of grading

C is a measure of SPT value considering effective overburden pressure – $N_{1(60)}$, see Figure 4

The angularity varies with the wide range of materials within the Made Ground, but may be reasonably approximated in bulk behaviour as 'sub-angular'. Similarly, the soil grading may be reasonably approximated as 'uniform'.

Following Table 3, a conservative estimate of A, B and C is as follows: A = 2, B = 1, C = 1. Therefore:

$\phi'_{peak} = 34°$

$\phi'_{crit} = 33°$

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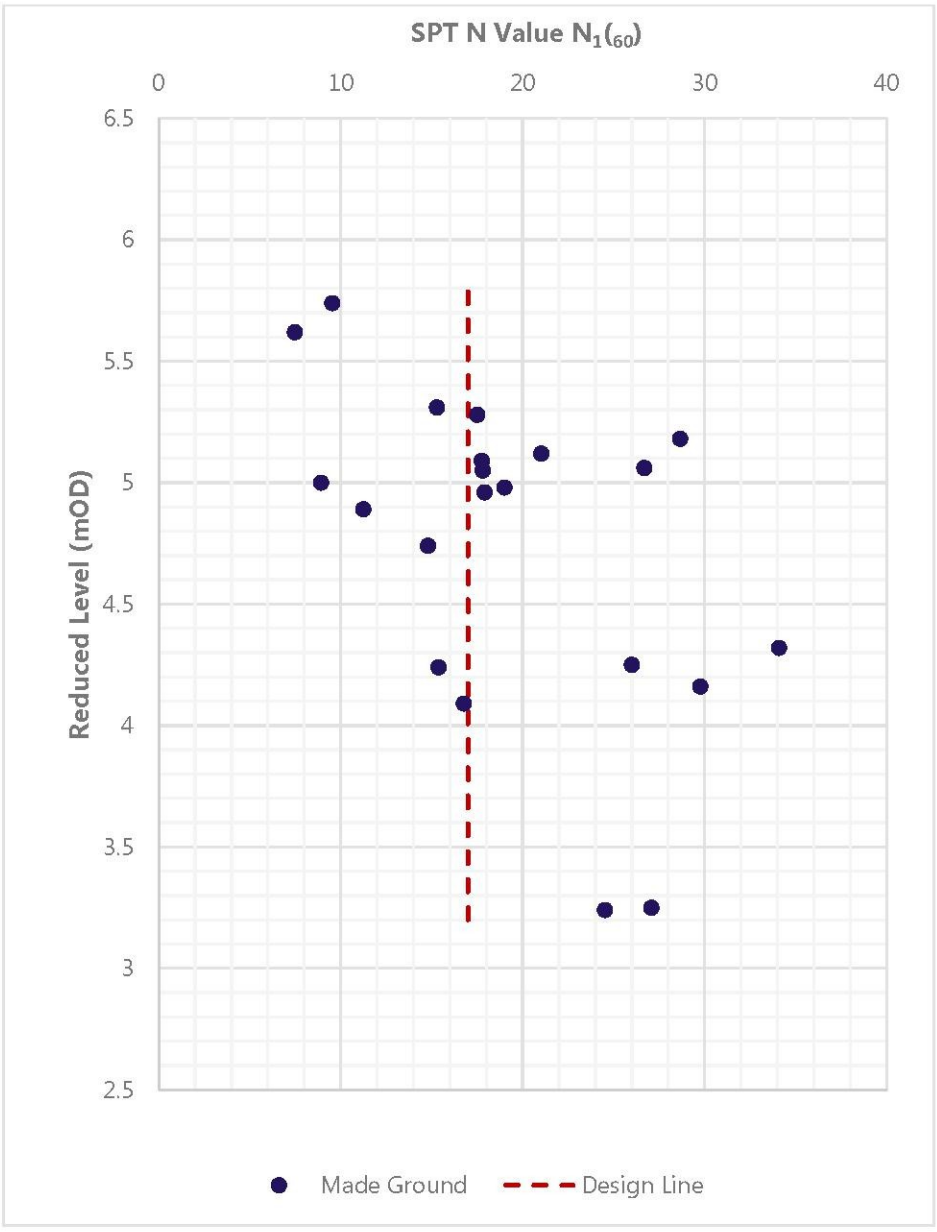


Figure 4 SPT $N_1(_{60})$ value for Made Ground (strength)

4.1.2 Stiffness

For normally consolidated sands and gravel, Stroud (1989) [5] presents a relationship between Young's modulus, E_v', and SPT value, N, for a variety of stress levels. In this case, $E_v' = 1N$ is appropriate for a moderately conservative design approach. For a moderately conservative mean SPT of N = 13 (see Figure 5 below) this gives $E_v' = 13$ MPa

The drained lateral stiffness, E_h', is assumed to be equal to E_v'.

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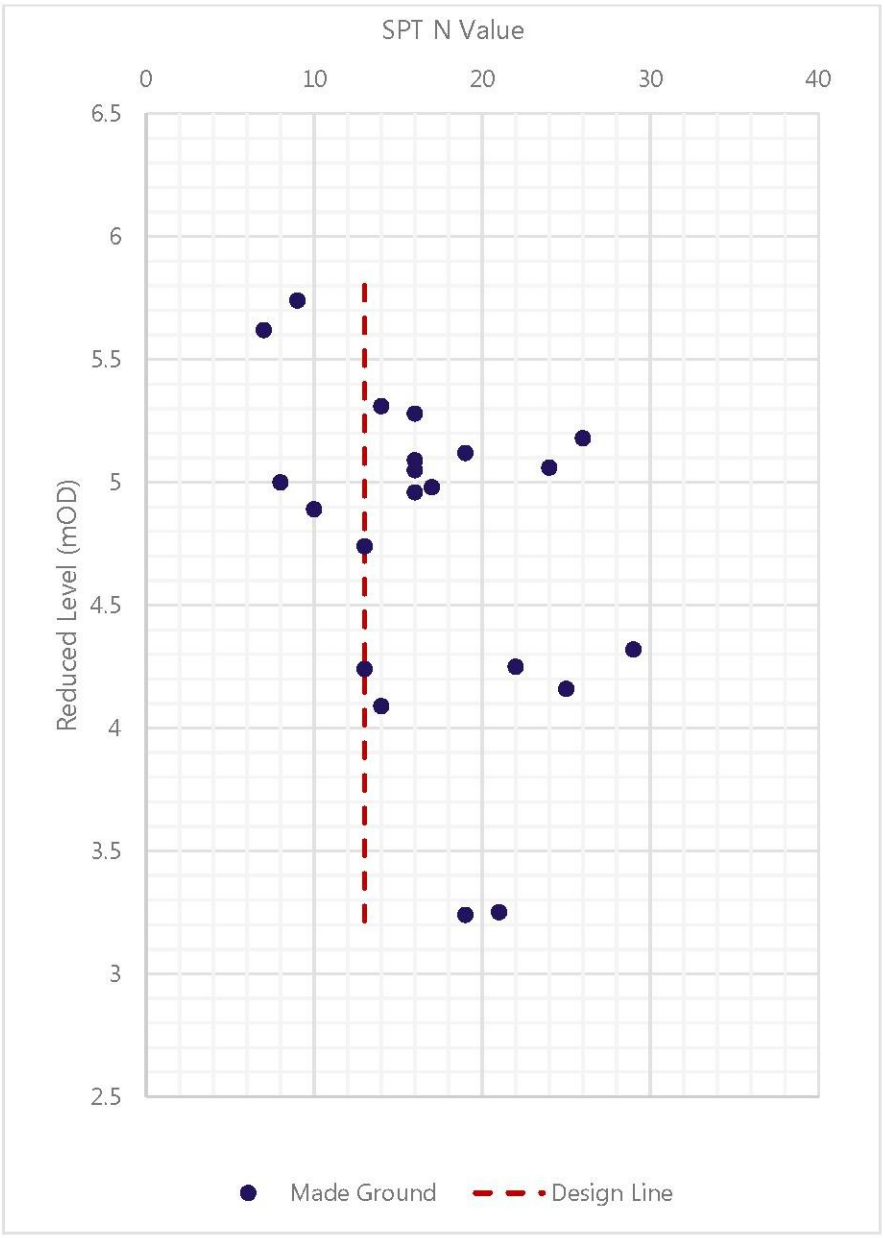


Figure 5 SPT N Value for Made Ground (Stiffness)

4.2 Upper Beach Deposits

4.2.1 Classification

Descriptions from the logs and the particle size distribution results show a relatively moderately graded material (fine sands and medium to course gravels), as shown in Figure 6. The PSD illustrates that a number of samples contain approximately 20-30% fine to medium sand. Two samples contained >50% fine to medium sand, these were TP-L-05 and TP-L-04 (both at 1m*bgl*).

A uniformity coefficient (D60/D10) has been calculated at approximately 2 , representing a uniformly to moderately graded material.

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No bulk density measurements were taken within the Upper Beach Deposits. A bulk unit weight of 20 kN/m^3 is recommended for design.

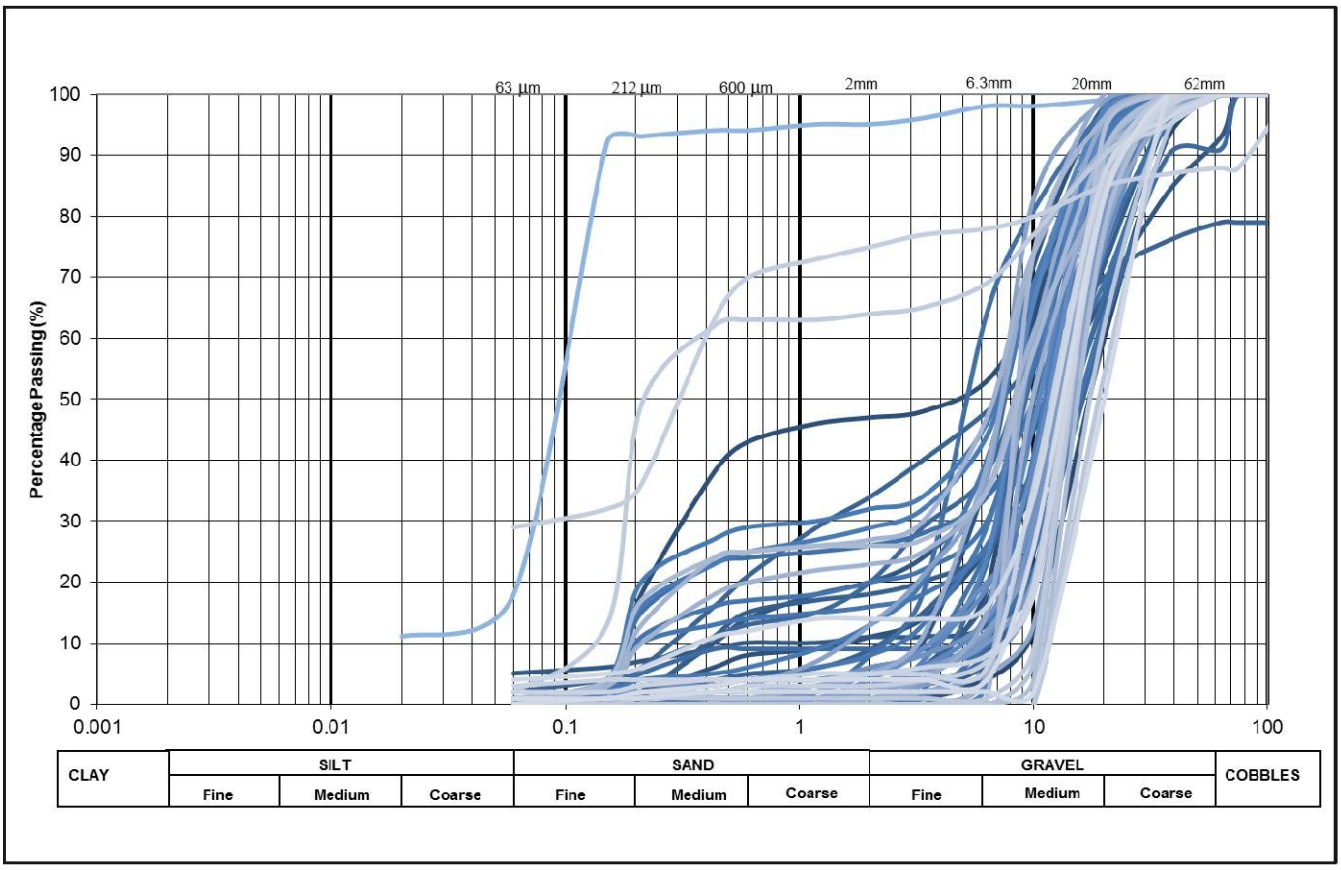


Figure 6 Upper Beach Deposit PSD analysis

4.2.2 Strength

Standard penetration tests (SPT N values) were carried out and typically these varied from 15 to 120 (102 tests undertaken).

For strength calculation, the SPT N value has been converted to $N_{1(60)}$. Overall a moderately conservative choice of $N_{1(60)}$ design for the Upper Beach Gravels has been analysed at N=22, (see Figure 7) i.e. loose to medium-dense.

The angularity is taken here between 'sub-rounded' and 'rounded'. The soil grading may be reasonably approximated as 'uniform', see Figure 6.

