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GEOTECHNICAL INVESTIGATION REPORT
BAYSWATER MARITIME PRECINCT
21 SIR PETER BLAKE PARADE
BAYSWATER

Prepared For:

Bayswater Marina Holdings Limited
21 Sir Peter Blake Parade
Bayswater
Auckland 0662

REPORT ISSUE AUTHORISATION

Geotechnical Investigation Report

Bayswater Maritime Precinct

21 Sir Peter Blake Parade

Bayswater

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1. INTRODUCTION

At the instruction of Bayswater Marina Holdings Limited (BMHL), KGA Geotechnical Group Limited (KGA) have carried out a geotechnical engineering investigation at 21 Sir Peter Blake Parade, Bayswater, in relation to the proposed Bayswater Maritime Precinct development.

Specifically, the scope of our investigation was to carry out a visual inspection of the site, and to investigate the subsurface conditions using a combination of:

- Mechanically excavated test pits;
- Rotary cored boreholes using machine-operated equipment;
- Cone Penetration Tests (CPTs), and
- Laboratory testing.

The information obtained from our investigation has been applied within this report to assess the ground conditions and groundwater regime, assess the perceived geotechnical constraints to the proposed development, and provide preliminary recommendations with regards to foundation design.

This report, presenting our findings and conclusions, has been prepared in support of a Resource Consent application.

This report is not sufficent support the detailed design, and is not intended to be lodged in support of a Building Consent application. It is expected that further geotechnical analysis and assessment will be required in support of the detailed design and Building Consent application; such input was beyond the scope of our brief for this investigation.

2. SITE DESCRIPTION

2.1 Legal & Location Description

The Bayswater Maritime Precinct site currently occupies the property of 21 Sir Peter Blake Parade, which is legally described as Lot 1 DP 309604, and has plan area of 3.3415ha. We understand that this property comprises reclaimed land.

The site is approximately rectangular in plan shape, with the long axis approximately orientated north to south. It is surrounded by the Waitemata Harbour to the north and southeast, berths for the Bayswater Marina to the west and south, a commercial property to the east, and the cul-de-sac end of the Sir Peter Blake Parade road reserve to the northeast, as indicated on our drawing entitled "Site Plan, Aerial Image Underlay", Sheet No. KGA 1, presented within Appendix 1.

2.2 Topographic Description & Existing Features

During our initial walkover visit, the site topography was noted to be near level to undulating with a very gentle westerly aspect. The highest part of the site, approximately just to the south of the centre of the site, was noted to be no more than 5m above mean sea level.

The ground cover of the approximate southern two-thirds of the site consists of paved car parking areas with intermediary grass berms and swales, along with a grass field. Trees are discretely located around the field and grass berms. The remaining northern third of the site comprises a commercial boat yard operation.

There are only a few buildings on the site at present, which include sheds for the boat yard operation, two separate toilet/shower blocks, offices for the Bayswater Marina at the extreme southern end of the site, and covered gate houses to the south and west at the entrances to the gangways to the marina berths.

From our reconnaissance walkover of the site, it was noted that the existing structures around the development area appeared to be sound, with no obvious indicators of distress that would suggest inadequate performance of their existing foundations or notable differential movement of the land beneath them.

The topography of the site, along with existing site features is shown on the site survey drawing by Hampson & Associated Limited (HAL), entitled “Bayswater Marina”, reference No. 6211, dated 20 December 2013. For reference, a copy of the HAL survey plan is presented within Appendix 2. Further, certain features of relevance have been transposed from the HAL survey plan on to our drawing entitled “Site Plan, Aerial Image Underlay”, Sheet No. KGA 1, presented within Appendix 1.

2.3 Auckland Council GIS Viewer

The Auckland Council GIS viewer, GEOMAPs, indicates that a 150mm diameter public sewer line with connections, is located just inside the eastern site boundary, whilst a 150mm PVC water supply pipe is located just beyond the eastern boundary.

The historic aerial photograph imagery available in the GEOMAPs system date from 1959, 1996, 2001, 2003, 2006, 2008, 2010, 2012, 2015 and 2017.

The 1959 image shows that the land within the site boundaries was not in existence at that time, but that neighbouring property at 23 Sir Peter Blake Parade was.

The 1996 image shows the land within the site boundaries being in existence, and that the breakwater for the marina was partially in place. The image colourations suggest that the land reclamation was under construction at the time the image was captured, with the northern one-third of the site being more established than the southern two-thirds.

The images from 2001 to 2017 reflect the existing site cover (discussed in Section 2.2 above) and document the growth of the trees planted on the site.

3. PROPOSED DEVELOPMENT

We are in receipt of a drawing prepared by Paul Brown Architects Limited (PBA), entitled “Bayswater Terraces, Bayswater, Auckland”, dated 22 January 2021, Drawing No. 839-SK200, GA Plans, which depict the general extent of the proposed development. For reference, a copy of this drawing is presented within Appendix 2.

In general, the PBA drawings show that it is proposed to construct eighteen buildings across the site in three distinct precincts, which are labelled as South, Central and North. Each precinct will be serviced by an extension of Sir Peter Blake Parade, along with four additional new roads, labelled as North Lane, Link Street, Cross Street and South Street. Car parking will be provided on part of North Lane, Cross Street and Link Street, and in the centre of all three precincts, while boat trailer parking will be provided on Sir Peter Blake Parade.

Three three of the proposed buildings will be mixed use, with commercial units on the ground floor, and residential apartments above. It is not specifically indicated on the PBA drawing, however we understand that the mixed use buildings will also include car parking basement levels.

All other buildings indicated on the PBA plans will ultimately comprise terraced houses. We understand that the final form of the terraced houses is not intended to be determined by our Client. Instead, a set of development rules will be provided to the future owners of the terraced units, and it will otherwise be up to the future owners to determine the final details of their individual units prior to the construction of each, which, unless several are bought by the same owner, will be undertaken independently of any other.

We understand that an esplanade boardwalk will be constructed around the perimeter of the development. The majority of the boardwalk will be constructed on land, however there will be timber viewing platform decks constructed at intervals which will extend beyond the land and be supported on piled foundations.

For reference, the general outline of the proposed buildings has been transposed on to our drawing entitled “Site Plan, Aerial Image Underlay”, Sheet No. KGA 1, presented within Appendix 1.

In addition to the PBA plan provided, we have also been provided with a drawing set prepared by Airey Consultants Limited (ACL), entitled “Bayswater Marina Holdings Limited, Bayswater Maritime Precinct, December 2020 – Resource Consent”, Job No. 12582-01. The drawing set contains multiple drawings depicting the proposed civil works associated with the site redevelopment, however for reference we have specifically attached the following drawings within Appendix 2:

- Drawing No. 200 – Proposed Contours Overview;
- Drawing No. 201 – Proposed Contours Sheet 1 of 4;
- Drawing No. 202 – Proposed Contours Sheet 2 of 4;
- Drawing No. 203 – Proposed Contours Sheet 3 of 4;
- Drawing No. 204 – Proposed Contours Sheet 4 of 4;
- Drawing No. 210 – Cut And Fill Overview;
- Drawing No. 211 – Cut And Fill 1 of 4;
- Drawing No. 212 – Cut And Fill 2 of 4;
- Drawing No. 213 – Cut And Fill 3 of 4;
- Drawing No. 214 – Cut And Fill 4 of 4;
- Drawing No. 220 – Earthworks Cross-Sections;
- Drawing No. 221 – Temporary Timber Pole Retaining Wall;
- Drawing No. 220 – Gabion Basket Retaining Wall;
- Drawing No. 240 – Proposed Retaining Wall Overview;
- Drawing No. 241 – Proposed Retaining Wall 1 of 4;
- Drawing No. 242 – Proposed Retaining Wall 2 of 4;
- Drawing No. 243 – Proposed Retaining Wall 3 of 4;
- Drawing No. 244 – Proposed Retaining Wall 4 of 4;
- Drawing No. 504 – Wastewater Plan 4 of 4, and;
- Drawing No. 520 – Wastewater Pump Station.

Notably, the ACL drawing set indicates that the three mixed-use buildings will comprise basement parking levels, with finished basement levels of RL1.3, RL0.6 and RL1.2m. The remainder of the terraced units are also indicated to have a finished basement levels ranging from RL3.4m to RL3.7m.

The ACL cut and fill plans and cross sections suggest that bulk excavation will largely be undertaken through the centre of the site, with the fill won from the cuts to be placed around the perimeter to raise the ground level above costal inundation levels. This is indicated on the ACL drawing No. 220, which shows that the finished basement level of the terraced units will otherwise be at, or just about at the existing, pre development ground level.

In general, the bulk cut depths are indicated on the ACL drawings to range from 0.0m to 1.2m depth, while filling depth will largely range from 0.0m up to 1.5m thick. Deeper cuts of up to 3.6m will be undertaken locally for the three mixed-use buildings basements.

The filling around the perimeter of the development is shown to be supported through the use of an engineer designed, gabion basket retaining wall. The ACL drawing No. 220 suggests that the gabion basket wall will be up to 2.5m high max, and will be founded at the mean high water level, on top of the existing revetment materials.

Internal to the development, in order to facilitate the future terraced unit basement levels, temporary timber pole retaining walls are shown to support the fill. The ACL drawing No. 221 provides details and geotechnical design parameters assumed in the design of the temporary timber pole walls.

The proposed services plans indicate that reticulated stormwater, wastewater and water supply services will be installed as part of the development. Stormwater from hard-standing areas will ultimately be directed towards the sea, whereas wastewater will be directed to a pump station that will connect into the existing infrastructure further uphill within Sir Peter Blake Parade to the north. Notably a pump station is shown to be located within the boundaries of the neighbouring property to the east.

4. BACKGROUND INFORMATION

4.1 Property File Examination

An examination has been made of the Property Files held by Auckland Council for:

- 13 Sir Peter Blake Parade;
- 21 Sir Peter Blake Parade, and;
- 23-27 Sir Peter Blake Parade.

No specific geotechnical investigation or completion reports pertaining to the original construction of the subject site reclamation were identified within the Property Files, however two reports pertaining to later proposed developments were identified, which include:

- Beca Carter Hollings & Ferner Limited letter report entitled "Bayswater Marina Geotechnical Review", dated 21 July 2003, and;
- Beca Infrastructure Limited letter report entitled "Beca Infrastructure Limited Letter Report", dated April 2004.