Following Table 3 from BS8002, a conservative estimate of A, B and C is as follows: A = 1, B = 2 and C = 2. Therefore:

$\phi'_{peak} = 35°$

$\phi'_{crit} = 33°$

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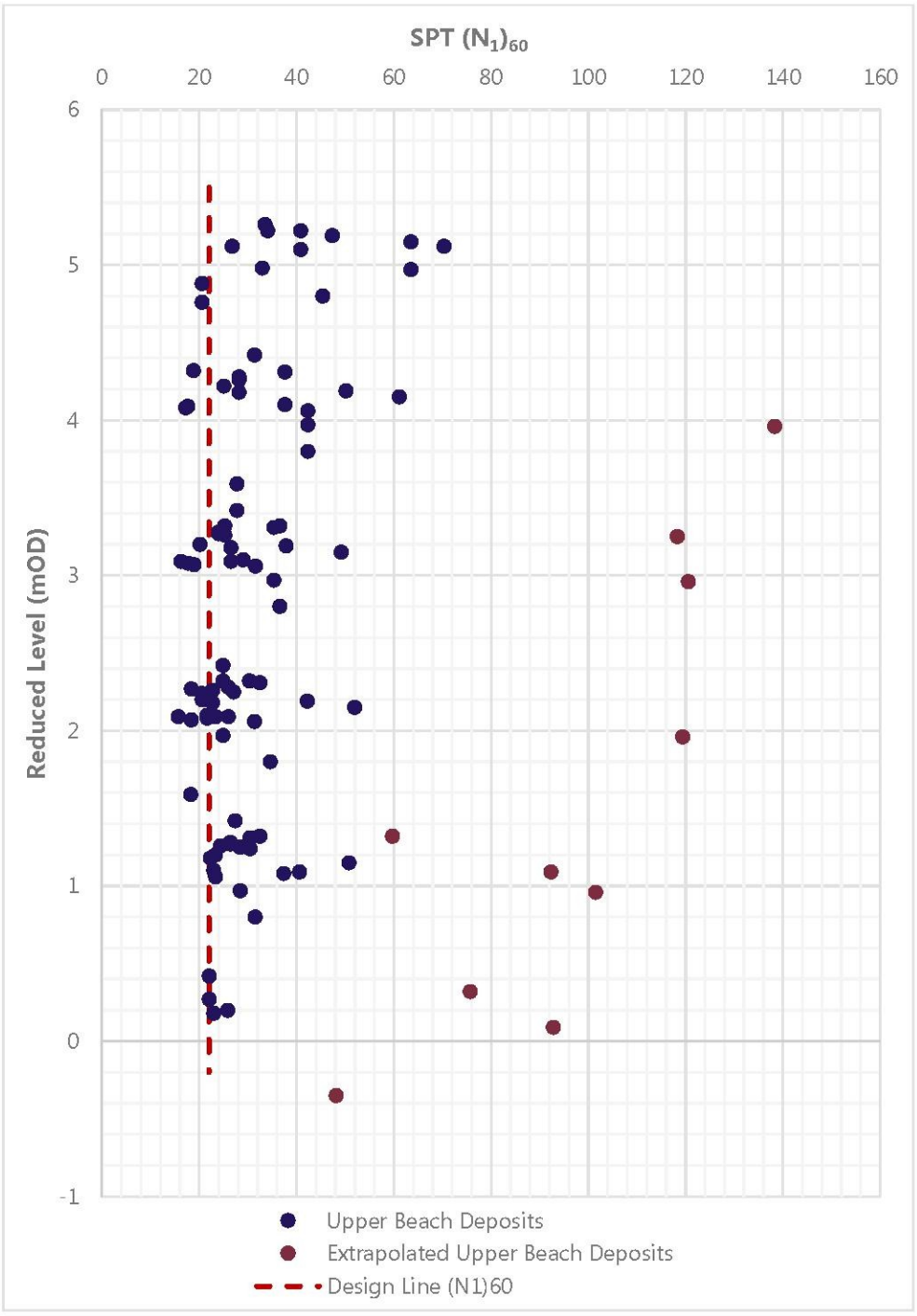


Figure 7 Upper Beach Deposits - SPT $(N_1)_{60}$ values with depth (strength)

4.2.3 Stiffness

For normally consolidated sands and gravel, Stroud (1989) [5] presents a relationship between Young's modulus, E_v', and SPT value, N, for a variety of stress levels. In this case, E_v' = 1N is appropriate for a moderately conservative design approach. For a moderately conservative mean SPT of N = 14+2.2z, from 5.4mOD (see Figure 8 below) this gives E_v' = 14 – 27 MPa.

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The drained lateral stiffness, E_h', is assumed to be equal to E_v'.

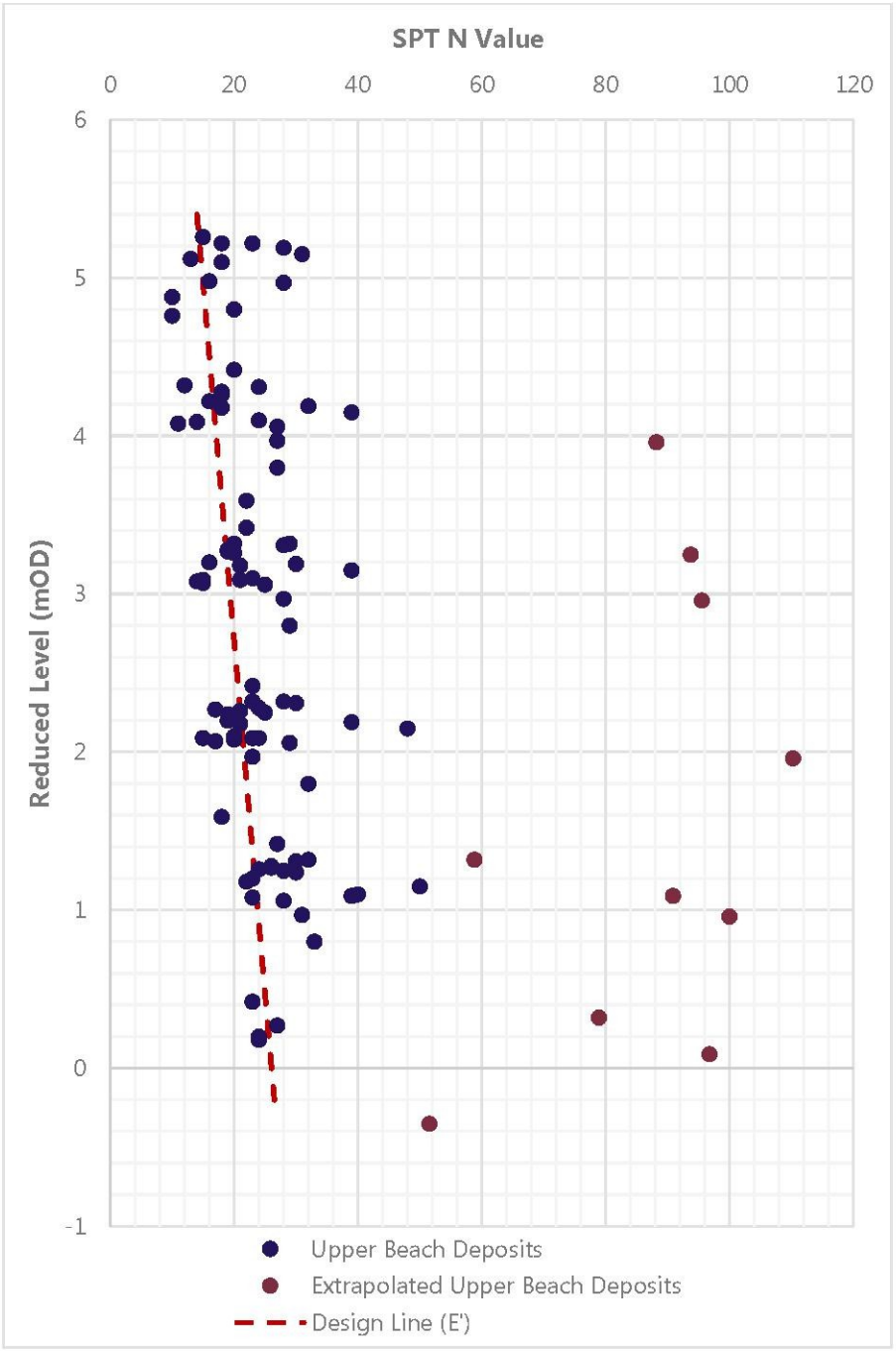


Figure 8 SPT 'N' value with depth (stiffness)

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4.3 Lower Beach Gravels

4.3.1 Classification

Descriptions from the logs and the particle size distribution results show a relatively well graded material (fine sand and medium gravels), as shown in Figure 9. The PSD illustrates that the majority samples are fine sands with some medium to coarse gravels; there are no medium to coarse sands.

A uniformity coefficient (D60/D10) has been calculated as approximately 30, representing a well graded material.

No bulk density measurements were taken within the Lower Beach Deposits. A bulk unit weight of 20kN/m^3 is recommended for design.

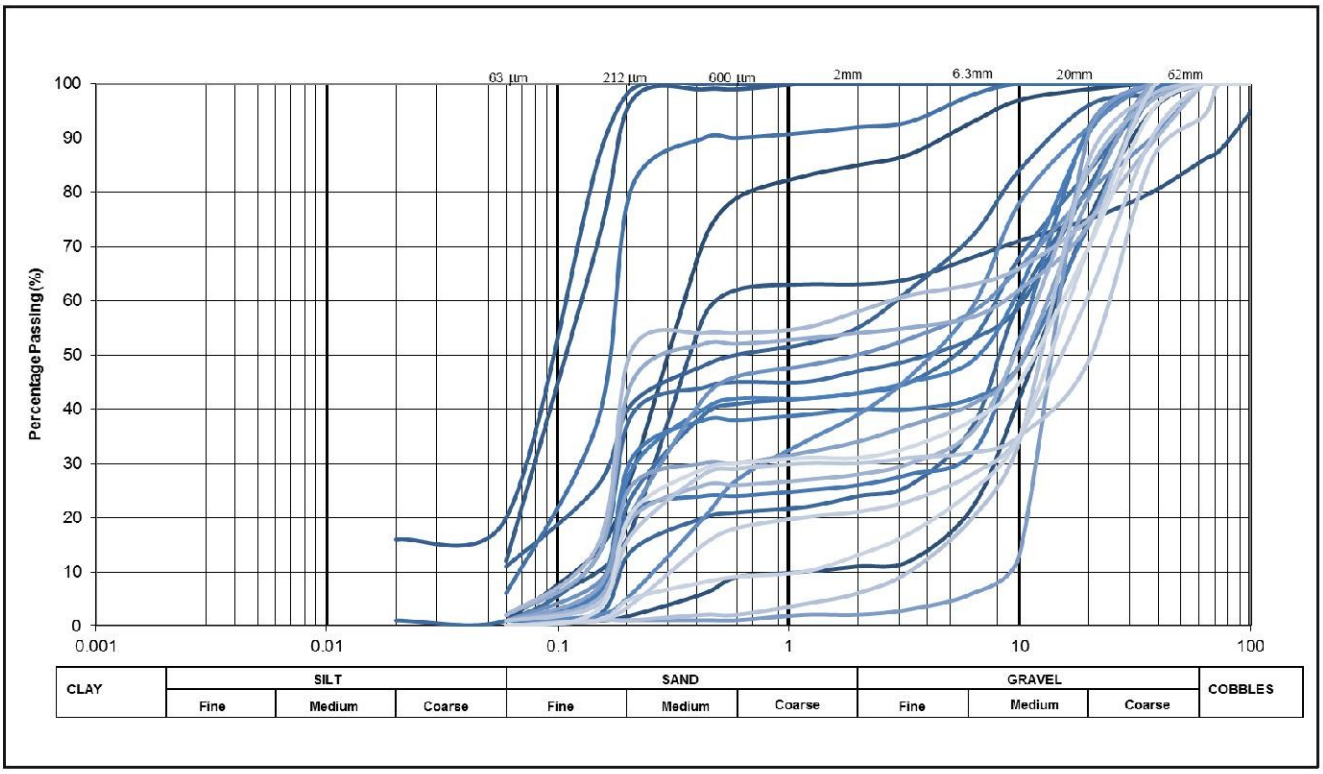


Figure 9 Lower Beach Deposit PSD analysis

4.3.2 Strength

Standard penetration tests (SPT N values) were carried out and typically these varied from 25 to 115 (66 tests undertaken).

For strength calculation, the SPT N value has been converted to $N_{1(60)}$. Overall, a moderately conservative choice of design SPT $(N_1)_{60}$ value for the Lower Beach Gravels has been analysed at N=32, (see Figure 10) i.e. medium-dense to dense.

The angularity is taken here between 'sub-rounded' and 'rounded'. The soil grading may be reasonably approximated as 'moderately graded to uniform', see Figure 9.

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Following Table 3 from BS8002, a conservative estimate of A, B and C is as follows: A = 1, B = 4 and C = 4. Therefore:

$\phi'_{peak} = 39°$

$\phi'_{crit} = 35°$

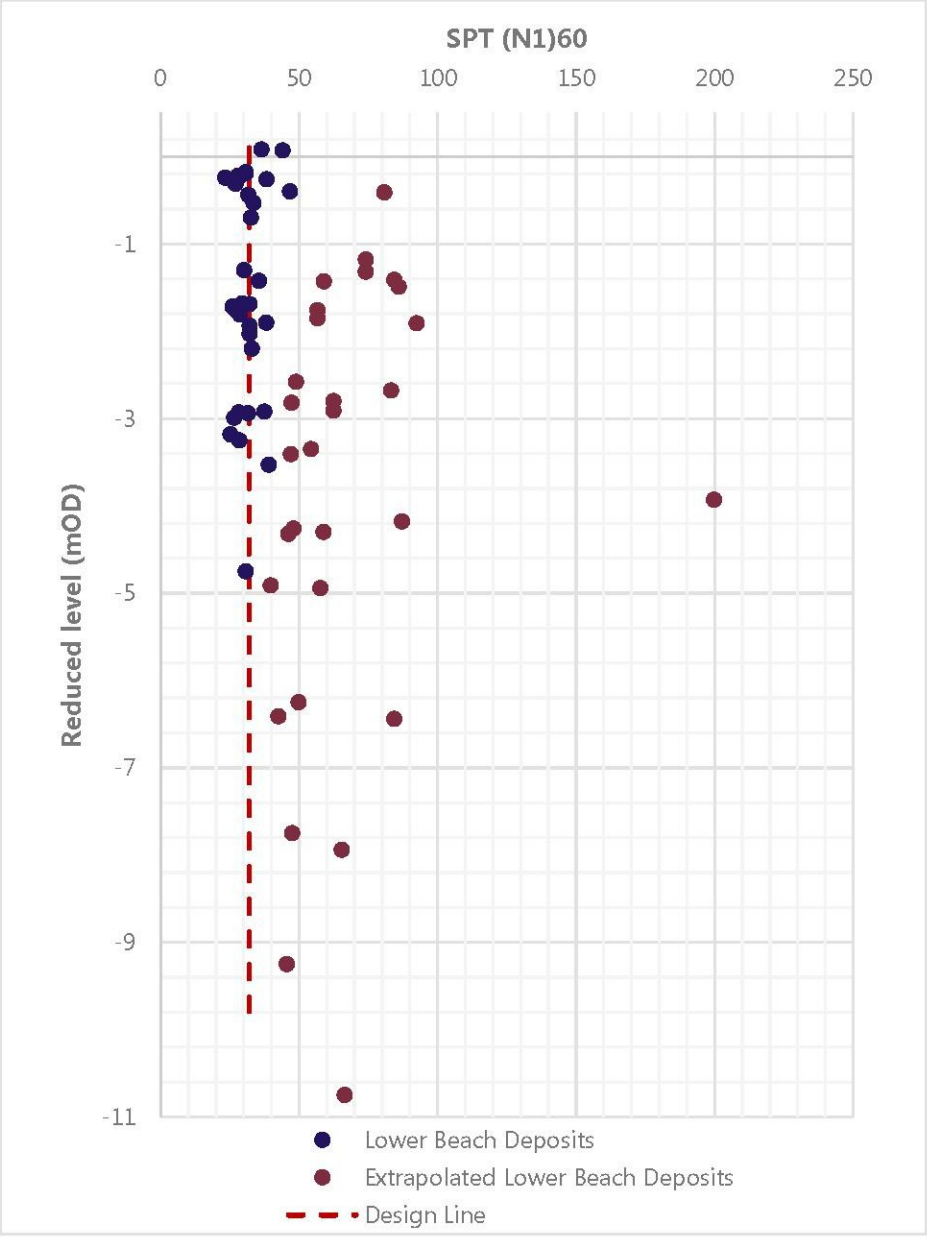


Figure 10 Lower Beach Gravels - SPT $(N_1)_{60}$ Value with depth

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4.3.3 Stiffness

For normally consolidated sands and gravel, Stroud (1989) presents a relationship between Young's modulus, E_v', and SPT value, N, for a variety of stress levels. In this case, $E_v' = 1N$ is appropriate for a moderately conservative design approach. For a moderately conservative mean SPT of N = 32+1.3z, from 0.15mOD (see Figure 11) this gives E_v' = 32 – 45 MPa

The drained lateral stiffness, E_h', is assumed to be equal to E'v.

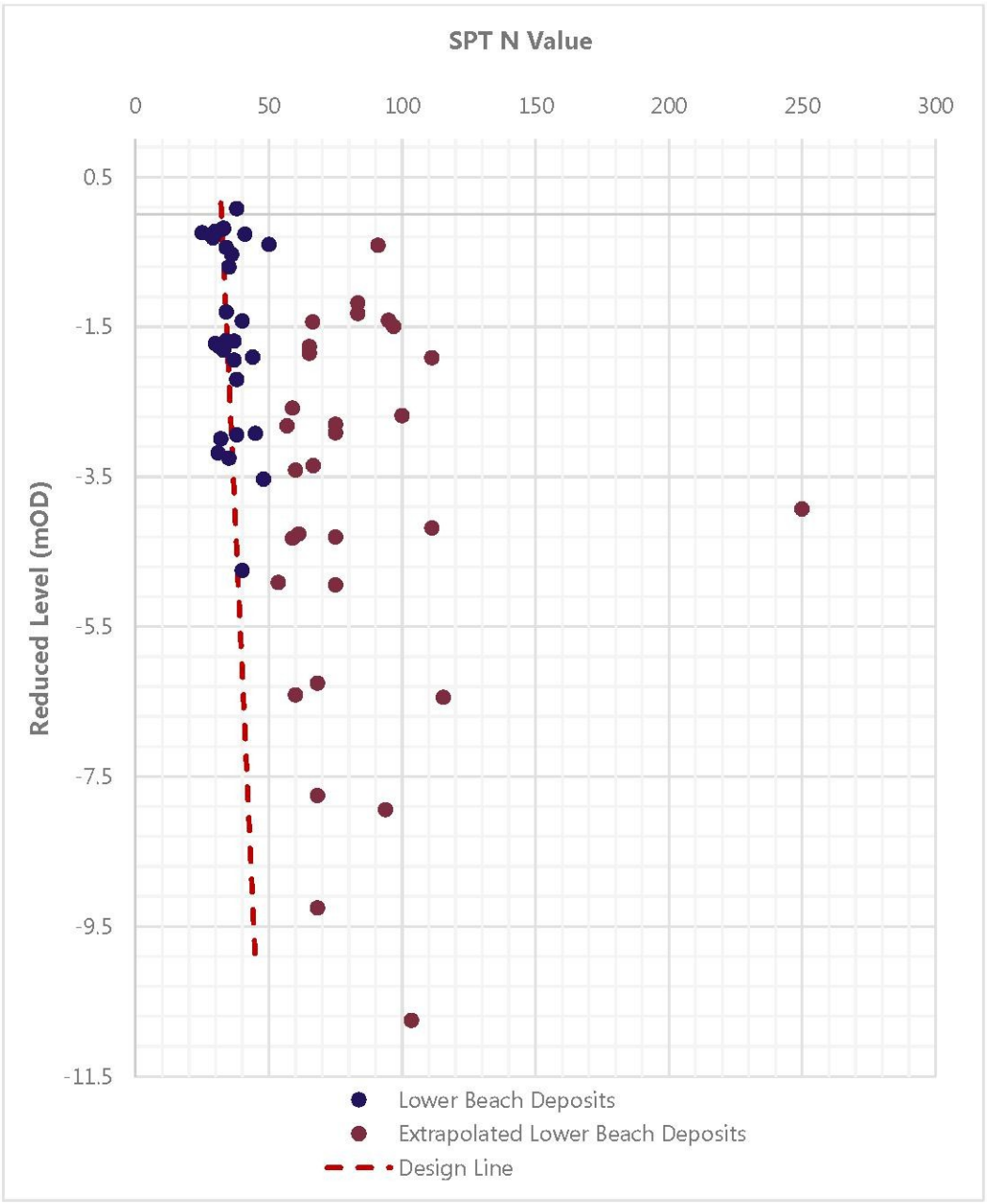


Figure 11 SPT 'N' value with depth (stiffness)

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4.4 Sandgate Bed Formation

The Sandgate bed Formation comprises a sequence of interbedded sands and clays. The material becomes more clayey with depth, eventually grading into a weak weathered Mudstone. Ground movements in clays are usually divided into two distinct phases:

1. Undrained movements occurring over a limited period of time in which excess pore water pressures generated by the change in stress state of the soil do not dissipate. As the pore water pressures remain unchanged, the volume of the soil does not change.

2. Drained movements occur as the clay swells or consolidates and the excess pore water pressures dissipate over time.

4.4.2 Classification

Descriptions from the logs and the particle size distribution results show a relatively uniformly graded material (fine sand and silts/clays), as shown in Figure 12. The PSD illustrates that many of the samples are fine sands. Approximately 11 samples are a fine sand with the remaining samples containing approximately >20% clay/silt (which correlates with the descriptions of from the logs).

A uniformity coefficient (D60/D10) has been calculated at approximately 1.6 to 2, representing a uniformly graded material.

No bulk density measurements were taken within the Sandgate bed Formation. A bulk unit weight of 20kN/m^3 is recommended for design.

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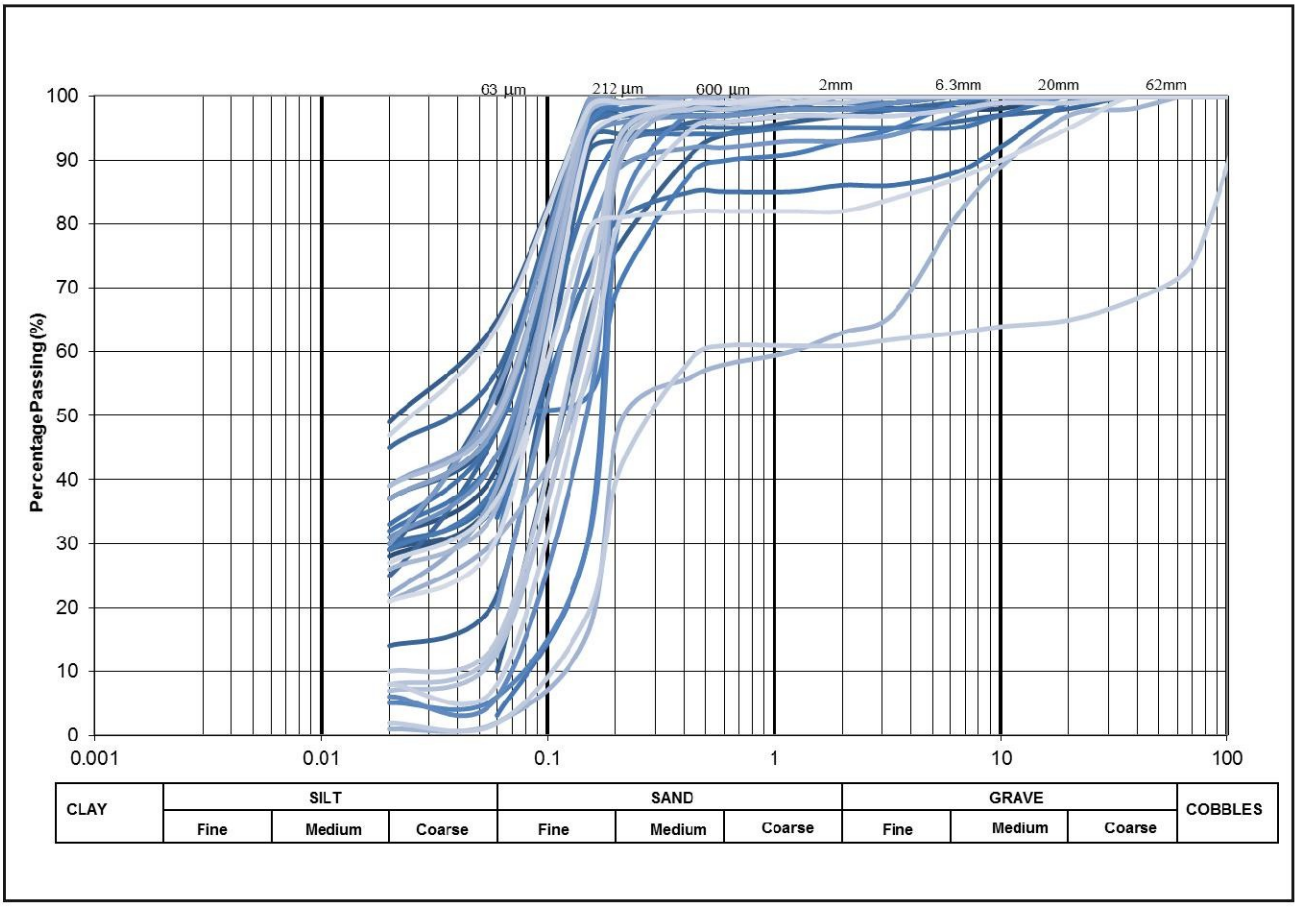


Figure 12 PSD for Sandgate bed Formation

4.4.3 Undrained Shear Strength (clay layers and mudstone)

Stroud (1989), based on the plasticity index of the material, recommends a relationship of C_u = 5N (where N is SPT Value), shows the undrained shear strength correlated from SPTs. It is clear that there is much scatter in the data, however, certain trends can be identified. A moderately conservative estimate of the design line for the lower sandy clay layers is C_u = 34, and a second design line for the clay and weathered mudstone layers (including all extrapolated data) is C_u = 175 + 12z.

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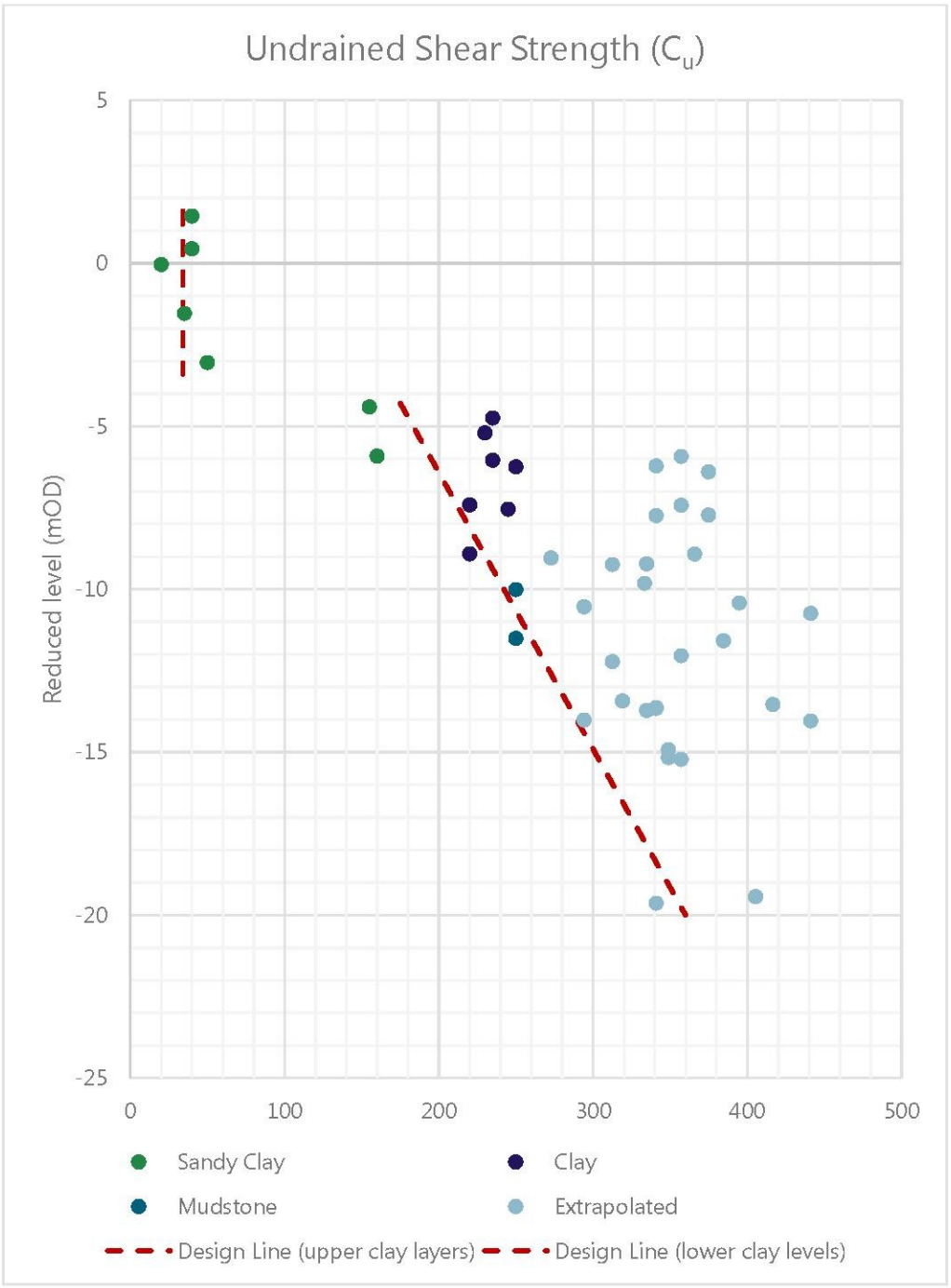


Figure 13 Undrained Shear Strength (C_u) of clays within the Sandgate bed Formation

4.4.4 Drained Strength

4.4.4.1 Granular layers

Standard penetration tests (SPT N values) were carried out and typically these varied from 13 to 40 (66 tests undertaken).