No other information of note was identified within the Property Files.

4.2 2003 Beca Carter Hollings & Ferner Limited Letter Report

The 2003 letter report presents the settlement and stability aspects of the Bayswater Marina reclamation and implication for the proposed future development works, including the construction of new pavements and buildings. Although the report makes mention of previous geotechnical works, none were appended to the letter, or identified within the Property Files.

The report indicates that the marina seawall bund and new reclamation was progressively constructed from late 1994 to 1996. The bund was constructed using a core of 'McCullum's Chip' (chert) and basalt bolder armouring, placed on varying thicknesses of in situ marine sediments and/or Waitemata Group Formation bedrock. The reclamation comprised of varying thicknesses (4 to 6+ m) of marine sediments dredged from the marina basin area to the west. The top 1 to 2 m of reclaimed fill was lime stabilised to create a stiffer crust upon which paving and other infrastructure could be constructed.

The report provides details about the degree of settlement that has been occurred during the three years following the completion of construction. The findings showed that the reclaimed carpark had settled at an average rate of 20 – 30 mm per month during construction, and 10 – 15 mm per month in the following years. The marina seawall bund was measured to have total settlement ranging from 400 – 600 mm in the 1.5 to 2 year period between beginning of construction and completion. This is likely owing to the greater mass and length of time that the seawall construction has been in place compared to the reclaimed fill.

The report discusses that the bunds were design with an initial Factory of Safety (FoS) against slope stability of 1.2 or better at the time of construction to retain the reclamation and pavements. However, they theorised that the FoS of the bund would increase to approximately 1.5 as the ground fill and in situ materials beneath consolidated and increased in density and shear strength.

Concluding recommendations were made regarding potential new developments, these included:

- Filling should be kept to a minimum, as any additional fill material would be likely to result in further consolidation of the underlying materials which would translate into further settlement of the ground surface.
- Any new building would likely be required to be supported piles that were socketed into the Waitemata Group rock at depth, with careful detailing around service connections and access to buildings where the ground is still settling.
- Drainage for stormwater and wastewater was recommended to make use of flexible pipes and connections so as to be able to accommodate any differential movement between buildings and the surrounding land. Pipes were also recommended to be placed at shallow depths, and surface cross-falls should be directed to areas that were expected to settlement more than others.

4.3 2004 Beca Infrastructure Limited Letter Report

The 2004 letter report presents an evaluation of the reclaimed ground conditions and assessment of anticipated settlement for the foundations of a Total Span®, steel and timber framed (9 x 9 m) building and a Versitile® (6 x 10 m) building for temporary use on Lot 4. Both buildings were designed with an intended life span of 3 to 4 years.

The report indicates that the ground underlying Lot 4 consists of 1.2 m of lime stabilised marine muds overlaying 3.2-4.2 m of reclamation fill of dredged marine sediment and 1-2 m of in situ marine muds, all overlying Waitemata Group rock at depth.

The two buildings discussed in the report were to be founded on shallow foundations within the lime stabilised fill. On-going surface settlement as a result of consolidation of the fill and in situ materials beneath was estimated to be up to 100 mm over the 3 to 4 year life span of the buildings, and as such it was recommended that the foundations should allow for 50 mm of differential settlement.

4.4 Bayswater Marina Office Records

KGA were granted access to drawings related to the reclamation construction that are held within the Bayswater Marina Office.

A construction programme identified within the marina office records indicates that the physical reclamation works commenced on 18 July 1994, and continued through to 18 January 1997. The reclamation was undertaken in two stages (north and south), with the southern stage generally indicated to take longer to construct than the northern stage.

Reclamation survey drawings prepared by Hampson & Associates Limited, dated 6 October 1997, along with bund wall cross section As Built drawings prepared by Fletcher Construction dated September 1994 generally indicate that the bund walls were constructed on either the in situ ‘Weathered Sandstone’, or in situ marine sediment. The drawings do not depict any significant undercutting other than dredging to the design depth of the marina basin.

Notes contained within the Fletcher Construction As Built states that no excavation was to take place beneath the intermediate bund (bund constructed between Stage 1 and Stage 2). The details further show that the core of the bunds is to comprise MacCullum’s chip (chert), with heavy rock armouring on the seaward sides of the bunds.

Notes contained on the Hampson & Associated Limited drawings indicate that levels are in terms of Chart Datum, and a correction of 1.74 is provide to convert the levels to be in terms of the Lands & Survey Datum M631 S054463 AL15.345.

We also identified a further Fletcher Construction As Built drawing within the marina office records, entitled “Seabed and Sandstone Contours And Dredge Depth Plan, Stage I”, dated August 1994. This drawing details the extent of the existing historic and subject site reclamations with respect to the seabed and top of in situ rock depths across the site and marina basin. The drawing also indicates the location of the subject site reclamation bund walls.

It is noted that the contours for the seabed and rock level shown on this drawing were in terms of the Auckland Harbour Board Chart Datum. The drawing does not reference any background geotechnical data, so it cannot be determined what information this drawing is based on, however it generally indicates that the alignment of the subject reclamation generally follows the alignment of a shallow sandstone spur that extends to the southwest of the existing historic reclamation.

In addition to the above drawings, we have been provided with scanned copies of undated photographs that were taken during the reclamation works. The photos generally indicate that the intermediate bund was constructed first by progressively end-tipping granular material out into the harbour from the historic reclamation. Once the intermediate bund wall was extended to the western extent of the reclamation, the bund construction then turned northwards while another bund wall was extended to the west from the northwestern corner of the historic reclamation. These two walls eventually joined up to create the bund walls around the Stage 1 area. The Stage 2 area was constructed similarly whilst dredging's from the marina basin were beginning to be placed inside the Stage 1 walls. Notably, the construction photographs show that the intermediate bund was armoured, despite the cross section drawings identified detailing that the intermediate bund was not supposed to be armoured.

4.5 Hampson & Associates Topographical Survey

We are in recipet of a topographical survey plan of the subject site, prepared by Hampson & Associates Limited, entitled 'Bayswater Marina', dated 20 December 2013. The drawing provides spot heights and shows the location of existing structures, vegetation and paved areas within the subject site.

The levels shown on the Hampson & Associates Limited 2013 drawing are indicated to be in terms of the Geodetic datum 2000, which we understand has a vertical difference of 1.743m from the Auckland Harbour Board Chart Datum referenced on the mid 1990's Hampson & Associates Limited and Fletcher Construction drawings.

We understand that the 2013 Hampson & Associates Limited drawing has been used as the basis for the levels for the proposed development, and where appropriate, the contour information presented on these drawings has been transposed on to our drawings presented within Appendix 1.

4.6 Historic Aerial Photograph Interpretation

An examination of historic aerial photographs available through the online resource Retrolens, as well as the Auckland Council online GIS service, GeoMaps, has been undertaken.

The oldest photograph available dates from April 1940, which indicates the historic reclamation that neighbours the site to the east is already in existence at that time.

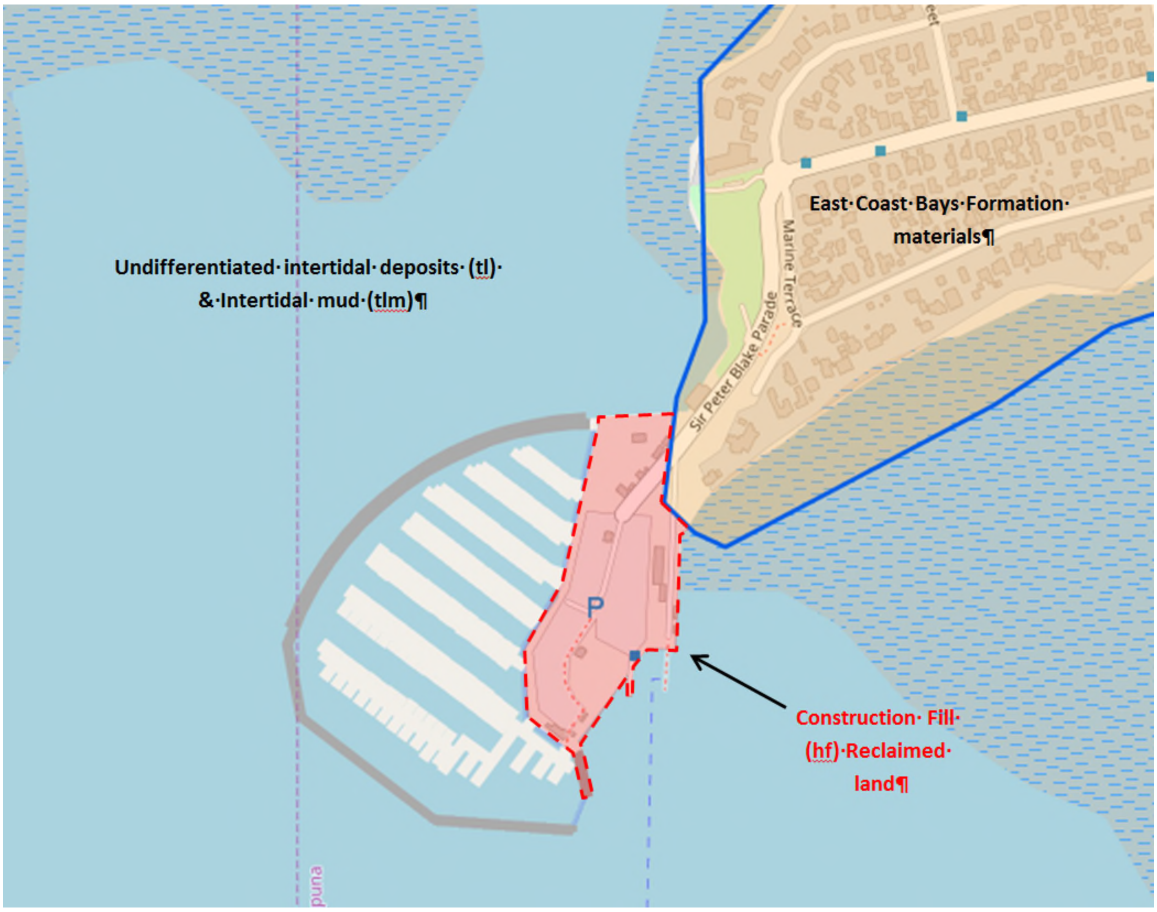
A photograph from 1996 depicts Stage 1 of the reclamation being largely completed, whilst Stage 2 is still underway; the marina breakwater structure has also been constructed at this time.