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For strength calculation, the SPT N value has been converted to $N_{1(60)}$. Overall a moderately conservative choice of design SPT $(N_1)_{60}$ value for the Lower Beach Gravels has been analysed at N=34, i.e. medium-dense to dense.

The angularity is taken here between 'sub-rounded' and 'sub-angular'. The soil grading may be reasonably approximated as 'moderately graded to uniform', see Figure 12.

Following Table 3 from BS8002, a conservative estimate of A, B and C is as follows: A = 2, B = 1 and C = 4. Therefore:

$\phi'_{peak} = 37°$

$\phi'_{crit} = 33°$

4.4.4.2 Clay layers

The drained strength of the cohesive layers is estimated using the standard correlations provided by Ladd er al (1977) and the correlations given in BS 8002 (2015). On this basis the recommended values for the clay layers are:

$\phi'_{peak} = 29°$

$\phi'_{crit} = 27°$

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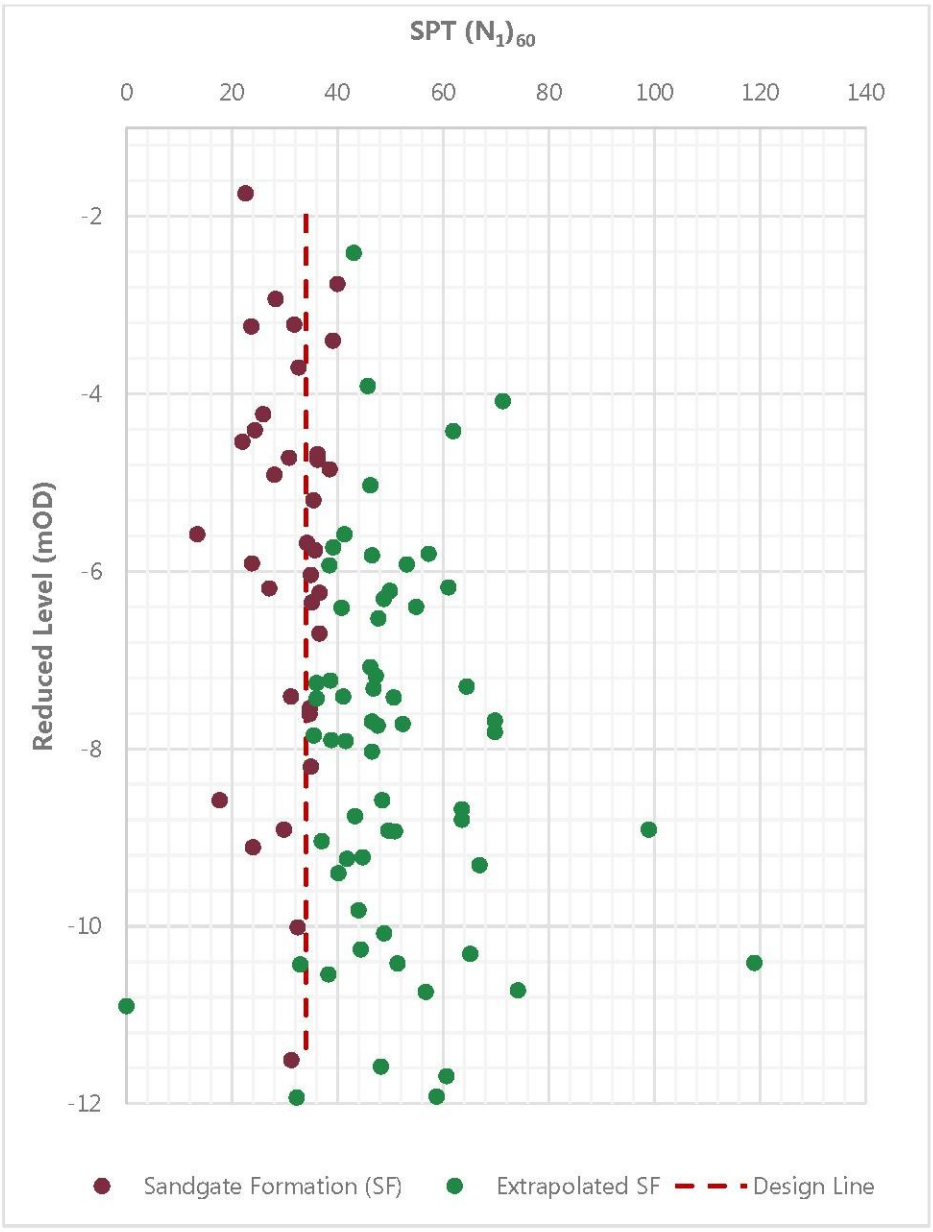


Figure 14 Sandgate Bed Formation SPT $(N_1)_{60}$ Value with depth

4.4.4.3 Uniaxial Compressive Strength and Rock Quality Density

Seven uniaxial compressive tests carried out on samples of the Sandgate Formation. The UCS values ranged from 0.1 to 0.4 MPA. Considering the effective stress at the depths the samples were taken then these results appear to be anomalously low.

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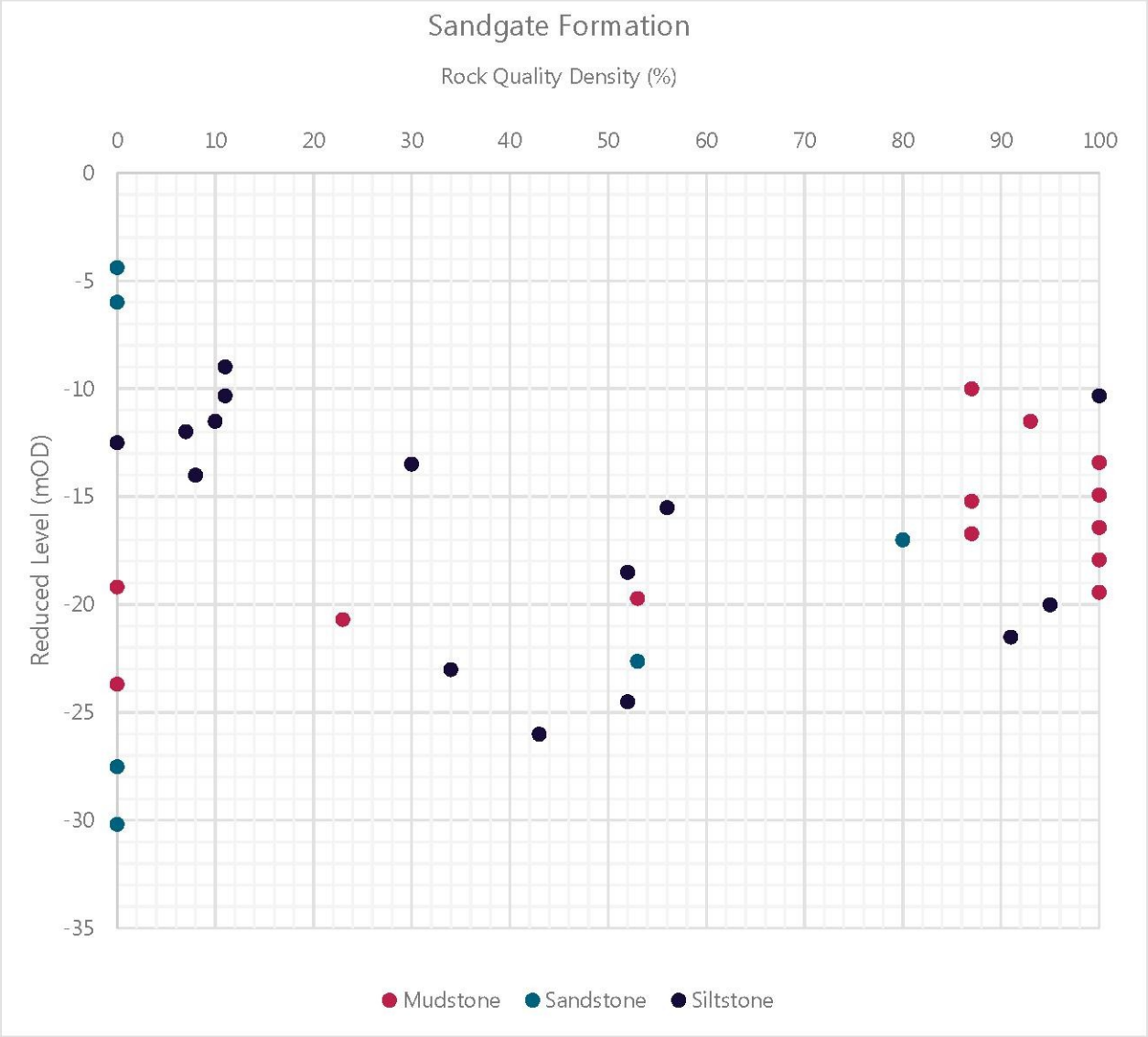


Figure 15 Rock Quality Density (%) of Sandgate Formation

Within the Sandgate beds there are intermittent clusters of data from 0-10%, 30 and 50% RQD and between 85 and 100% RQD. Mudstone is the predominant material with the higher RQD percentages. Refer to Figure 15.

4.4.5 Stiffness

4.4.5.1 Clay Material, Derived from Undrained Strength

Typical correlations between E and C_u have been derived from back analysis of case histories for Clay. For heave and settlement analysis, the following vertical stiffness values are recommended for assessing ground movements at formation level:

- Undrained Young's modulus: $E_{uv} = 450C_u$, $E_u = 78 – 128$ MPa, with Poisson's ratio $\nu = 0.5$;
- Drained Young's modulus: $E_v' = 0.75E_u$, $E_v' = 60 – 96$ MPa, with Poisson's ratio $\nu = 0.2$.

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4.4.5.2 Granular Material, Derived from SPT-N

E_v' = 1N is appropriate for a moderately conservative design approach. For a moderately conservative mean SPT of N = 27+1.7z, from +0.3mOD (see Figure 16) this gives E_v' = 27 – 60 MPa. The drained lateral stiffness, E_h', is assumed to be equal to E_v'.

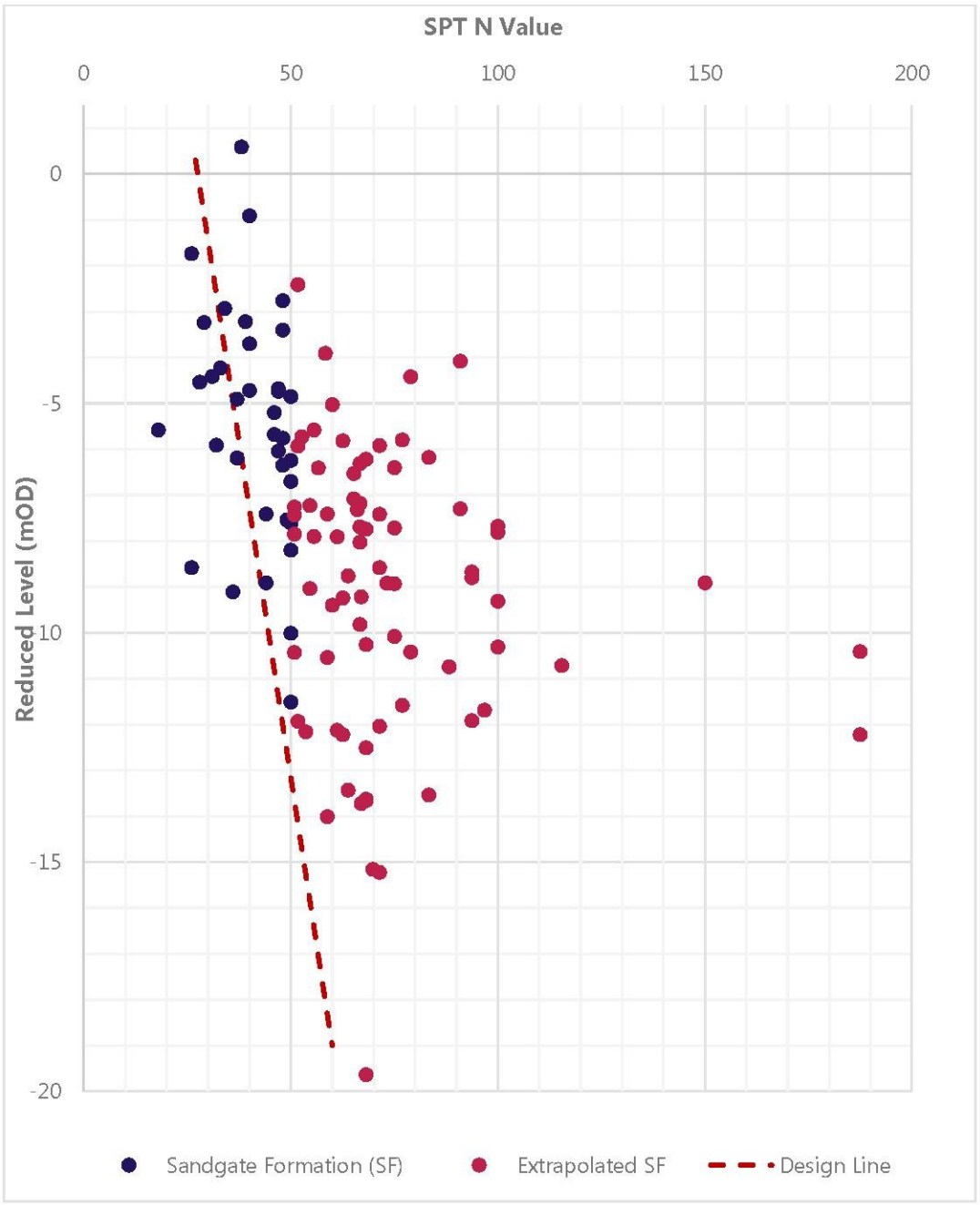


Figure 16 SPT 'N' value with depth (stiffness)

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4.4.6 Plasticity Index

A total of 19 samples were tested for plasticity index. The sandy slightly gravelly clay material within the Sandgate Formation has been assessed as a low to intermediate plasticity range, see Figure 17 below.

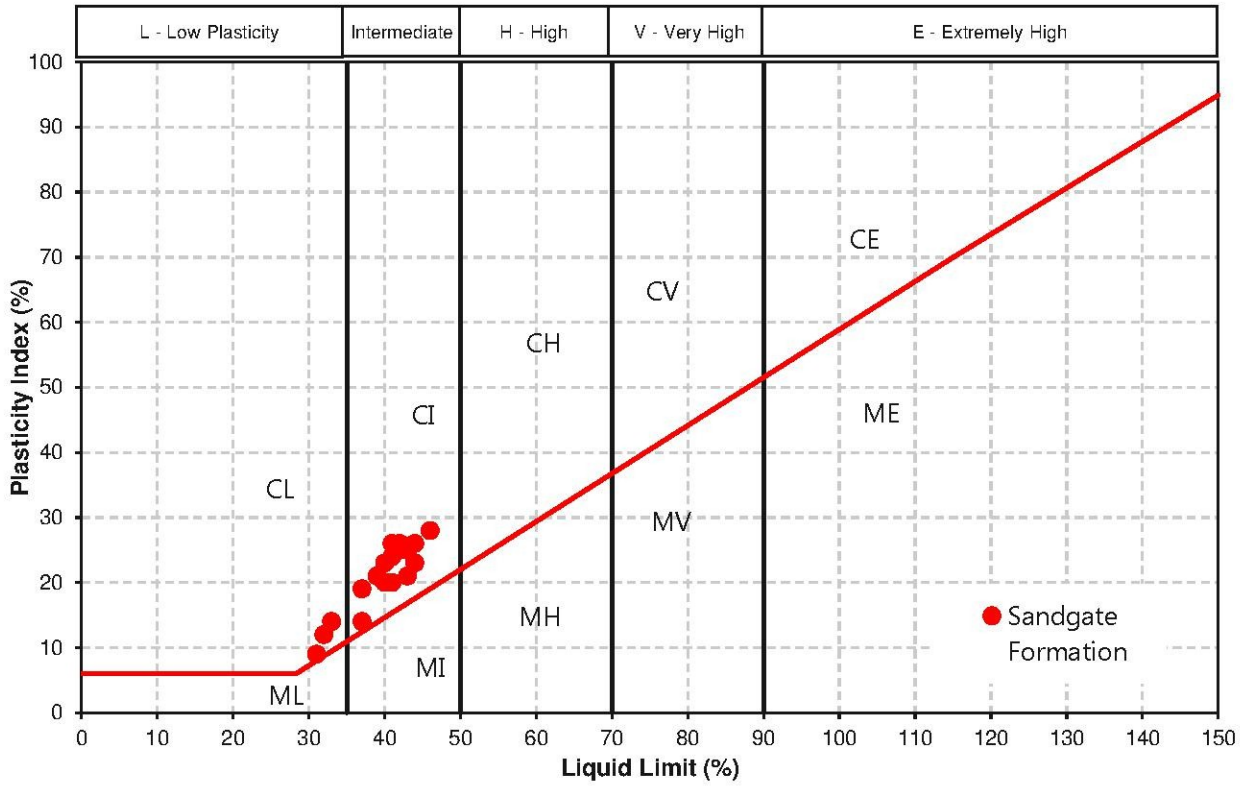


Figure 17 Plasticity Index with Liquid Limit for the Sandgate Formation

4.5 Hythe Bed Formation

4.5.1 Uniaxial Compressive Strength

The Hythe bed Formation comprises an interbedded sequence of siltstone, sandstone and limestone. The limestones and sandstones are generally weak to medium strong, however the siltstones are considerably weaker, described as very weak to weak. The majority of the sequence was made up from sandstones and limestones but there are layers of siltstone up to 6m thick. The uniaxial compressive tests carried out are presented below for the Hythe Formation.

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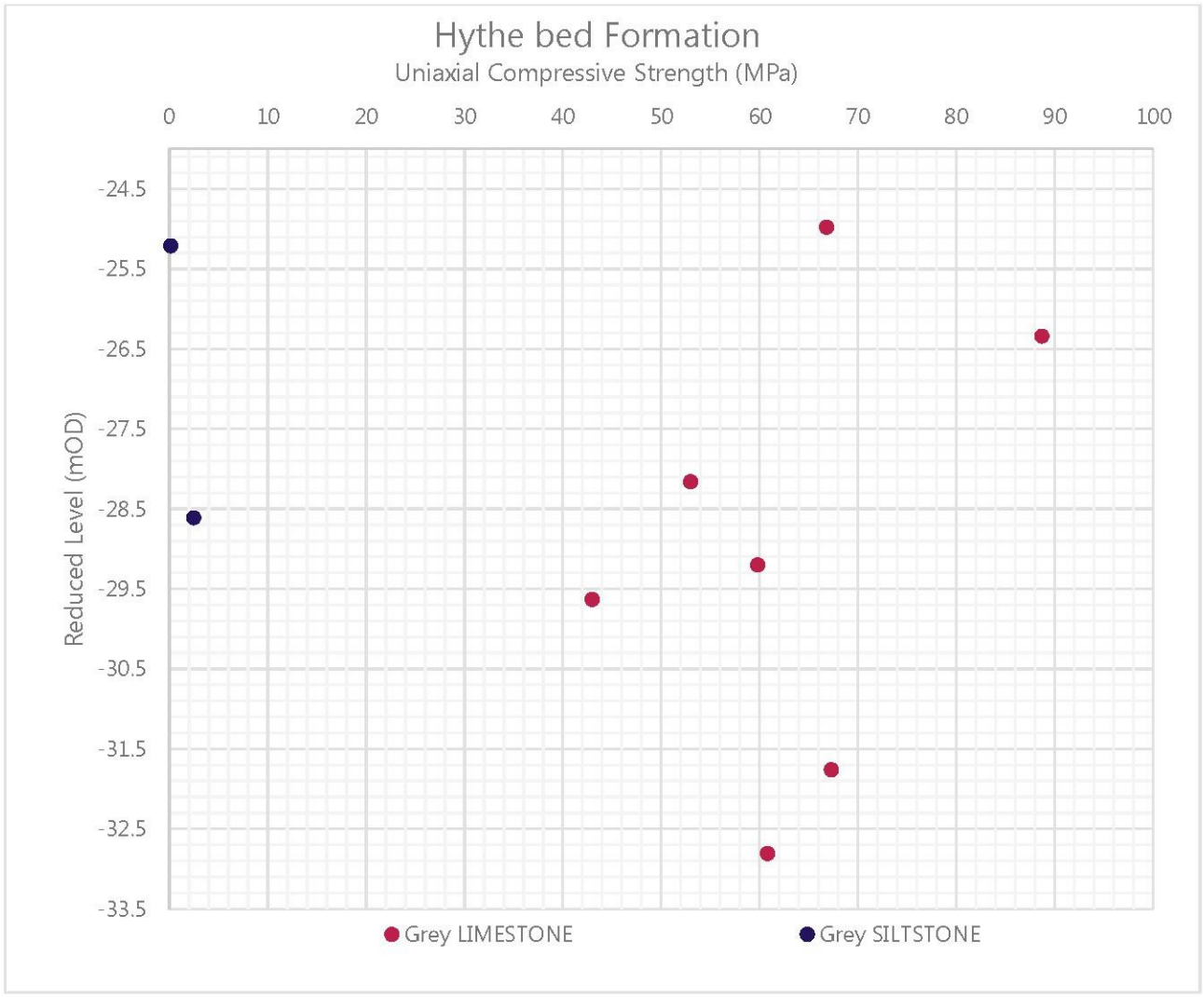


Figure 18 Limestone and Siltstone within Hythe Bed Uniaxial Compressive Strength (MPa)

4.5.2 Rock Quality Density and Fracture Index

Both the Rock Quality Density (RQD, %) and the fracture index have been considered for the Hythe Formation. The results are based on field description carried out during the Site Investigation.

It is clear there is much scatter in the data. The plot indicates the mudstones, sandstones and siltstones contain higher percentages of intact material with an RQD between 35 and 90% for these materials. For limestone, there is a concentration of RQD between 0 and 30% indicating a highly fractured zone; however, the values range up to 80% RQD. Refer to Figure 19.

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The fracture spacing recorded indicates a much of the material as 'non-intact', some of these fractures could have been drilling induced. This is seen across the mudstone layers, siltstone layers and limestone layers. Where fracture spacing could be recorded the estimated range of spacing between the fractures was recorded as frequently as 10mm spacing in the siltstone layers, 60mm spacing within the limestone and sandstone layers and 120mm within the mudstone layers.

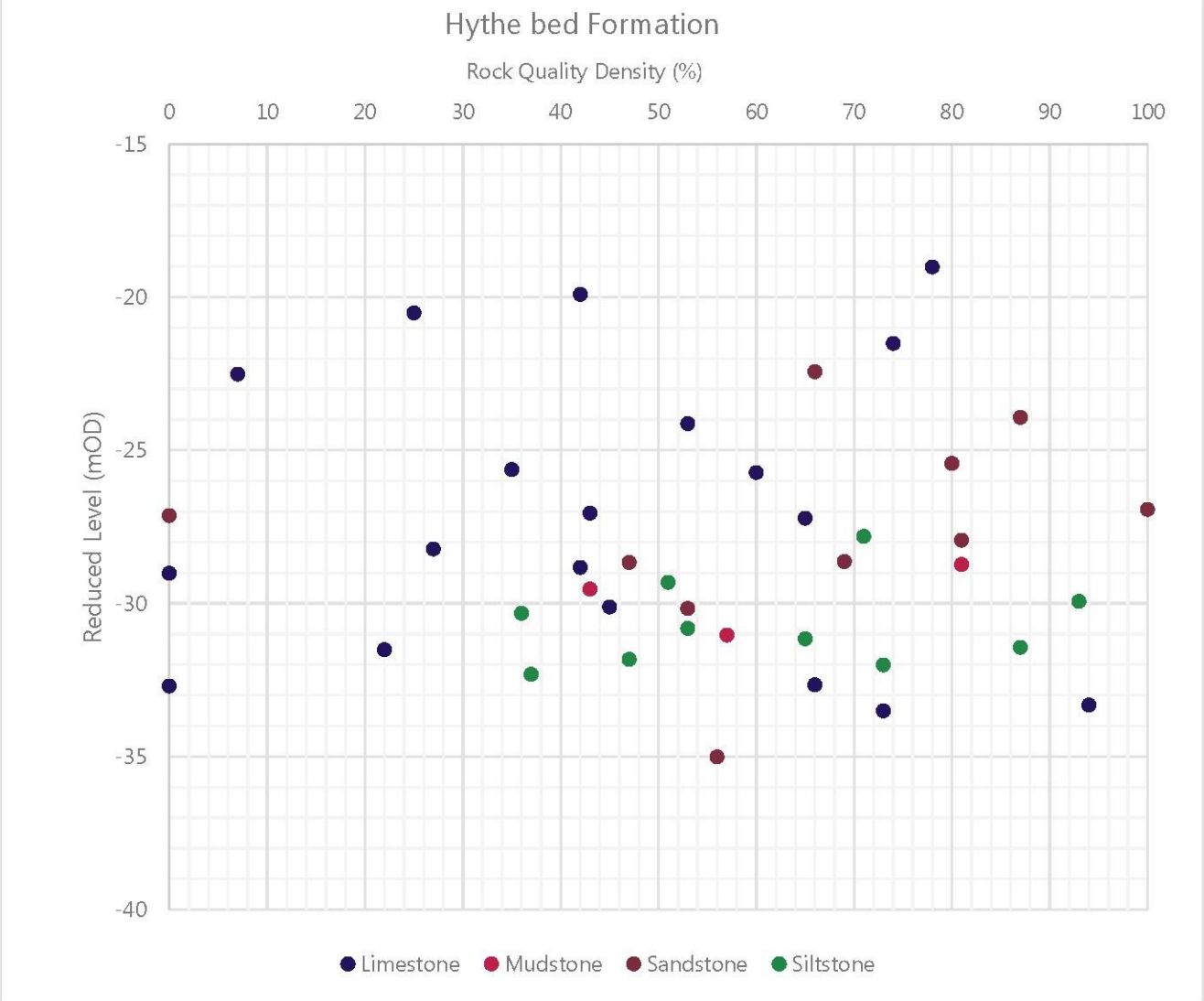


Figure 19 Rock Quality Density (%) of Hythe Beds

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5 Engineering Assessment

5.1 Shallow Foundations

Feasible foundation options may comprise pad/strip footings, raft foundations or piled foundations. The selection of these options is influenced by a number of factors relating to the ground conditions and proposed development, for example the sensitivity of the structure to ground movements. The structural design criteria will be derived as the design process progresses.