Subsequent photos identified show that the marina has been established and the current car parking in place and in use, as well as the establishment and progressive growth of trees around the site.

5. GEOLOGY

The geology of the site and surrounding area is detailed on the geologic map entitled "Geology of the Auckland Area', Institute of Geological & Nuclear Science, scale 1:250 000, geological map 3. This shows that the geology beneath the site and nearby properties comprises of Holocene aged, human-made deposits (reclaimed land) overlying an eroded platform of East Coast Bays Formation materials of the Warkworth Subgroup, part of the Waitemata Group materials of Miocene age. The surrounding harbour consists of undifferentiated intertidal deposits & intertidal.

Figure 1. Regional geology map extract from GNS Geology Web Map



The text associated with the geologic map describes the Holocene human-made deposits as re-compacted clay to gravel sized materials; which may include demolition debris.

The East Coast Bays Formation materials were originally deposited as submarine landslide, or turbidity current deposits, that have since lithified into alternating layers of siltstone and sandstone, which have subsequently been uplifted to be exposed above sea level today. Since being exposed above sea level, the materials have weathered in situ to form a mantle of residual soil over the rock and depth. Weathering in these materials is often gradational, with no distinct soil/rock boundary present.

The surrounding harbour materials are shown to consist of undifferentiated intertidal deposits and intertidal muds.

Regardless of the above, we point out that geologic maps are largely generated on a regional scale, often by utilising remote sensing techniques rather than direct subsurface information. As a result, the onsite presence or absence of any material indicated on the geologic map can only be confirmed via site specific, subsurface investigation methods.

6. FIELD EXPLORATION

6.1 Overview

Our investigation was undertaken in two stages, and comprised:

- Fifteen rotary cored boreholes (MH1 to MH15);
- Seven Cone Penetration Tests (CPTs) (CPT1 to CPT3), CPT(MH09), CPT(MH11), CPT(MH13) and CPT(MH15), and;
- Two mechanically excavated test pits (TP1 and TP2).
- Three push-tube samples were obtained, on each for MH5, MH6 and MH7.

The location of each of our investigation points was initially determined based on the location of the proposed development along with the layout of existing buildings, as well as existing private and public buried services on site.

Each investigation point was initially located by KGA using a hand held GPS, with each investigation location then being checked for buried services by Underground Service Locators Limited using both a service scanner and ground penetrating radar.

Following completion of our fieldwork, each investigation point was located by measuring the distance to each point from several known, fixed site features, as shown on the HAL topographic survey plan. Elevation control for each point was established by triangulating from the spot heights shown on the HAL survey plan. The HAL survey co-ordinates are in terms of the Geodetic Datum 2000, and elevation control is in terms of Auckland Vertical Datum 1946.

The location of each investigation point is indicated on our drawing entitled "Site Plan, Aerial Image Underlay", Sheet No. KGA 1, presented within Appendix 1. Copies of the logs of our subsurface investigation points are also presented within Appendix 1.

6.2 Rotary Machine Drilling

Machine boreholes MH1 to MH8 undertaken by DCN Drilling Limited (DCN) using a drill rig mounted on a trailer between 21 May and 1 June 2018. Machine boreholes MH9 to MH15 undertaken by Pro-Drill Limited (PDL) using a Fraste Eijkelkamp sonic drill rig on 6 and 7 July, 2020.

The core recovered from each drill hole was logged by a KGA team member. Piezometers standpipes were installed in boreholes MH1 to MH8 in order to allow the long term monitoring of groundwater levels in the drill locations. Push tube samples were obtained at various depths from boreholes MH5, MH6 and MH7 for later laboratory testing. The push tubes were sealed with wax immediately upon extraction in order to preserve each samples as close as possible to its natural moisture content.

6.3 CPT Probes

Three CPT's were undertaken by DCN on 28 May 2018, and four CPT's were undertaken by PDL on 6 and 7 July 2020. All soundings were intended to be advanced to a depth of 15 m below present ground level, or until effective refusal, whichever is encountered first. The primary purpose of the probes was to provide additional data that can be correlated against the machine boreholes to allow for a greater range of geotechnical parameters to be established for the subsurface materials, and to allow preliminary liquefaction assessments to be undertaken.

The CPT work undertaken by DCN used a small portable machine, using a cone with a cross-sectional area of 15cm². The CPT probes undertaken by PDL utilised equipment mounted on the same Fraste Eijkelkamp drilling rig that drilled their boreholes. For three of the CPT's undertaken by PDL (CPT(MH11), CPT(MH13) and CPT(MH15)) the sonic drilled hole was back-filled with fine-grained sand before the CPT probe was advanced down the same hole. CPT(MH09) was advanced from ground surface approximately 1m away from the borehole drill location.

Continuous measurements of pore pressure was undertaken during all CPT soundings. Tests were undertaken in accordance with A.S.T.M. Standard D 5778-12 procedure.

6.4 Mechanically Excavated Test Pits

Excavation services for the test pits was undertaken Frogley Earthmoving Limited on 31 May, 2018, with the materials recovered logged by a KGA representative. Both test pits were to be taken to the limit of the excavator (approximately 4.5m depth), or until effective refusal, whichever first.

6.5 Logging Standard & Shear Strengths

The ground conditions encountered within each rotary machine borehole and test pit were logged in general accordance with 'The guidelines for the classification and description of soil and rock for engineering purposes', December 2005 as outlined by the NZ Geotechnical Society.

A calibrated shear vane, used in accordance with New Zealand Geotechnical Society Guideline for Hand Held Shear Vane Test, 2001 was used at regular depths within the rotary machine borehole and test pits, in order to measure soil strengths, both in situ and remoulded. The vane shear strengths shown on the attached logs have been corrected in terms of BS 1377.

6.6 Groundwater Measurements

All test pits were checked for standing groundwater on 31 May 2018.

The machine boreholes MH2, MH3, MH7 and MH8 were checked for standing groundwater on the 1 June 2018, at which time automated, barometric groundwater level loggers were installed inside the piezometers placed inside these boreholes. The level loggers were left running continuously until 15 June 2018, when they were extracted. On 15 June, 2018, all machine borehole levels were checked for standing groundwater again, with the exception of MH1, which could not be checked as access was obstructed due to a vehicle being parked over the piezometer cap.

Where possible, the groundwater level within the CPT probes were checked by DCN Drilling Limited immediately following extraction of the probe.

The results of the groundwater monitoring period are discussed in further detail in Section 9 below.

Based on the findings of our 2018 groundwater monitoring period, the Pro-Drill Limited drilled boreholes were not checked for standing groundwater as it was assumed that similar findings to our 2018 monitoring period with automated data loggers would be obtained.

7. LABORATORY TESTING

After drilling, the push tube samples obtained from boreholes MH1, MH5, MH6 and MH7 were submitted to the IANZ accredited Babbage Geotechnical Laboratory (BGL) for the purpose of Hydrometer Particle-Size Distribution Testing and Atterberg Limits Testing.

For these testing undertaken, the following standards were used:

- NZS4402:1986:Test 2.1 (Water Content)
- NZS4402:1986:Test 2.2 (Liquid Limit)
- NZS4402:1986:Test 2.3 (Plastic Limit)
- NZS4402:1986:Test 2.4 (Plasticity Index)
- NZS4402:1986:Test 2.8.1 (Wet Sieve Test)
- NZS4402:1986:Test 2.8.4 (Hydrometer Test)

Upon extruding the samples for testing, it was identified that insufficient sample was recovered within the tube obtained from MH1, and therefore that sample was discarded.

Copies of the test results are presented within Appendix 4, while the results are discussed in further detail in Section 10 below.

8. SUBSURFACE CONDITIONS

8.1 Materials Encountered

The subsurface ground conditions encountered in each test pit and rotary machine borehole are briefly described below, and summarised in Table 1 below. For a full detailed description of the subsurface conditions encountered reference should be made to the logs presented within Appendix 1.

Fill (Reclaimed Land): Fill was encountered from ground surface in boreholes MH1, MH2, MH3, MH5, and MH7 to MH15, and in test pits TP1 and TP2.

The fill encountered from ground surface included asphalt, base course and topsoil. Beneath the asphalt, base course and topsoil, the fill was found to comprise soft to hard marine clay and silt, and very loose to loose sand and minor gravel, with varying subordinate fractions of each. Shells, and fragments of shells, were frequent throughout, and minor organic inclusions were also identified. Uncorrected SPT 'N' values taken within the fill often returned values of 'N' = 0, however in boreholes MH3, MH5 and MH12, the SPT 'N' values at 1.5m depth returned uncorrected values of 'N' = 20, 13 and 13 respectively.

It should be noted that, approximately the top 1.5 – 2.0m of fill was found to be notably stiffer than the remainder of the fill at depth. We consider that this corroborates our desk studying findings, which indicated that lime stabilisation of the surficial materials was undertaken as part of the reclamation works (See Section 4).

Bund Armour: The armouring of the bunds used around the perimeter of the reclamation works consisted of cobble to bolder size basalt, greywacke and concrete, with a matrix of fine gravel to cobble size scoria, greywacke and chert. The bund armouring was encountered within MH4 and MH5.

MH5 was anomalous in that it was drilled near to, but off the documented alignment of the intermediary bund constructed between the northern and southern reclamation areas. The construction As-Built drawings identified suggest that no armouring of this bund was to take place, however copies of construction photographs provided by Bayswater Marina Office suggest that armouring was undertaken.

It is possible that the bund armouring encountered within MH5 at depth is the very toe of the armouring that was placed on the intermediary bund.

Additionally, it is considered likely that the armouring boulders for this bund, particularly if these boulders were dropped from height instead of being specifically placed, could possibly have rolled off the bund and sunk into the alluvium following construction.

Bund Fill: Bund fill was encountered in boreholes MH2, MH4, MH6, MH13 and MH15, which consisted of fine to coarse gravel size chert (McCallum's Chip). Due to the strength, size, presence of water and the otherwise loose nature of the bund fill material, core loss was experienced within these materials in all five boreholes where encountered. The core loss is considered to be a result of the material being advanced downward with the drill and/or falling out of the core barrel upon extraction. Where undertaken, uncorrected SPT 'N' values recorded within the bund fill materials ranged from 'N' = 0 to 'N' = 37.

Difficulties were also experienced in establishing the exact base depths of the bund fill in boreholes MH2, MH4 and MH6. In many places it is likely the advancing drill for these holes was pushing bund gravel down into the soft, saturated, in situ alluvium below, in which poor recovery was also experienced. Similarly, as the larger gravel pieces from the bund fill were pushed downwards into the softer material beneath, it is also possible that some of the softer materials may have also been flushed out with the drilling fluid. Despite this, using a combination of the levels from the identified Seabed & Sandstone Contours Plan, and uncorrected SPT 'N' values at depth, as well as the sonic drilling data from MH13 and MH15, we have established the likely base depth of the bund in MH2, MH4, MH6, MH13 and MH15. The approximate level of the base of the bunds has taken into account some settlement of the bunds into the alluvial material below which will have taken place since construction was completed.

Alluvium (Harbour Mud): Alluvium, considered to be in situ harbour mud, was encountered underlying the reclaimed fill and bund fill in boreholes MH1 to MH6, MH9, MH10 and MH14. The alluvium was found to comprise very loose to loose silt and sand with varying subordinate fractions of clay and silt. Uncorrected SPT 'N' values taken within the alluvium ranged between SPT 'N' = 0 to 8.

We point out that, as the reclaimed fill is of similar origin to the alluvium, no definitive boundary between the two materials could be established during drilling on site. Instead, we have again made use of the identified Seabed & Sandstone Contour plan to establish an approximate horizon change, between materials, where appropriate.

Residual Waitemata Group Soils (WGS): A thin horizon of residually weathered, Waitemata Group soil was identified beneath the alluvium in boreholes MH3 to MH8, and in MH14. Where present, these materials were often found to comprise loose to medium dense sand with varying subordinate fractions of clay and silt.

Waitemata Group Rock (WGR): Waitemata Group rock was encountered at depth in all boreholes (except for boreholes MH11 and MH13) beneath all other materials encountered. These materials were found to comprise alternating sandstone and siltstone beds that were unweathered and otherwise homogenous with very little fracturing identified. Sandstone beds were found to range in thickness between 50mm to 80mm, whilst the siltstone beds were found to range in thickness between 10mm to 30mm. Uncorrected SPT 'N' values within these materials were generally greater than 'N' = 50+.

Table 1. Summary of Materials Encountered.

Point ID	Surface RL	Fill (Reclaimed Land)	Bund Armor	Bund Fill	Alluvium (Harbour Mud)	Residual WGS	WGR
TP1	4.1	0.0 – 2.0	n/a	n/a	n/a	n/a	n/a
TP2	4.16	0.0 – 3.0	n/a	n/a	n/a	n/a	n/a
MH1	3.43	0.0 – 6.0	n/a	n/a	6.0 - 7.0	n/a	7.0 – 10.11
MH2	2.88	0.0 – 1.5	n/a	1.5 – 5.5	5.5 – 7.5	n/a	7.5 – 10.71
MH3	3.26	0.0 – 5.8	n/a	n/a	5.8 – 7.95	7.95 – 8.4	8.4 – 11.61
MH4	2.62	0.0 – 1.4	1.4 – 1.8	1.8 – 5.35	5.35 – 7.3	7.3 – 8.8	8.8 – 11.87
MH5	3.79	0.0 – 5.45	n/a	n/a	5.45 – 8.0	8.0 – 9.5	9.5 – 12.6
MH6	2.84	0.0 – 0.4	0.4 – 0.8	0.8 – 5.0	5.0 – 8.1	8.1 – 10.45	10.45 – 12.3
MH7	4.16	0.0 – 5.9	n/a	n/a	n/a	5.9 – 6.25	6.1 – 9.12
MH8	3.85	0.0 – 5.85	n/a	n/a	n/a	5.85 – 6.8	6.8 – 9.3
MH9	3.29	0.0 – 5.7	n/a	n/a	5.7 – 6.3	n/a	6.3 – 9.14
MH10	3.96	0.0 – 6.7	n/a	n/a	6.7 – 7.8	n/a	7.8 – 10.07
MH11	2.67	0.0 – 4.5	n/a	n/a	n/a	n/a	n/a
MH12	4.76	0.0 – 7.0	n/a	n/a	7.0 – 7.1		7.1 – 10.62

MH13	3.07	0.0 – 1.5	n/a	1.5 – 3.2	3.2 – 4.3	n/a	n/a
MH14	3.46	0.0 – 6.0	n/a	n/a	6.0 – 8.0	8.0 – 10.0	10.0 – 12.62
MH15	2.98	0.0 – 3.7	n/a	3.7 – 5.2	5.2 – 8.1	n/a	8.1 – 11.08

Note, all depths indicated are in metres n/a = not encountered

Surface RL from Hampson & Associates survey data

8.2 Inferred Materials

The CPT probes undertaken did not return any specific samples for logging. However, inferences have been drawn from the probe results to the materials logged in nearby investigation points. The inferred materials from the CPT probes are discussed below, and a summary of the inferred materials is presented within Table 2.

Fill (Reclaimed Land): Fill is inferred to be present from ground surface in CPT01, CPT02 and CPT03. The CPT probe soil behaviour type classification output generally inferred similar material types to what was found within the nearby investigation points, namely clay silt and silty clay type materials, with only minor horizons of sand identified towards the base of each probe.

The cone resistance plot of the CPT soundings for CPT01 to CPT03, and for CPT(MH09) shows high resistance within the upper 2m of each sounding, which also correlates to the boreholes and desk study findings that the upper horizons of the reclaimed fill was subject to lime stabilisation.

The cone resistance plots for CPT(MH11), CPT(MH13) and CPT(MH15) are not as straightforward to interpret given that part of each of these sounding was undertaken through loose, fine-grained sand material that was used to back-fill the sonic drilled boreholes. The plots for CPT(MH11) and CPT (MH13) clearly show a zone of increased and varying cone resistance below the base of the drill depth, and a similar zone was also identified within CPT(MH15) at depth, however, suggesting that the probe at that point is likely being advanced through fill materials.

Alluvium (Harbour Mud): The cone resistance plots for CPT01 to CPT03 does not suggest any distinct horizon change between the fill and in situ alluvium. With reference to the identified Seabed and Sandstone Contour plan, at each CPT location, the interface between the reclaimed fill and in situ alluvium is estimated to be at approximately 5.7m below present ground level for CPT01 to CPT03.

The cone resistance plots for CPT(MH11) and CPT(MH15) both indicate a horizon of very low and uniform cone resistance beneath the likely fill materials. This horizon of material is inferred to be the in situ harbour mud.

Waitemata Group Materials: The cone resistance plots from all of the CPT probes show a sudden spike in resistance towards the base of each probe. This suggest that each probe has slightly penetrated into the very top of Waitemata Group soils/rock materials at depth before each probe was terminated.

Table 2. Summary of Inferred Materials.

Probe ID	Surface RL	Fill	Alluvium (Harbour Mud)	Waitemata Group Materials
CPT01	4.25	0.0 – 5.7	5.7 – 7.13	7.1.3 – 7.33
CPT02	3.48	0.0 – 5.7	5.7 – 7.57	7.57 – 8.37
CPT03	3.73	0.0 – 5.7	5.7 – 6.84	6.84 – 7.21
CPT(MH09)	3.29	0.0 – 4.8	4.8 – 6.0	6.0 – 6.1
CPT(MH11)	2.67	0.0 – 6.5	6.5 – 7.9	7.9 – 8.4
CPT(MH13)	3.07	0.0 – 4.7	4.7 – 4.9	4.9 – 6.2
CPT(MH15)	2.98	0.0 – 6.2	6.2 – 7.1	7.1 – 8.1

Note, all depths indicated are in metres Surface RL from Hampson & Associates survey data

8.3 Subsurface Model

Using the topographic survey information provided, the results of our desk study and our subsurface investigation, we have generated three cross sections through the location of the bund in selected locations around the site (A – A’, B – B’ & C – C’) in order to graphically represent our objective interpretation of the subsurface conditions in these parts of the site. The locations of our cross sections are shown on our drawing entitled “Site Plan, Aerial Image Underlay”, Sheet KGA 1, with each cross section shown on our drawings entitled “Cross Section A - A’”, “Cross Section B - B’” and “Cross Section C - C’”, Sheet KGA 3, 4 and 5 respectively, presented within Appendix 1.

We point out that in preparing our objective subsurface models, the ground conditions have been inferred between and away from our investigation points. It must be accepted, however, that, the conditions may vary between each investigation point from what has been inferred on our models.

Additionally, considering the discrete nature of the information obtained compared to the overall extent of the investigation area, assumptions and inferences have been made from the investigation data obtained for those parts of the site where subsurface information is otherwise sparse. In this respect, we point out that there is less certainty as to the location, presence, or absence, of materials at greater distances away from our investigation points.

9. GROUNDWATER MONITORING RESULTS

The depth to groundwater was monitored for a period of two weeks, between 1 to 15 June 2018, in the piezometers that were installed within machine boreholes MH2, MH3, MH7 and MH8. The monitoring was undertaken in order to gain an indication of the groundwater response to rainfall, and, as the site is adjacent to the Waitemata Harbour, to see if tidal fluctuations had any effect on the standing groundwater level.

An indicative historic daily rainfall record for the general area of the site was taken from the Meteorological Service of New Zealand Limited (Metservice) Auckland Airport weather station. An indicative daily tide record was taken from the Land Information New Zealand (LINZ) Auckland tidal gauge. The daily rainfall records, together with the recorded groundwater levels in each borehole are shown in Figure 2 below. The tide records together with the groundwater levels in each borehole are shown in Figure 3 below.