Depending on the structural loads and column grid a pad/strip foundation scheme maybe feasible. Beach Deposits/Sandgate Formation will provide a suitable founding strata for a shallow foundation option (pads/strip/raft) subject to specific design calculations being undertaken. The uniform nature of the materials may lead to creep settlement and the foundation design should be assessed in light of these movements.

It will be necessary to have a competent geotechnical engineer onsite while excavating at each foundation location to confirm suitability of the formation as a bearing surface.

5.1.1.1 Preliminary Pads/Strips Bearing Capacity

The bearing capacity is used as the ultimate limit state check against overall failure. A safety factor of three is commonly adopted to ensure safe design. The bearing capacities of various pad sizes are summarised below in Table 5-1 founding at 1.5m bgl. The bearing capacities are based on the pad footing being founded within the Upper Beach Gravels

Table 5-1 Bearing capacity founding at 1.5m bgl

Pad Size (m x m)	Founding Depth (m bgl)	Safe Bearing Capacity (based on FoS = 3) for Upper Beach Gravels (phi' 33°) (kPa)
1 x 1	1.5	235
2 x 2	1.5	258
3 x 3	1.5	281
4 x 4	1.5	308

5.2 Foundation Piles

The most appropriate piling method for this project is likely to be either traditional bored piles or continuous flight auger (CFA) piles. The investigation has shown that the Sandgate Beds comprise a sequence of interbedded sands and clays which eventually grade in to mudstones and siltstones. Due to the interbedded nature of this formation the carrying capacity of piles in this layer is dictated by the strength of the weakest material. In addition, this layer also limits the capacity of piles that are founded in the shallower more dense Upper and Lower Beech Gravels.

CFA Piles

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CFA piles are often found to be the more economical option (generally in instances where traditional bored piles require significant casing) and are generally suitable in most strata (gravels, sands, silts, clays and soft rocks). However, CFA piles have limitations on pile diameter, depth of boring and length of reinforcement; selection of an appropriate rig can be crucial depending on the underlying strata. Specifically, the following potential issues associated with CFA piling should be considered:

- geology – very stiff strata are likely to affect the rate of boring and pile construction, as well as increasing the potential for flighting or remoulding of the pile shaft interface;
- obstructions – CFA piles cannot penetrate through hard obstructions, natural or man-made. Existing foundations may be present onsite from former buildings;
- construction depth – CFA piles are limited to a maximum length of approximately 27m.
- Pile installation level – piles may be constructed from a level higher than formation level. The use of the CFA technique will require concrete break down to pile cut-off level after excavation and introduce problems related to reinforcement cage installation.

Traditional Bored Piles

Bored piles are suitable for all ground types, including rock, and are able to accommodate large loads with pile diameters up to 3m and lengths up to 70m. Bored piles are drilled with an auger while a temporary steel sleeve is inserted to maintain the bore through layers of unstable or water bearing ground. At this Site, it is likely that temporary casing will be required in the Beach Deposits due to the presence of groundwater and the nature of the ground.

5.2.1 Preliminary Pile Design

Pile capacities have been assessed for CFA and Rotary Bored piles.

The pile capacities have been calculated using a generic ground model based across the entire site. For detailed design this model should be reviewed and revised for the specific locations in which structures are to be based.

The total axial capacity of the piles has been calculated as the sum of both the shaft resistance and end bearing capacity, which will be generated within the Beach Gravels and the Sandgate Formation. Preliminary indicative pile load capacities have been calculated following Eurocode 7 (BS EN 1997-1 : 2004) design approach 1, combination 2:

- model factor of 1.4 adopted assuming no preliminary pile testing. It is assumed that working pile tests will be undertaken on at least 1% of piles;
- the concrete grade used for pile construction has been assumed to be C35 or greater (C35, i.e. characteristic cubic strength of 35N/mm^2). The evaluated preliminary allowable capacities for various pile diameters are presented in **Error! Reference source not found.** and allowable pile tensions in Figure 20.

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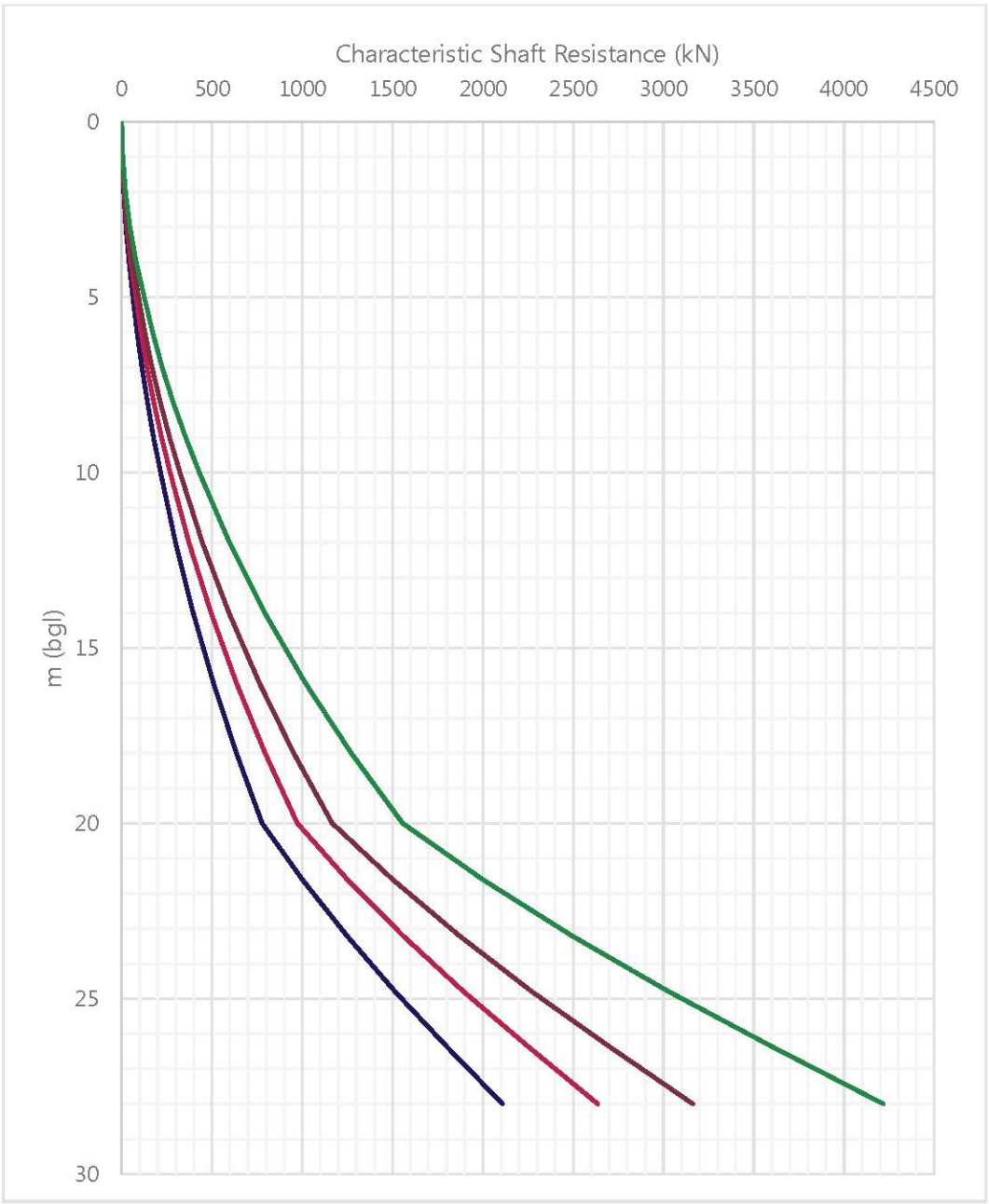


Figure 20 Allowable pile tension (kN)

5.3 Basement Structure Design

At present, a single level basement has been proposed for a car park. The form of construction will depend upon several factors including excavation induced ground movements, basement waterproofing requirements, structural loading and spatial restrictions. Possible options for the basement retaining wall include:

- temporary and permanent sheet-pile wall;
- temporary and permanent secant pile wall.

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The design of a basement structure must account for anticipated groundwater conditions and pressures below the basement slab. The current assumptions with regard to groundwater levels are presented in Section 3.6. Basements should also be designed to an appropriate minimum waterproof grade as detailed in BS8102. The contractor will be responsible for the temporary dewatering required during excavation.

The stability of the retaining structures will be subject to temporary and permanent works design requirements. The design of the temporary and permanent works will depend on both the geometry and sequencing of the works and must be designed to ensure wall deflections and associated ground movements do not adversely affect neighbouring structures and infrastructure. Careful consideration of temporary and permanent works and construction staging is essential.

The construction monitoring strategy will be developed in order to satisfy stakeholders and surrounding structures and infrastructure. Additionally, the performance of the excavation will need to be monitored from a stability and serviceability perspective.

When using the design parameters from Section 4.1 for Beach Gravels and Sandgate Formation, the following should be noted:

- Beach Deposits are moderate to well graded and the formation is subjected to tidal fluctuations, therefore, a conservative approach is adopted in deriving these parameters;
- the Sandgate Formation (formed of sands and clays) was observed in firm to stiff state with low to intermediate plastic nature.

The parameters are derived on the basis of field observation and laboratory testing. Strength properties in terms of undrained shear strength are discussed in detail in Section 4.4.3. Earthworks, Filling and Basement Excavations

Large scale earthworks are to be undertaken across the Site as part of beach nourishment exercise and filling will be undertaken with harbour deposits. Material is being dredged/excavated from the outer harbour and being transported across to another area on the site to be used as construction fill. The material was tested for stabilisation purposes by PowerBetter Development Ltd, the results are summarised below and can be found in more detail within [6].

5.3.1 Soil Modification

PowerBetter Development Ltd were commissioned by BuroHappold on behalf of Folkestone Harbour (CP) Ltd to carry out soil modification testing on the harbour material. The aim of the testing carried out by PowerBetter Development Ltd on the 13th October 2016 was to provide an indication of the suitability of the harbour materials as a construction material. The process comprised the following; moisture content (MC) and optimum moisture content (OMC) of the natural material, Californian Bearing Ratio (CBR) and uniaxial compressive strengths (UCS). The MC and OMC results lead to the decision to use 2%, 4%, 6%, 8% and 10% hydraulic binder.

At each point of the OMC, a CBR was made and tested. This indicated what strength could be achieved at OMC, and if the material could be used in its natural state. The results indicated that the material would be adequate for use as a bulk fill material prior to placement of any additional aggregates (OMC – 11.1% and CBR of 28%). An important factor to consider is that the material will be susceptible to water ingress. As a consequence of this, the CBR of the natural material was placed in a soak tank; it lost cohesion within 12 hours.

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Based on the visual description of the soils, the binder PowerBetter Development Ltd chose as being most suitable was cement as the sampled soils acted with little cohesion. A total of 4% cement was added to the natural material which resulted in a higher OMC (12.5%) and higher CBR value (45.1%)

Additional testing was carried out using only lime. The reason behind choosing to do lime testing was because the cement only options were not reducing the moisture content (MC) sufficiently after the mix, preventing the material from stiffening sufficiently. This was particularly apparent when the CBR and UCS test specimens were compacted into their moulds, which appeared to bring more moisture out of the sample. The result of this within the UCS test specimen meant a longer curing and solidifying stage (several days) as the specimen would otherwise deform on extraction. The use of high lime powder did alleviate this issue and enabled UCS specimens to be extracted before curing.

All the soil samples, when tested to achieve CBR results, indicated re-use as a construction material was feasible. When tested for UCS the material did not adhere the same results. This could be due to moisture content levels 60-80% above the optimum moisture content level. In order to achieve a usable construction material, the results indicated on site Mix-in-Place stabilisation methods. This is likely to require high percentages of hydraulic powder to reduce the moisture content adequately and make the material immediately workable.

If Mix-in-Place stabilisation is not viable, other options are available. This could include a dredging/de-watering by centrifugal pump solution, dredging/stockpiling the material on a drainage blanket and letting the material drain over a set period of time before being used.

For the results summarised in PowerBetter Development Ltd report to be replicated on site, it is important that the subgrade underlying soils should provide an adequate, consistently maintained and stable supporting formations to provide appropriate support/bearing requirements.

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6 Conclusions and Recommendations

The proposed development has not been finalised therefore a variety of foundation options have been discussed which could vary if more localised, development specific, design ground models are used.

The relevant geotechnical parameters for the Beach Gravels, Sandgate Formation and Hythe Bed geological stratum below the Site have been calculated and are summarised in Section 4.

The beach gravels were divided into upper and lower bounds as the material was slightly different with depth. The Upper Beach Gravels have been defined as uniform to moderately graded fine sands and medium-course sub rounded to rounded gravels. The Lower Beach Gravels were assessed as a well graded material with a greater fine sand concentration than the Upper Beach Gravels. The Sandgate Formation was also divided up in the assessment as it contained both granular and cohesive material. Where the material became more clayey with depth undrained shear strengths have been provided. In respect of pile design, it is necessary to consider the worst case scenario in design, so the end bearing of piles should be based on the strength of the clay, whilst the skin friction based on the strength of the granular soils. The Hythe bed formation comes in at approximately 25-28m bgl and comprises Limestone with occasional Sandstone layers.

Foundation options have been discussed in detail in Section 5. The assessed options comprised; shallow pad or strip foundations within the Beach Gravels. The assessment considered a factor of safety of 3. The maximum bearing capacity, was 308kPa on a 4x4m pad.

It is anticipated that piles will be founded within the beach gravels or the Sandgate Formation. The assessment considers piles down into the base of the Sandgate Formation. It should be noted that the increase in capacity within the beach gravels decreases due to the influence of the clay layers in the Sandgate Formation.