Figure 2 - Groundwater Monitoring Results vs Recorded Rainfall

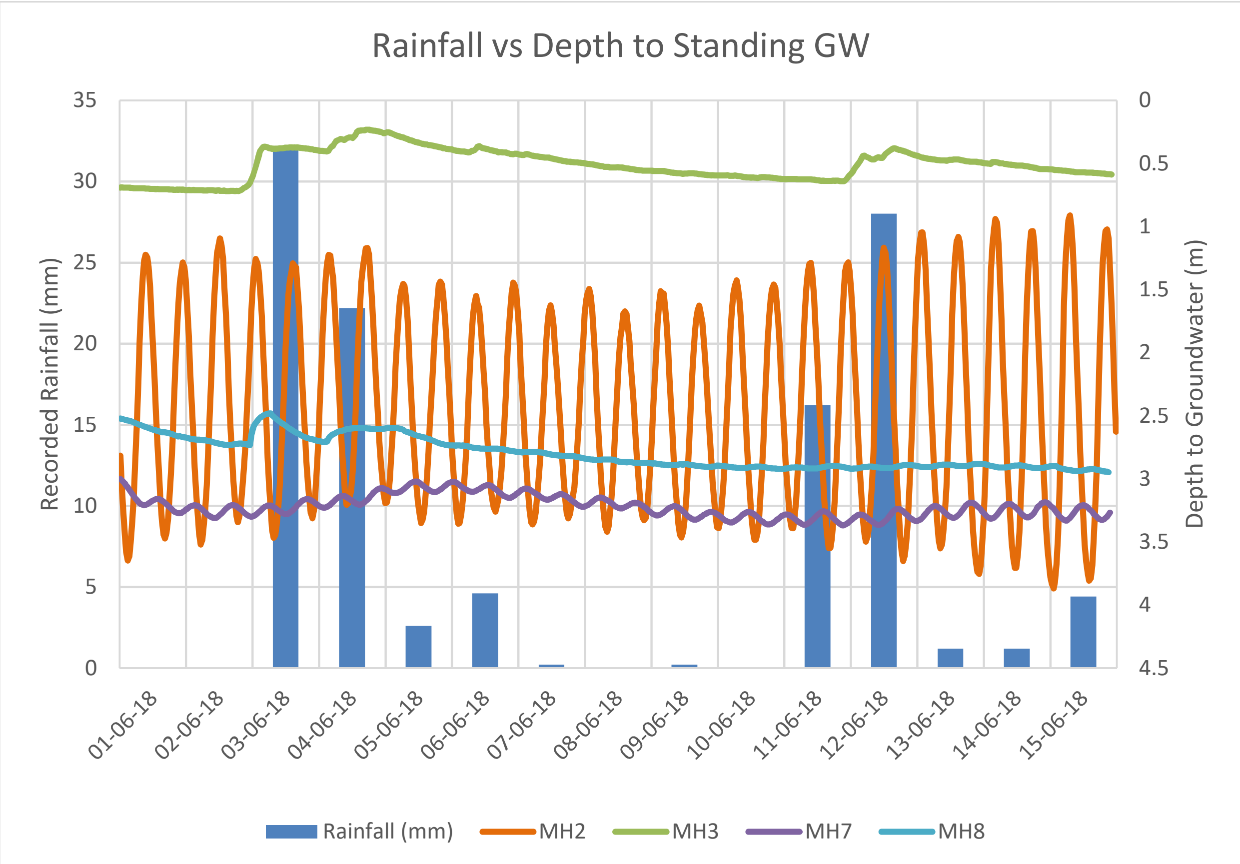
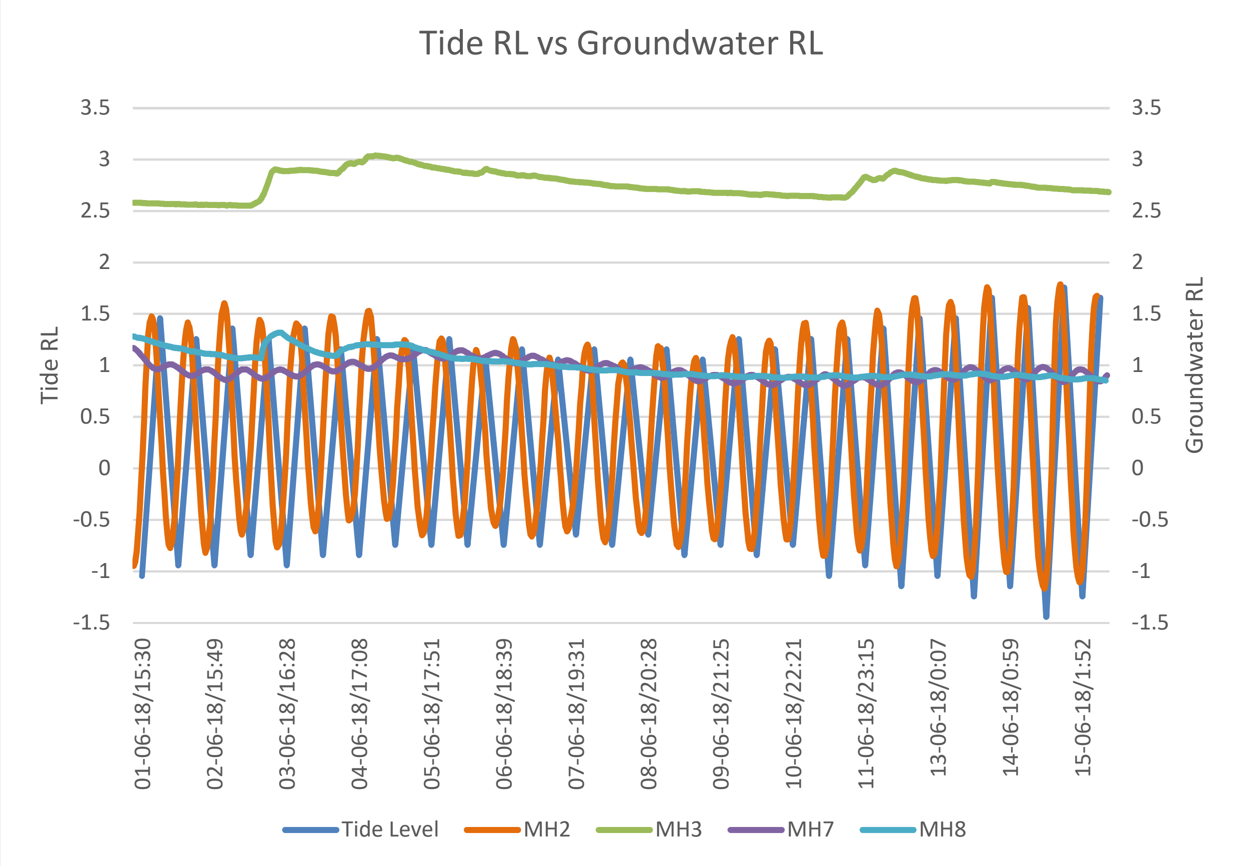


Figure 3 – Groundwater Monitoring Results vs Recorded Tide Levels



From the monitoring, the following observations and comments can be made:

- MH2 showed a notable response to tidal fluctuations.
- MH3 showed a response to rainfall, but not to tidal fluctuations.
- MH8 showed a response to the rainfall events around 3-6 June 2018, but not to the rainfall around 11-15 June 2018. After the effects of the 3-6 June 2018 rainfall had dissipated, MH8 showed only a very minor response to tidal fluctuations.
- MH7 showed a delayed and minor response to tidal fluctuations. MH7 also appeared to show a minor and delayed response to rainfall following the rainfall event around 3-6 June 2018, and also possibly to the rainfall event around 11-15 June 2018.
- MH3 was anomalous in that groundwater was recorded at much shallower depths than in any other borehole. Attempts to purge this Piezometer were fruitless, as groundwater was found to recharge the piezometer just as fast as it could be purged. In this instance we consider that, as the borehole was drilled within the carpark, it is possible that the pavement basecourse material is saturated, and that water was able to enter the Piezometer and recharge it up to the level we measured.

- It is possible that the seal around the top of MH8 had not properly formed prior to the rainfall events of 3-6 June, but that it had effectively sealed following that event, which is why no further response to rainfall was noted in borehole MH8.
- Whilst MH7 is one of the boreholes located farthest from the sea, the tidal response shown in this piezometer is likely due to seawater tracking through the intermediary, high permeability bund, and also along the interface between the historic reclamation revetment to the east and the subject site.
- The tidal response shown later in the monitoring period in MH8 is considered likely to be influenced similar to MH7.

Ignoring the anomalous reading in borehole MH3, the maximum, minimum and average levels recorded over the monitoring period are presented within Table 3 below:

Table 3. Summary Measured Groundwater and Tidal Levels Recorded from 1 June to 15 June 2018.

Location ID	Maximum RL	Minimum RL	Average RL
MH2	1.79	-1.17	0.29
MH7	1.17	0.80	0.95
MH8	1.32	0.85	1.01
Sea Level	1.76 (high tide)	-1.44 (low tide)	0.16

Note, all levels are in terms of Auckland Vertical Datum 1946

As indicated in Table 3, and in Figure 2, high tide RL is above the maximum water level in boreholes MH7 and MH8, however the average tide level is lower than the average water in both MH7 and MH8.

Based on the above, unless located within 5m of an existing bund within the site, the average groundwater level is otherwise considered to be at approximately RL1.0. Near to and within the existing bunds, however, groundwater levels should be expected to fluctuate with tidal variations.

Lastly, as the groundwater levels are considered to be sensitive to tidal fluctuations, and the sites general proximity to the sea, groundwater beneath the site should be considered to be saline. For conservatism, in lieu of any specific testing, we suggest that the groundwater should be assumed to be of an equivalent salinity to sea water.

10. LABORATORY TEST RESULTS

10.1 Particle Size Distribution

Hydrometer Particle-size Distribution Tests were performed on the samples extruded from the push tubes taken from boreholes MH5, MH6 and MH7. The results summary table presented within the BGL laboratory report has been reproduced in Table 4 below.

Table 4. Summary of Particle Size Distribution tests.

Borehole ID	Depth (m)	Hydrometer Grading (% of Dry Mass)			
		Gravel (2 - <9.50mm)	Sand (0.06 – 2.0mm)	Silt Fraction (0.002 – 0.06mm)	Clay Fraction (< 0.002mm)
MH5	4.50 – 5.00	2	25	38	35
MH6	8.10 – 8.70	0	76	15	9
MH7	6.00 – 6.25	1	24	36	39

The testing generally suggests that the samples from MH5 and MH7 are silt/clay dominant and therefore more likely the behave as plastic materials. The sample from MH6, however, was more sand dominant with only a very minor clay component.

10.2 Atterberg Limit Testing

The Atterberg limit test were performed on the samples extruded from the samples extruded from the push tubes taken from boreholes MH5 and MH7. The results table presented within the BGL laboratory report has been reproduced in Table 5 below:

Table 5. Summary Atterberg Limit tests.

Borehole ID	Sample Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
MH5	4.5 – 5.0	56.1	75	25	50
MH7	6.0 – 6.25	65.5	77	26	51

In terms of the Unified Soil Classification System (USCS) plasticity (Casagrande) chart, both samples plotted above the A-Line and are therefore classified as highly plastic clay.

11. SITE CLASSIFICATIONS

In general it is considered that the existing site soils are susceptible to swelling and shrinking under seasonal variations of water content. The November 2019 Amendment 19 of NZBC Structure B1/AS1, clause 3.2 “slab-on-ground in expansive soils” provides a new Clause 7.5.13, of the same name, into NZS3604. For the purposes of design, the site may be designated as Moderately Reactive (Class M) in accordance with Clause 7.5.13.1.2 within that document. Soils within this expansive class can be assumed to have an SLS 500 year design characteristic surface movement of 44mm.”

We point out that the soil reactivity classification given for the near surface soils has been done so with the knowledge that the upper horizons of fill were chemically dried through the addition of lime. In this sense, it should be noted that the two samples used for Atterburg limit testing, as discussed in Section 10.2 above, were obtain from deeper soil horizons where no lime was added during construction.

The site has been classified as Subsoil Class C in accordance with NZS1170.5:2004 (seismic design).

12. LIQUEFACTION ASSESSMENT

Liquefaction analyses were completed for both SLS (annual exceedance probability of 1/25) and ULS (annual exceedance probability of 1/500) design criteria, using the CPT data referenced in Table 2 above. Calculations were performed over the full depth investigated at the probe locations of CPT1, CPT2, CPT3 and CPT(MH09). The calculations performed on the sounding results of CPT(MH11), CPT(MH13) and CPT(MH15) were undertaken from the base of the borehole back-fill material that was poured into the hole to maintain borehole stability, prior to the CPT probe being undertaken.

The seismic design requirements adopted for use in the analyses are in accordance with Section 5.1 of Module 1 of the Earthquake Engineering Practice (MBIE and NZGS), which recommends the use of unweighted seismic hazard factors as per the New Zealand Transport Authority Bridge Manual (2014). These are:

- Buildings of higher importance than normal use (Importance Level 2).
- Shallow soil sites (Class C).
- Magnitude M5.8 EQ event and Peak Ground Acceleration (PGA) of 0.04g (SLS), and 0.15g (ULS).
- Boulanger and Idriss (2014) methodology for liquefaction triggering.
- Zhang et al. (2002) volumetric densification calculation.

For the purposes of the analyses, the in situ groundwater level, both prior to and during an earthquake, was assumed exist at 1.0m below ground level.

We have analysed the CPT data using the software 'CLiq2' and a copy of the output from the analyses has been included within Appendix 5. The software includes for normalisation of the data for overburden pressure and is considered to provide improved indications of liquefaction potential. The results are presented in Table 6.

Table 6: Liquefaction Induced Global Settlements Analysis Results

Test	Calculation Limit Depth (m)	Calculated Vertical Settlement (mm)	
		SLS	ULS
CPT01	7.33	0	< 10
CPT02	8.37	0	< 10
CPT03	7.21	0	< 10
CPT(MH09)	6.15	0	10
CPT(MH11)	4.5 – 8.45	0	35
CPT(MH13)	4.2 – 6.3	0	15
CPT(MH15)	6.00 – 8.2	0	< 10

Under SLS seismic demands, the above calculated settlements indicate that the liquefaction potential is negligible for lower levels of seismic demand, and low to medium for ULS seismic demands, with predicted soil subsidence of less than 10mm across much of the site after a 1/500 year seismic event. The testing has highlighted, however, that localised settlements could be higher, with one analysis result suggesting ULS settlements in the order of 35mm. We point out that the predicted liquefaction under ULS demands usually occurs in discrete layers at depths greater than 5m below present ground level.

The settlements presented in Table 6 describe the settlement of ground not occupied by a building, occurring due to dissipation of excess pore water pressure generated during earthquake shaking.

The Liquefaction Potential Index (LPI) is a parameter developed by Iwasaki (1978) that indicates the expected severity of damage due to liquefaction. We have analysed the CPT data using the current software 'CLiq2, and the calculated LPI values are presented in Table 7 below.

Table 7: Liquefaction Potential Index (LPI)

Test	LPI values	
	SLS	ULS
CPT01	0	< 1
CPT02	0	< 1
CPT03	0	< 1
CPT(MH09)	0	< 1
CPT(MH11)	0	1
CPT(MH13)	0	< 1
CPT(MH15)	0	< 1

Key to LPI Values	
LPI Values	*Risk Estimation*
0 – 5	*Low risk*
5 – 15	*High risk*
15+	*Very high risk*

The LPI values presented in Table 7 suggest that the site soils have low liquefaction risk potential for the modelled SLS and ULS level events.

Based on the CLiq2 analyses undertaken, we consider that the site presents a low potential for liquefaction. However, the reclaimed and in situ alluvium (harbour mud) comprises highly compressible/sensitive soils that results in an effective susceptibility to a loss of shear strength during an earthquake event. This process is called cyclic softening, and will result in further vertical settlement, and form a layer which may induce some form of soil lateral spreading in the area near the free face to the harbour. In this case, the soil would potentially squeeze out, or deform laterally beneath the loading of the upper soil crust, and/or any structure that makes use of shallow foundations.

In addition to the CLiq2 assessments, we also used the laboratory test results, as described in Section 10.1, to assess the liquefaction susceptibility of the soils, and compare them to the results with the CPT based liquefaction assessment. When comparing the results of the grading tests undertaken to the grading envelopes published by the Ministry of Transport in Japan, this suggests that the samples from MH5 and MH7 are clearly outside the range of soils with possibility of liquefaction, but that the sample from MH6 is within the range of soils with the possibility of liquefaction.

The plasticity index (PI) of the samples tested, as described in Section 10.2, was also checked in order to assess the liquefaction susceptibility of the soils, which is characterised as follows:

- PI<7 – Soil behaves as sand-like soils, and may be susceptible to liquefaction.
- 7<=PI =12 – Soils moderately susceptible to liquefaction.
- PI>12 – Soils behave as clay-like soils and are not considered susceptible to liquefaction.

Again this reinforces the conclusion that MH5 and MH7 correspond to soils that are not susceptible to liquefaction, but that MH6 is potentially liquefiable. This conclusion appears to be in line with the CPT based liquefaction assessment, which shows that sandy like materials were found below the alluvium (harbour mud) marking the transition to the rock.

13. STABILITY ANALYSES

13.1 Slope Stability

Aside from the gently undulating nature of the site, which is considered to be related to localised settlement as a result of post reclamation consolidation of the reclaimed fill and in situ alluvium materials below, the visual assessment undertaken on the property identified no evidence of current instability, either shallow or deep seated.

In order to assess the stability of the existing bund around the site, and the effect of the proposed development on the stability of the site, we have used our subsurface models (discussed in Section 8.3) as the basis of a series of computational slope stability assessments. To model the proposed ground surface, we have utilised the proposed final surface information shown on the drawing set prepared by ACL.

Details of the cross section are shown on our drawings entitled “Cross Section A - A’”, “Cross Section B - B’” and “Cross Section C - C’”, Sheets KGA 3, 4 and 5 respectively, presented within Appendix 1, and also within the slope stability calculations contained in Appendix 6. The analyses were performed using the computer-based programme “SLIDE” by Rocscience.

In carrying out our analyses, we have modelled the existing ground surface utilising four separate load case scenarios:

1. Static conditions, groundwater at high-tide level,
2. Seismic conditions with a seismic horizontal acceleration coefficient of 0.15g, calculated in accordance with the NZGS Guidance Module 1: Overview of The Guidelines,
3. Seismic conditions with a seismic horizontal acceleration coefficient of 0.10g, calculated in accordance with the Auckland Council Code of Practice for Land Development and Subdivision, and,
4. Static conditions, with reduced parameters to model the conditions immediately after the cessation of shaking from a ULS earthquake event, when the soils would be reduced to residual strength values due to cyclic softening.

For all scenarios modelled we have assessed both potential circular and non-circular surfaces using the Spencer method. In conducting our non-circular surface analyses, we have used both the Path search and Block search methods. The geotechnical parameters used in the analyses are presented within Table 8 below:

A summary of the Factor of Safety results from the analyses are presented in Table 9 and 10 below. For the detailed results, however, please refer to Appendix 6.