The concrete design class assessment based on the properties of the Made ground beneath the Site, the design sulphate class for buried concrete should be DS-2, and the ACEC (Aggressive Chemical Environment for Concrete) class should be AC-2.

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7 Bibliography

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Appendix A Site Plan

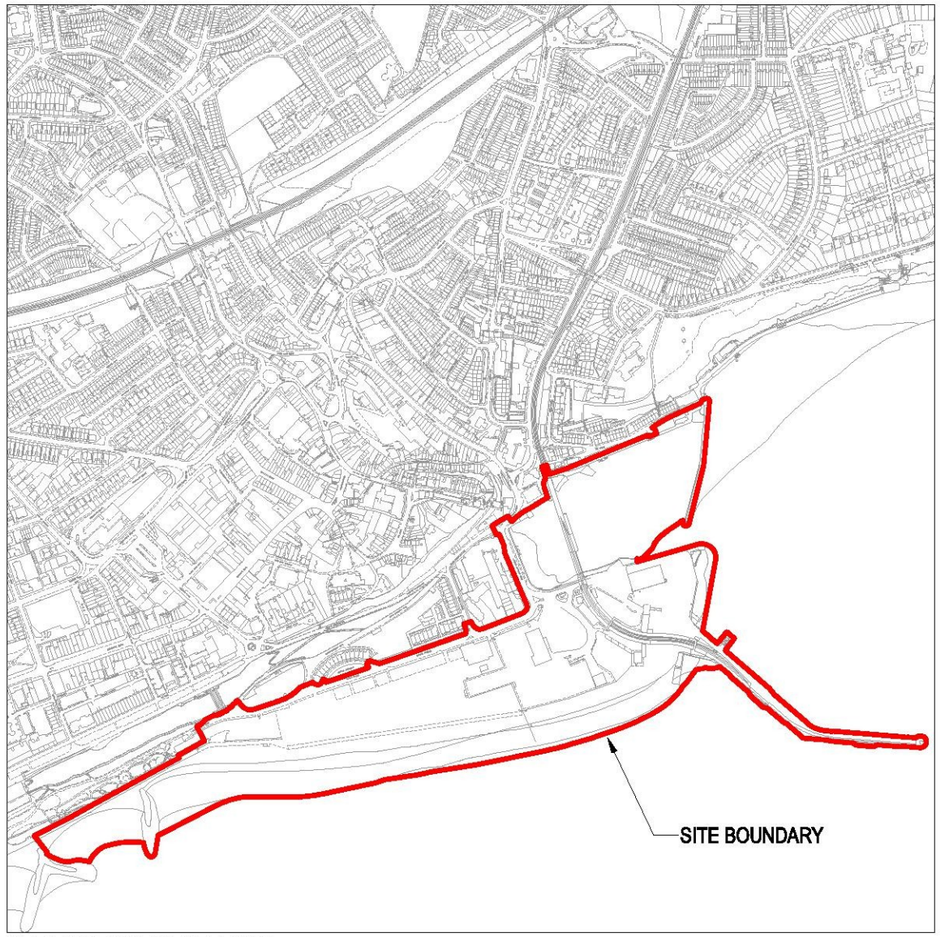
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SITE LOCATION SCALE 1:50000

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SITE LOCATION SCALE 1:10000

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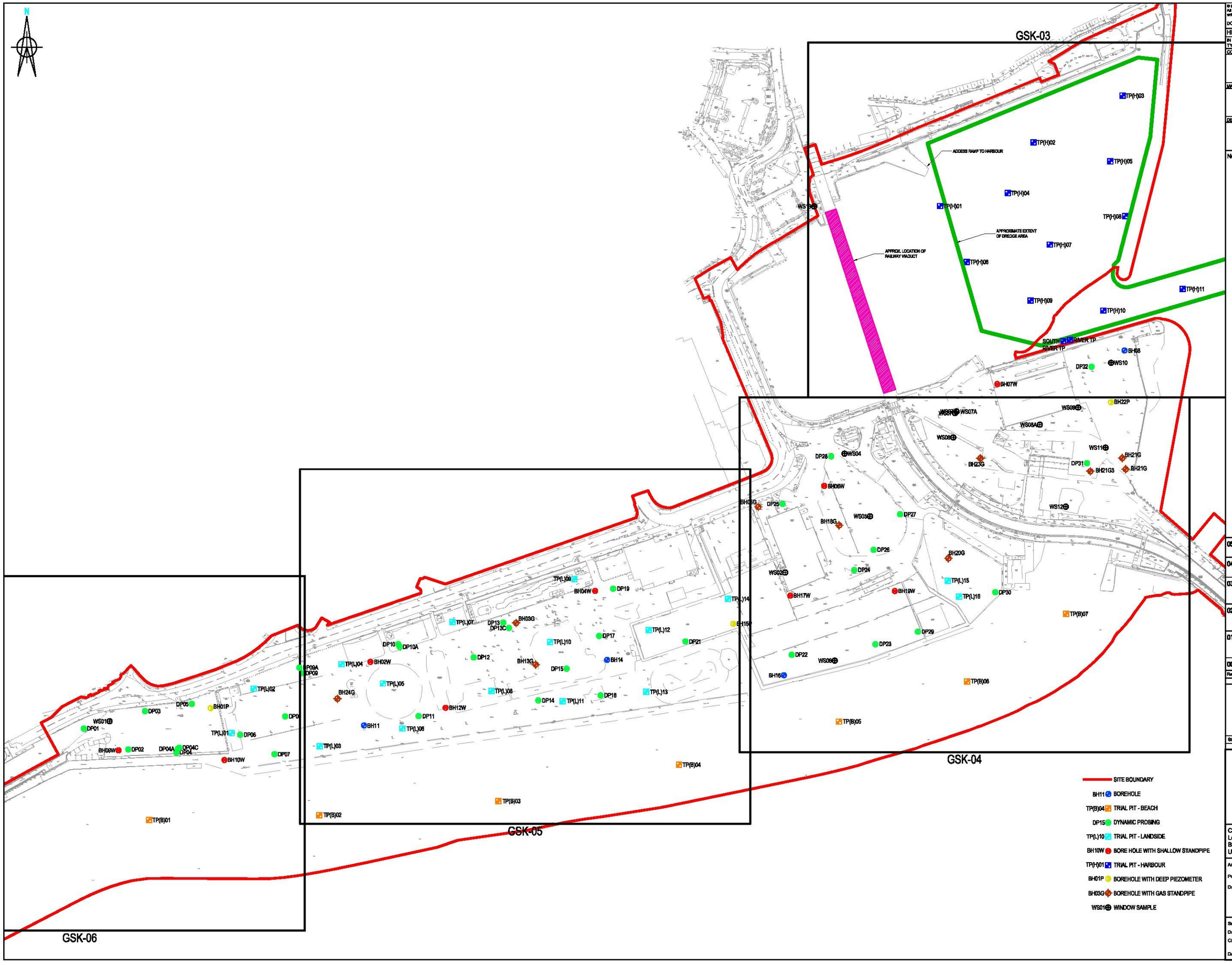
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Job No. 034029
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03	WINDOW SAMPLES ADDED AMENDMENTS TO GI LAYOUT ISSUED FOR TENDER	14.03.16	NG	FG
02	HARBOUR TRIAL PITS REINSERTED ISSUED FOR TENDER	11.12.15	JW	RF
01	HARBOUR TRIAL PITS REMOVED SHEET LAYOUT UPDATED ISSUED FOR TENDER	30.06.15	NG	RF
00	TENDER	15/04/15	JW	NP

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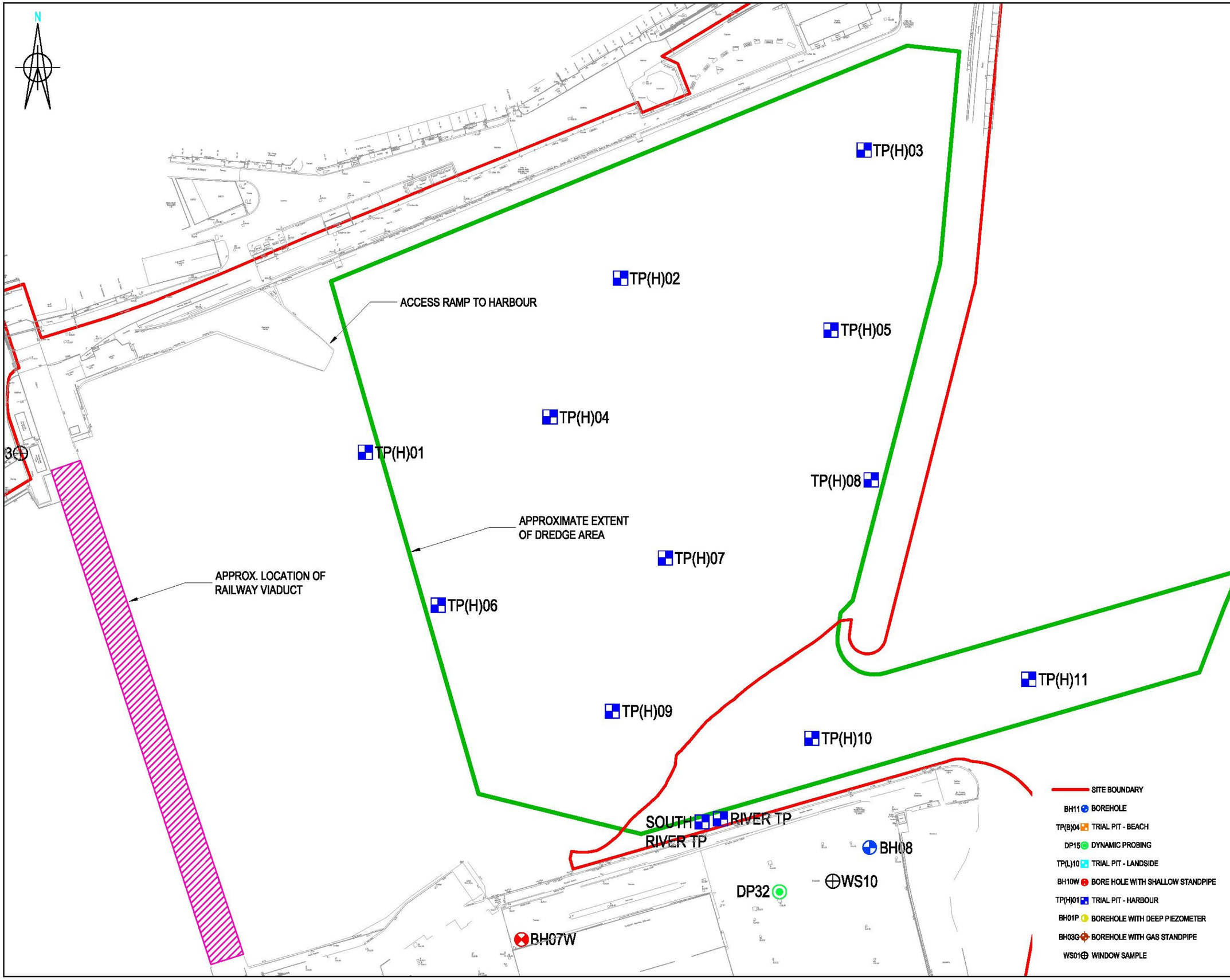
Architect JLL Ltd

Project FOLKESTONE SEAFRONT DEVELOPMENT

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Drawn by JW
Checked by NP
Date 15/04/15

Job No. 034029
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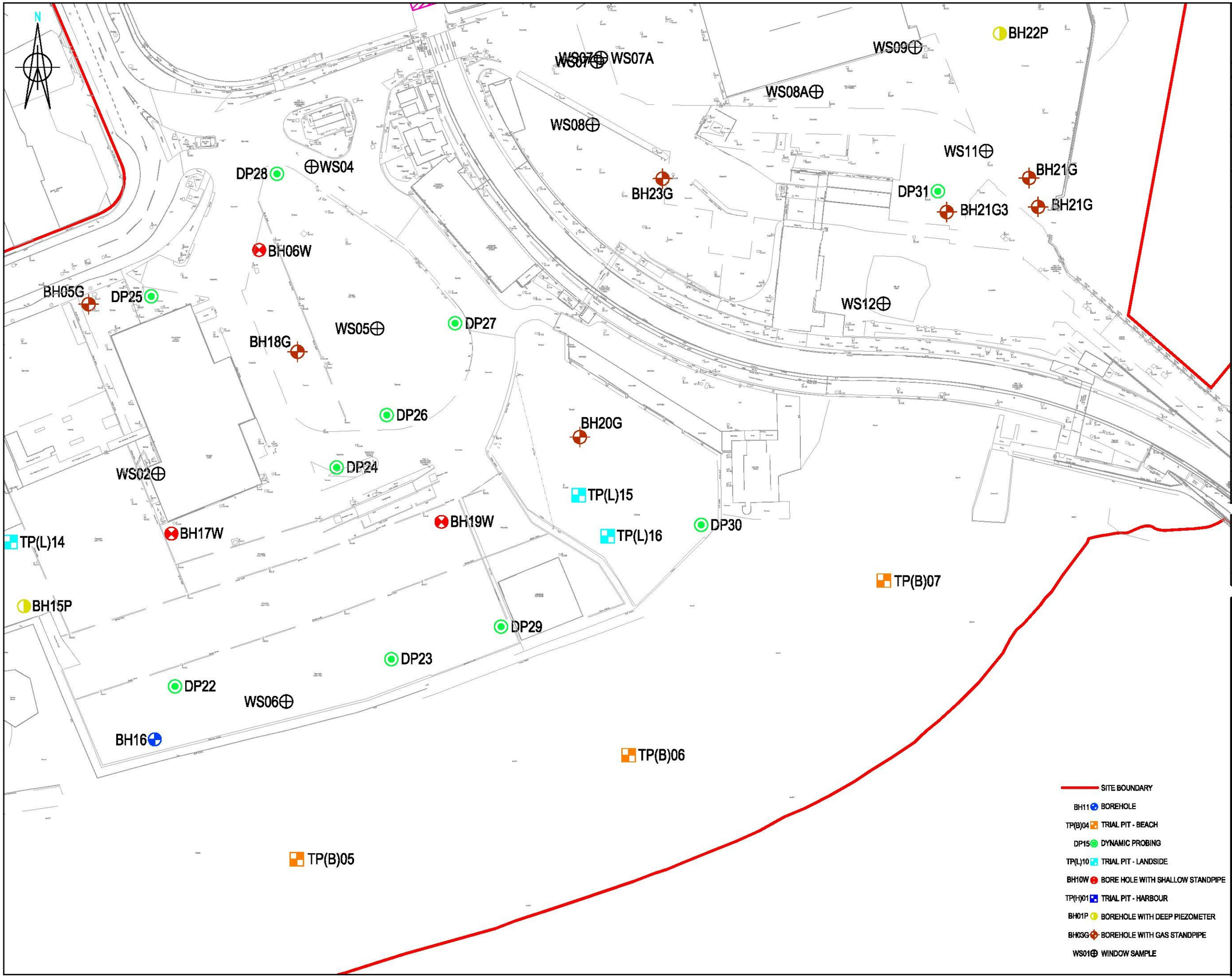
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Drawn by JW
Checked by RF
Date DECEMBER 2015