Table 8: Geotechnical Parameters used in Stability Analyses

Stratum	Bulk Unit Weight γ (kN/m^3)	Effective Cohesion c´ (kPa)	Effective Angle of Friction, Ø´ (deg)	Undrained Shear Strength s_u (kPa)	Seismic Reduced Effective Cohesion c' (kPa)	Seismic Reduced Effective Angle of Friction, Ø´ (deg)
Bund Core	20	n/a	38	n/a	n/a	34
Bund Armour	22	n/a	40	n/a	n/a	38
Lime Stabilised Reclaim Fill	16	5	30	60	"Vertical Stress Ratio" $s_u/\sigma_v' = 0.07$	
Un-Stabilised Reclaim Fill	15	5	20	12.5	"Vertical Stress Ratio" $s_u/\sigma_v' = 0.06$	
Alluvium (Harbour Mud)	15	3	23	12.5	"Vertical Stress Ratio" $s_u/\sigma_v' = 0.05$	
Residual Waitemata Group Soil	18	5	35	100	5	35
Waitemata Group Rock	20	Generalised Hoek-Brown Parameters Used				
Proposed Fill	16	5	30	60	"Vertical Stress Ratio" $s_u/\sigma_v' = 0.07$	
Proposed Gabion Wall Bedding	20	0	38	n/a	n/a	
Proposed Gabion Wall	24.2	Infinite Strength				

Table 9: Factor of Safety obtained in Stability Analysis – Existing Site Profile

Cross Section	Factor of Safety											
	Static, current conditions			Seismic (NZGS)			Seismic (ACCOP)			Static, immediately after shaking		
	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)
A – A'	2.0	1.9	1.9	1.4	1.4	1.4	1.7	1.7	1.9	1.8	1.8	1.6
B – B'	2.0	1.9	1.9	1.6	1.6	1.6	1.9	1.8	1.8	1.8	1.7	1.7
C – C'	1.9	1.7	1.7	1.4	1.4	1.4	1.6	1.6	1.7	1.5	1.3	1.3
Minimum Accepted Values	1.5			1.0			1.2			1.0		

Table 10: Factor of Safety obtained in Stability Analysis – Proposed Site Profile

Cross Section	Factor of Safety											
	Static, current conditions			Seismic (NZGS)			Seismic (ACCOP)			Static, immediately after shaking		
	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)	Circular	Non-Circular (Path)	Non-Circular (Block)
A – A'	1.8	1.7	1.6	1.5	1.5	1.6	1.8	1.7	1.8	1.4	1.4	1.5
B – B'	1.8	1.6	1.6	1.3	1.2	1.4	1.6	1.4	1.6	1.4	1.3	1.3
C – C'	1.7	1.5	1.5	1.2	1.2	1.3	1.4	1.4	1.5	1.3	1.3	1.2
Minimum Accepted Values	1.5			1.0			1.2			1.0		

The slope stability results generally indicate that the bund is presently safe and stable in its current state, and would remain so during a seismic event. The results also suggest that the bund and land behind it can be constructed and will remain stable (from a global stability perspective), for all scenarios modelled.

13.2 Consolidation Settlement

Considering the proposal to place bulk fill on site, we consider that the additional mass of fill could potentially induce additional consolidation settlement to occur over time. In order to assess the potential for consolidation settlement, we have undertaken a series of settlement analyses using the computer based software CPeT-IT by Geologismiki, and the results from CPT1, CPT 2 and CPT3. To assess the potential settlements that may arise from the proposed filling, we have modelled the additional of 1m of bulk fill, and also the addition of 1.7m of bulk fill (the approximate local maximum depth of fill to be placed).

The analyses do not provide an estimate of the time required for consolidation, however user defined input values of 6 months for primary consolidation, and 600 months for secondary consolidation have been selected for the purpose of the indicative results output. The results of our settlement analyses are summarised in Table 11 below, with the detailed results presented within Appendix 7.

Table 11: Static Settlement Analysis Results

Probe ID	1.0m Fill			1.7m Fill		
	Primary Settlement	Secondary Settlement	Total Settlement	Primary Settlement	Secondary Settlement	Total Settlement
CPT1	4 cm	1 cm	5 cm	6 cm	1.5 cm	6.5 cm
CPT2	4.5 cm	2 cm	6.5 cm	8 cm	2 cm	10 cm
CPT3	4.5 cm	2 cm	6.5 cm	8 cm	2 cm	10 cm

The results generally suggest that, where up to 1m of fill is to be placed, future settlements arising from consolidation of the underlying fill and alluvial materials may potentially be in the order of 50-65mm. Where fill depths will be thicker, a greater degree of settlement should be expected, with the results suggesting the maximum settlement in the location of the thickest filling could be in the order of 100mm.

We point out that the settlement estimates presented above are based on the results of the probe data from CPT1, CPT 2 and CPT3. In other parts of the site, it is possible that thinner or thicker horizons of compressible material may be present, which could result in significant variations to the estimates indicated in Table 11 above.

14. SITE FORMATION WORKS

14.1 General Earthworks

Aside from the three mixed-use buildings, the general earthworks across the site appears to be relatively minimal in terms of the depths of bulk cuts and fills.

Where excavated, the surficial lime stabilised horizon is expected to be suitable for re-use as bulk engineered elsewhere, where needed. However, where encountered, the non-stabilised reclaimed fill is expected to be overly wet of optimum upon excavation. Based on this, we recommend that provision should be allowed for conditioning of these materials, or that they be disposed of off site if they cannot be reused elsewhere.

Where any bund materials are encountered during the general earthworks, these materials should be observed and judged by a suitability qualified geotechnical professional who is also familiar with the site and the contents of this report. These observations should be undertaken in order to identify which materials could be re-used as bulk fill on site, and which materials should be undercut and how much would need to be undercut, and which materials may be able to be left in situ.

Where the site won materials are not considered suitable for re-use as engineered fill and a deficit of good quality fill material arises, any additional material required for filling should be imported from an off-site source. It is recommended that, unless the off-site source is an established and recognised aggregate quarry, all materials intended to be imported to site should first be observed by the geotechnical professional for the project, prior to importation to site, in order to determine their suitability for use as bulk fill.

Irrespective of the above, all earthworks undertaken must be carried out in general accordance with NZS4431:1989 and the Auckland Council Code of Practice for Land Development and Subdivision, Section 2, V1.6 dated September 2013.

14.2 Basement Excavation Retention

The PBA and ACL information provided indicates that the three mixed-use buildings will have a basement parking levels that are below the current ground level. The excavations to achieve these depths will likely remove the existing, stabilised crust of filled ground and exposed the un-stabilised, loose and saturated filled ground beneath.

Based on the existing site ground conditions, cut faces for deep excavations are unlikely to be stable at anything but very shallow batter gradients, which will not be feasible. As a result, temporary retention works are recommended to help ensure that the excavations can proceed to the respective target depths whilst maintaining the ground stability around the works. For such works, consideration should be given to the use of a combination of driven/vibrated steel tubes and sheet piles to aid with excavation stability, and to reduce the volume of material that will need to be removed.

Driven tubes and sheet piles would need to be subject to specific design for the ground conditions present, but it is anticipated that they would not achieve any significant embedment into the rock materials at depth, unless the rock is pre-drilled first. Further, the reclaimed fill and alluvium above the rock may not provide sufficient passive resistance to movement. As such, sheet piles may need additional restraint, such as a reinforced capping beam, be tied back to a second row of piles, or be propped off the other walls prior to excavation beginning. Regardless of the solution adopted, this should be part of the temporary retaining wall design.

In addition to the above, groundwater influx is anticipated to be problematic for basement excavations. The average depth to standing groundwater is considered to be at approximately RL1.0m, which in many parts of the site is also expected to be affected by tidal fluctuations. As a result, groundwater in-flux into the basement excavations for the mixed-use buildings should be expected, particularly with deeper excavation depths. Should groundwater influx not be appropriately controlled, it is considered likely to result in excess silt and sedimentation build-up.

14.3 Working Surface For Basement Foundation Construction

Once completed, the basement excavations are expected to expose the low strength fill and/or in situ alluvium at depth. From the results of our investigation, these materials are considered unlikely to be sufficient to support heavy construction plant in their current state.

A reasonable working surface for construction plant is likely to be provided at basement subgrade level where ground improvement methods are employed.

Where piled foundations are to be utilised for these buildings, and ground improvement is not undertaken, a working surface could be provided for construction plant through the use of a geotextile reinforced gravel pad that is constructed across the top of the exposed subgrade soils. Such a reinforced gravel pad would need to be subject to specific design for the construction plant that it is intended to carry, however in general we envisage that this would utilise a biaxial or triaxial polyester geogrid, a non-woven geotextile separator cloth, and approximately 800mm to 1000mm of compacted granular material.

15. RETAINING WALLS

15.1 Basement Retaining Walls

The recommendations provided below should be treated as preliminary only, and must be reviewed at the detailed design stage in order to confirm their validity.

At this time, no information has been provided as to the permanent retaining walls supporting the excavations for basements of the mixed-use buildings. However, we understand that the temporary works used to permit the basement excavations could potentially be retained as the permanent basement retaining walls.

In addition to retaining the excavations, making use of the walls for the temporary works could also help to provide a barrier to lateral groundwater influx by having the seams between the sheets and/or tubes welded during excavation.

Regardless of the above, it suggested that the preliminary design of the basement retaining walls, may be designed assuming the geotechnical parameters presented within Table 11 below.

Table 11: Geotechnical Parameters For Preliminary Basement Retaining Wall Design.

Parameter	Soils		Rocks	
	Lime Stabilised Fill	Non Stabilised Fill / Harbour Mud	Bund Armour & Bund Core Materials	Waitemata Group Rock
Average Depth Below Ground Level (m)	0.0 – 2.0	2.0 – 7.0	0.0 – 6.5m	> 7.1m (at bund) > 8.3m (apartment buildings)
SPT 'N'	n/a	0 ~ 5	10 ~ 20	> 50
Unit Weight, γ (kN/m^3)	16	15	20	20
Effective Shear Strength (c' = kPa, Ø' = deg)	c' = 5 Ø' = 30	c' = 5 Ø' = 20	n/a	n/a
Undrained shear strength (kPa)	60	35	n/a	n/a
Unconfined Compressive Strength (MPa)	n/a	n/a	1 - 2	5 - 7
Skin Friction (kPa)	- 20	- 15	- 35	+ 75
Elastic Modulus, E' (MPa)	10	8	100	300
Poisson Ratio, ν'	0.35	0.4	0.25	0.25
Coefficient of Earth Pressure at Rest, K_0	0.5	0.58	0.36	0.36
Coefficient of Active Earth Pressure, K_a	0.33	0.41	0.22	0.22

* = estimated.

In addition to the above parameters, all retaining wall surcharges (sloping ground, vehicles, buildings and property boundaries) must be designed in accordance the Auckland Council Practice Note AC2231, Retaining Walls, V5 (March 2019).

15.2 Perimeter Fill Retaining Wall

The ACL drawing set provided includes details of a proposed gabion basket retaining wall that will be constructed around much of the perimeter of the site in order to retain the proposed bulk filling works. For such a wall type, we recommend that bund armouring is removed to expose the top of the bund core materials, and the wall founded directly on top of said materials.

Prior to construction of the gabion basket retaining wall bedding materials, the exposed bund core should be compacted with suitable plant. Provided that this is carried out, for the purposes of preliminary design, the bund core materials may be assumed to offer an ultimate unfactored bearing capacity of 300kPa for the gabion basket retaining wall.

As the individual gabion basket units are often supplied in fixed dimensions, the bund core materials may need to be undercut further in order to achieve the final design height.

Given the likelihood of further consolidation settlement following construction of the gabion basket wall and bulk filling behind, it is recommended that the gabion basket wall be design so that it can accommodate some consolidation settlement. In this respect, the baskets themselves should be designed to be stacked in a staggered pattern.

In order to allow the bulk filling behind the gabion basket wall, an appropriate, heavy grade geotextile separator cloth and reinforcing geogrid must be draped over the inside face of the wall prior to construction of the bulk fill. Such measures will help facilitate the construction of the bulk fillings, as well as prevent fines migration from the bulk fill through the gabion basket wall, particularly during king tides and severe storm events.

16. PRELIMINARY FOUNDATION RECOMMENDATIONS

16.1 General Comments

The comments and design recommendations provided below should be treated as preliminary only, and must be reviewed at the detailed design stage in order to confirm their validity.

Considering the subsurface results and the size of the buildings proposed, in general it is recommended that the foundations are taken through the fill and alluvium to either found on, or within the in situ rock at depth. In this respect, options for the building foundations include, but are not limited to:

- Piled foundations, or;
- Reinforced concrete raft type foundations, provided that ground improvement has been undertaken first.

Irrespective of foundation option selected, the top of the Waitemata Group Rock was identified in our investigation as being present at circa 7.7m depth below present ground level, or at an average elevation of RL-4.3. It is pointed out that these figures are averages only, and that the rock surface itself does vary across the site, as demonstrated in Tables 12 and 13 below.

Table 12. Depth to Rock and Rock Levels Identified From Machine Boreholes.

Rotary Machine Borehole ID	Surface RL	Depth to Top of Waitemata Group Rock	Top of Waitemata Group Rock RL
MH1	3.43	7.00	-3.57
MH2	2.88	7.50	-4.62
MH3	3.26	8.40	-5.14
MH4	2.62	8.80	-6.18
MH5	3.79	9.50	-5.71
MH6	2.84	10.45	-7.61
MH7	4.16	6.25	-2.09
MH8	3.85	6.80	-2.95
MH9	3.29	6.30	-3.01
MH10	3.96	7.80	-3.84
MH12	4.76	7.10	-2.34
MH14	3.46	10.00	-6.54
MH15	2.98	8.10	-5.12

Note, all depths indicated are in metres Surface RL from Hampson & Associates survey data

Table 13. Depth to Rock and Rock Levels Identified From CPT Probe Points.

CPT Probe Point ID	Surface RL	Depth to Top of Waitemata Group Rock	Top of Waitemata Group Rock RL
CPT01	4.25	7.33	-3.08
CPT02	3.48	8.37	-4.89
CPT03	3.73	7.21	-3.48
CPT(MH09)	3.29	6.1	-2.81
CPT(MH11)	2.67	8.4	-5.73
CPT(MH13)	3.07	6.2	-3.13
CPT(MH15)	2.98	8.1	-5.12

Note, all depths indicated are in metres Surface RL from Hampson & Associates survey data

As highlighted in previous sections, the basement level of the three mixed-use buildings is likely to be at, or near to the average depth to standing groundwater level, which itself is influenced by tidal variations, and hence groundwater levels may rise and fall on a daily basis. In this sense, where the basement walls are designed to resist hydrostatic forces, we point out that the basement floors should also be sufficiently waterproofed, and designed to resist uplift forces generated from potentially tidal groundwater pressures.

Where reinforced concrete foundations are proposed to be used, the foundations must be design to take into account the present of saline groundwater conditions, the level of which will fluctuate daily.

16.2 Piled Foundations

Piled foundations may comprise, but should not be considered to be limited to, reinforced concrete piles cast in pre-drilled pile holes. With such foundations, however, temporary excavation stability is likely to be problematic, particularly at depth within the low strength fill and in situ harbour mud alluvium below the average depth to standing groundwater. Where bored piles are specified, provision is recommended for casing all pile holes, or, alternative construction methods should be employed to help ensure pile excavation stability during construction.

The geotechnical parameters presented within Table 11 presented within Section 15.1 above may be assumed in the preliminary design of piled foundations.

16.3 Raft Slabs on Improved Ground

Where a ground improvement strategy, incorporating either stone mixed columns, Rammed Aggregate Piers (RAPs), or similar, has been undertaken, it may be possible to design much of the basement level foundations as raft slabs, with only key load points pile supported where vertical loads need to be transferred to the rock at depth, and/or where building tension and uplift restraint is required.

16.4 Discussion

Of the foundation options suggested above, at this time we understand that a ground improvement strategy, comprising either stone mixed columns or rammed aggregate piers, is currently the preferred option. However, considering the low strength nature of the site soils, bulging of any ground improvement techniques into the surrounding soils is a concern for this site, and must be taken into consideration during the specific design.

In order to provide additional strength to the ground improvement undertaken and help to limit any bulging, it is suggested at this time that any ground improvement will also likely need to be cement stabilised. Irrespective, the different ground behaviour between the piled sections of any structures, and the areas where ground improvement has been undertaken will need to be taken into consideration in the final decision for the structural design of the building foundations.

17. BOARDWALK

In general, we understand that the perimeter boardwalk will largely be constructed on top of the gabion basket retaining wall and bulk filling, but that there will also be several timber viewing platform decks that will extend out over the existing revetments and will comprise piled foundations.

Given that the gabion basket retaining wall will be founded solely on the existing revetments, in order to reduce the likelihood differential settlement between the viewing platform decks and the gabion basket wall, the piled foundations for the viewing platform decks may be founded within the bund core materials.

For preliminary design purposes, the geotechnical parameters presented within Table 11 presented within Section 15.1 above may be assumed in the preliminary design of the piled foundations for the timber viewing platforms decks.

18. DEWATERING POTENTIAL

We note that the proposed basement levels for the three mixed use buildings will be located near to average depth to standing groundwater, which is considered to be at approximately RL1.0. However, given that the groundwater level cross the site is considered to be affected by tidal fluctuations, we do not consider dewatering to be a significant issue, as the standing groundwater level beneath the site is effectively recharged approximately twice daily.

In addition, the perimeter sheet piling that may be used to permit the basement excavations for the buildings will also help to limit the amount of groundwater ingress and any drawdown.

Further, as there are there are no significant neighbouring buildings nearby, we consider that groundwater dewatering effects on neighbouring properties and structures will be negligible. As a result, groundwater dewatering has not been considered further.

19. FURTHER GEOTECHNICAL INVESTIGATION

This document has been prepared for Resource Consenting purposes only. The design parameters and recommendations given within Section 15, Section 16 and Section 17 must be treated as provisional only, and have been given to allow preliminary designs to be started, but not finalised.

In this respect, the information presented herein is not considered sufficient to support the detailed design and Building Consent applications for the development, and therefore further geotechnical investigation, analysis and design will be required at a later date in order to support that phase of the development.

Further investigation is also highly recommended in the proposed location of the pump station within the neighbouring property to the east. This is because we currently do not have any knowledge of how the original reclamation to the east was undertaken, and therefore what subsurface materials may be encountered in that location. Further investigation is therefore required in order to identify the ground conditions present and any potential constraints to the construction of the pump station and the immediately associated infrastructure.

20. GEOTECHNICAL RISK ASSESSMENT

Based on our investigation and analyses, Table 14 below presents a matrix of the perceived geotechnical risks to the project and suggested controls for each hazard.

Table 14. Geotechnical Risk Assessment Matrix.

Identified Hazard	Risk	Risk Score (native)	Suggested Minimum Hazard Controls	Risk Score (w+ Controls)
Groundwater	Groundwater Draw-down	N	No control needed; groundwater was found to have a tidal influence. There are also no nearby neighbouring structures which could be affected.	N
	Excavation collapse due to influx of water	H	Excavations should make use of temporary retaining to help maintain stability. Employ pumps to remove excess water that flows into the excavations.	L
Global Stability	Landslide / Global slope instability	N	No control needed; site is gently sloping and is currently safe and stable. Our analyses show that the proposed works will affect the state of stability, but that the returned results are all within Council accepted results for slope stability.	N
	Settlement	M - H	Provided all future buildings are pile supported, with piles embedded into the in situ Waitemata Group rock materials at depth, the buildings themselves will not be affected by on-going consolidation settlement. The bulk filling, roadways and perimeter gabion basket retaining wall, however, will all experience some degree on consolidation settlement in the years following the completion of construction.	M
Seismic Hazard	Liquefaction / lateral spreading / cyclic softening	M	Our analyses suggest that while the site materials are not prone to catastrophic liquefaction, there remains a high potential for cyclical softening of the site soils during a seismic event, which could result in a significant loss of strength immediately after the cessation of shaking. Providing all buildings are pile supported and design accordingly, cyclical softening is not expected to pose a significant risk to the future buildings. The bulk filling and external infrastructure is, however, likely to be affected as a result of cyclical softening.	N – M

Note: N = None L = Low M = Moderate H = High

21. CONCLUSIONS

Based on the results of our investigation, and provided that our recommendations above are adopted in the detailed, engineering design for the development, along with additional further detailed geotechnical analysis, we consider that the development is feasible from a geotechnical perspective. Our investigation has not identified any geotechnical constraints that are insurmountable on this site, and see no specific geotechnical condition, either pre-existing or that would be created through the site formation works, that would otherwise prevent or severely restrict this development from proceeding.

22. LIMITATIONS

The conclusions made in this report are based upon the results of our desk study, the data obtained from the subsurface investigation points, which were spaced about the site as appeared appropriate for the proposed development with respect to the existing site constraints, and the analyses and interpretations that were undertaken with the information resulting from our investigation.

We point out that, while the subsurface investigation points were extended as deep as reasonably possible, only limited information was obtained from the underlying slightly weathered materials. Whilst our investigation did not identify any deep-seated bedding plane defect, or any other adverse lithological feature within the underlying parent rock, it cannot be said that any such features do not exist, and that should any such features exist, that they do not present a hazard, even remotely, to the greater area surrounding the site.

This report was prepared in the context defined in Section 1 above for the sole benefit of our Client, and must not be relied upon by any other party, other than that for whom it was prepared, and the relevant Territorial Authority. It has been compiled with respect to the brief given to us, and must not be relied upon in any other context or recreated for any other purpose.

The recommendations given in this report are provided as an overall strategy to minimise risks from geotechnical hazards. It should be noted that they are unlikely to remain effective if they are adopted in a piecemeal manner.

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