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Architect *Farrels*

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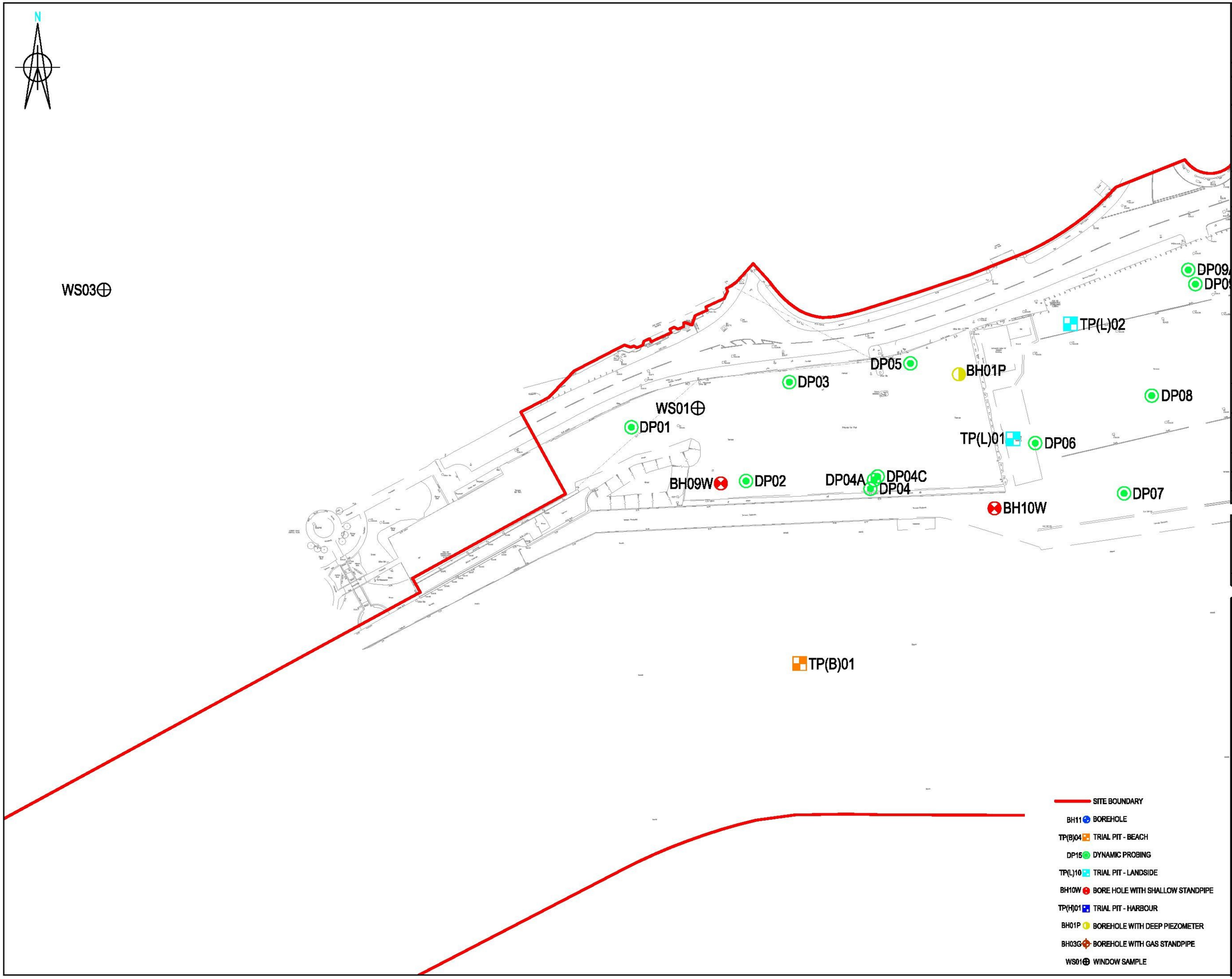
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Architect: *JLL Ltd*

Project: **FOLKESTONE SEAFRONT DEVELOPMENT**

Drg Title: ORGANIC CONTAMINANT SAMPLES EXCEEDING SCREENING CRITERIA

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Job No.	034029
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GSK-02

TP(H)03
TP(H)02
ACCESS RAMP TO HARBOUR
TP(H)05
TP(H)04
WS13
TP(H)01
TP(H)08
APPROXIMATE EXTENT OF DREDGE AREA
TP(H)07
APPROX. LOCATION OF RAILWAY VIADUCT
TP(H)06
TP(H)11
IN-1
IN-2
TP(H)09
TP(H)10
SOUTH RIVER TP
IN-3
IN-4
BH08
WS10
DP32
BH07W
BH22P
WS07
WS07A
WS08
WS08A
WS11
BH21G2
DP28
WS04
BH23G
DP31
BH21G3
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DP25
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TP(B)02

SITE BOUNDARY
BH11 BOREHOLE
TP(B)04 TRIAL PIT - BEACH
DP15 DYNAMIC PROBING
TP(L)10 TRIAL PIT - LANDSIDE
BH10W BORE HOLE WITH SHALLOW STANDPIPE
TP(H)01 TRIAL PIT - HARBOUR
BH01P BOREHOLE WITH DEEP PIEZOMETER
BH03G BOREHOLE WITH GAS STANDPIPE
WS01 WINDOW SAMPLE

PAH SAMPLE EXCEEDANCE - COMMERCIAL/INDUSTRIAL CRITERIA
PAH SAMPLE EXCEEDANCE - RESIDENTIAL WITHOUT PLANT UPTAKE CRITERIA
PAH SAMPLE EXCEEDANCE - RESIDENTIAL WITH PLANT UPTAKE CRITERIA

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GSK-02

ACCESS RAMP TO HARBOUR
APPROXIMATE EXTENT OF DREDGE AREA
APPROX. LOCATION OF RAILWAY VIADUCT
SOUTH PIER TP
RIVER TP

SITE BOUNDARY
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TP(B)04 TRIAL PIT - BEACH
DP15 DYNAMIC PROBING
TP(L)10 TRIAL PIT - LANDSIDE
BH10W BORE HOLE WITH SHALLOW STANDPIPE
TP(H)01 TRIAL PIT - HARBOUR
BH01P BOREHOLE WITH DEEP PIEZOMETER
BH03G BOREHOLE WITH GAS STANDPIPE
WS01 WINDOW SAMPLE

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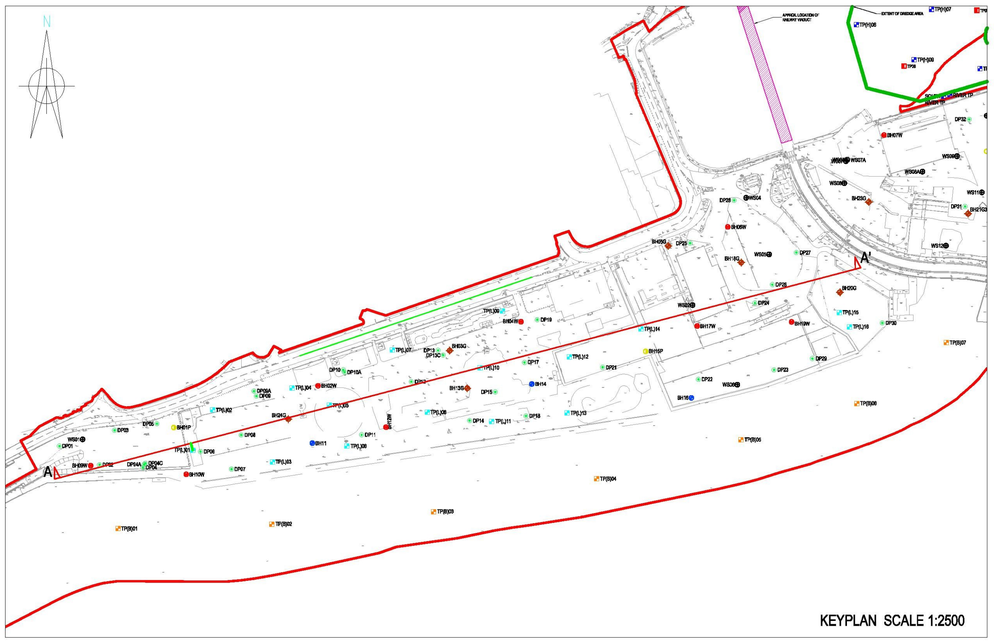
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Architect	JLL Ltd
Project	FOLKESTONE SEAFRONT DEVELOPMENT
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Drawn by	NG
Checked by	FG
Date	SEPT 2016
Job No.	034029
Drawing No.	GSK-08
Rev	00

Appendix B Geological Cross Section across Site

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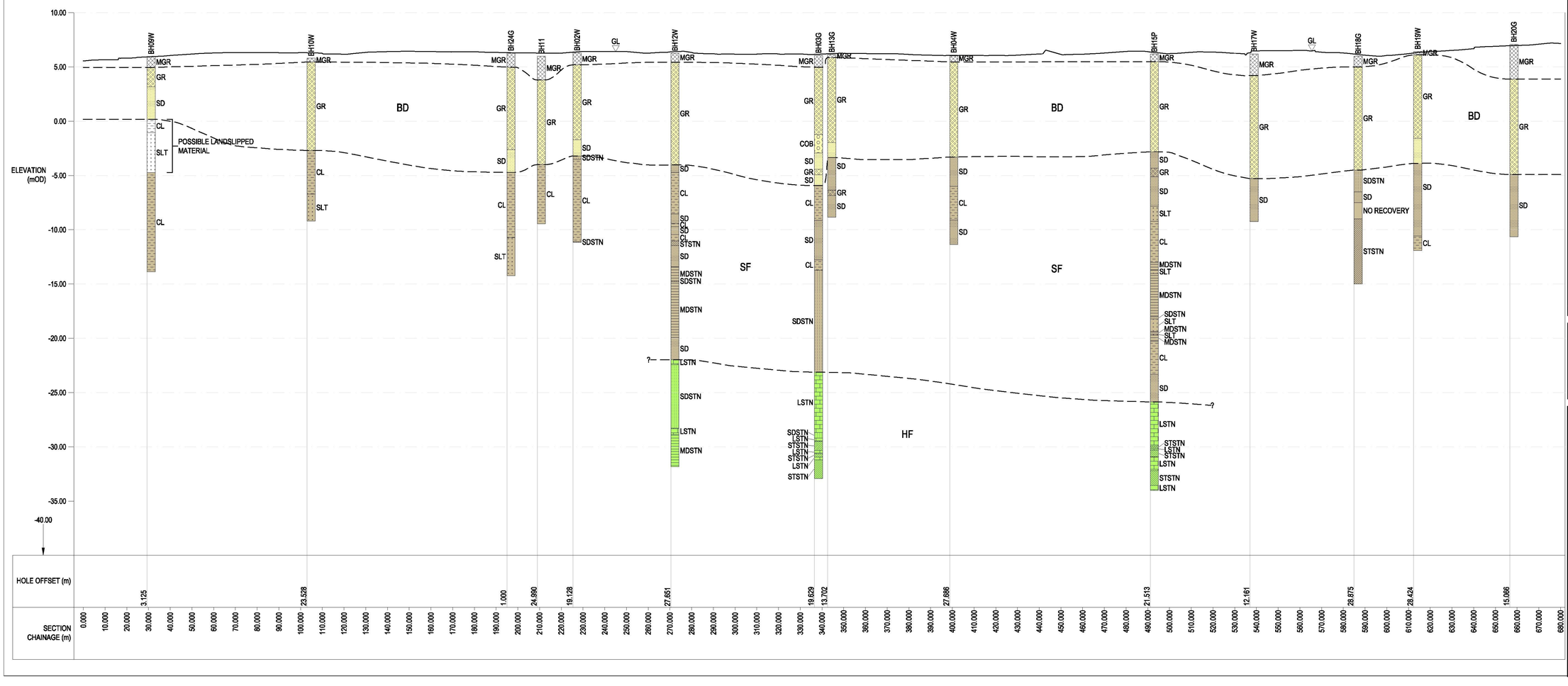
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VERT. 1:200
HORIZ. 1:1000

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CLAY	CL	
SILT	SLT	
SAND	SD	
GRAVEL	GR	
COBBLES	COB	
SILTSTONE	STSTN	
SANDSTONE	SDSTN	
MUDSTONE	MDSTN	
LIMESTONE	LSTN	
GEOLOGICAL STRATA	**REFERENCE**	**HATCH**
BEACH DEPOSITS	BD	
SANDGATE FORMATION	SF	
HYTHE FORMATION	HF	

A

A'



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Architect ACME
Project FOLKESTONE SEAFRONT DEVELOPMENT
Drg Title GEOLOGICAL CROSS SECTION A - A'

Scales@A1 AS SHOWN
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Date NOV 2016

Job No. 034029
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