REPORT TO

HORTON COASTAL ENGINEERING PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

NEWPORT SURF LIFE SAVING CLUB
394 BARRENJOEY ROAD, NEWPORT, NSW

Date: 19 October 2021
Ref: 32537RErpt Rev2

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Table of Contents

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations and additions at the Newport Surf Life Saving Club (SLSC), 394 Barrenjoey Road, Newport, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Peter Horton of Horton Coastal Engineering Pty Ltd (HCEPL) in an email dated 8 July 2019. The commission was on the basis of our fee proposal (Ref. P49373R) dated 18 April 2019.

We have been provided with the following relevant information:

- Architectural Plans (Project Ref. NSC, Drawing Nos. 000^{A} to 019^{A}, all dated 25 August 2021) prepared by Adriano Pupilli Architects (APA).
- 'Initial Coastal Engineering Advice on Newport SLSC Redevelopment' report (Ref. lrJ0153, dated 14 August 2018) prepared by HCEPL.
- 'Assessment of Options for Redevelopment of Newport SLSC, with Updated Consideration of Risk from Coastal Erosion/Recession' report (Ref. rpJ0153, dated 17 February 2020) prepared by HCEPL.
- 'Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment' (Ref. rpJ0153, dated 9 November 2020) prepared by HCEPL.
- 'Structural Feasibility Report' (Ref. 2018S0141.00-ps-report, dated 5 June 2018) prepared by Partridge Structural Pty Ltd (Partridge).
- Site survey plan (Ref. 17692detail, Sheet Nos. 1 to 5 17692detail, Sheets 1 to 5, Issue 1, dated 13 April 2018) prepared by CMS Surveyors Pty Ltd (CMS).

Based on a review of the provided information, we understand that following partial demolition, the proposed alterations and additions will include:

- A new one and two storey 'L' shaped extension constructed over the footprint of the demolished northern portion of the SLSC and also extending to the west, and will include a new first floor terrace. The finished floor Reduced Level (RL) of the ground floor level (GFL) will be formed at RL5.66mAHD and RL5.79mAHD within the proposed boat shed. We have assumed excavation to a maximum depth of approximately 0.5m will be required over the north-western corner of the extension to achieve design surface levels. Elsewhere, unless the ground floor slab is suspended, ground levels will need to be raised approximately 0.17m to achieve the proposed GFL.
- Construction of a lift in the central portion of the existing SLSC building. APA have advised bulk excavations to a maximum depth of approximately 0.5m will be required to form the proposed lift pit.
- Extension of the male amenities room in the south-western corner of the SLSC building approximately 1.2m to the north.
- Internal alterations and additions throughout the ground and first floor of the SLSC. The modifications will include raising of ground floor levels within the central and southern portions of the SLSC to match the proposed GFL of RL5.66mAHD, the addition of a first floor balcony to the south of the existing deck area, and replacement of internal walls. From our discussions with APA the existing ground floor slabs to the south of the extension will be demolished and rebuilt.

We note that an additional geotechnical investigation was carried out in conjunction with the geotechnical investigation for the proposed SLSC development for a proposed buried piled seawall. The factual results of both investigations were provided to HCEPL in an email dated 12 February 2020. Council's preferred methodology to manage coastal erosion issues at the site is to construct a buried piled seawall on the seaward side of the existing SLSC building. The results of the additional geotechnical investigation will be used to prepare a separate report on the geotechnical aspects of the proposed seawall as part of the detailed design.

Structural loads for the SLSC alteration and additions have not been provided and typical loadings for this type of development have been assumed.

The purpose of the current investigation was to obtain geotechnical information on the subsurface conditions and to use these as a basis for providing comments and recommendations on site preparation, retention, footing design, earthworks, floor slabs and external pavements for the SLSC redevelopment

The geotechnical investigation was carried out in conjunction with a preliminary acid sulfate soil screening by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E23537BGlet-ASS, for the results of the acid sulfate soils screening.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 8 and 9 August 2019 and comprised the drilling of four boreholes (BH1 to BH4) to depths of 9.15m (BH3 and BH4) and 12.15m (BH1 and BH2) below existing surface levels using our track mounted JK308 drilling rig. The boreholes were auger drilled to depths of 4.65m (BH1), 3.2m (BH2) and 9.15m (BH3 and BH4) and the existing concrete paved surface at BH1 was initially diatube cored (with water flush). BH1 and BH2 were then extended by wash bore drilling techniques (with water flush) to their termination depth of 12.15m.

BH3 and BH4 will be used to assist with the design of the proposed buried piled seawall. However, the boreholes have been included within this report to provide additional information on the subsurface conditions relevant to the proposed SLSC redevelopment.

Prior to the commencement of fieldwork, the borehole locations were scanned for the presence of buried services by a specialist sub-contractor.

The borehole locations, as shown on the attached Figure 2, were completed at, or close to, the locations agreed upon between HCEPL and JKG prior to the commencement of the fieldwork and were set out using taped measurements from existing surface features. Figure 2 is based on aerial imagery sourced from 'Nearmap' with the outline of the proposed SLSC extension and lift pit superimposed. The coordinates and surface RL's of the borehole locations were measured using a Topcon GRS-1 differential surveying unit. The coordinates and levels were measured within an accuracy of 50mm and checked in relation to spot levels indicated on the provided survey plan. The survey datum is the Australian height Datum (AHD).

The relative density and strength of the marine and alluvial soils were assessed from the Standard Penetration Test (SPT) 'N' values and augmented by hand penetrometer readings on cohesive soil samples recovered in the SPT split-spoon sampler.

Groundwater observations were made in the boreholes during drilling. Reliable measurements of groundwater levels could not be obtained on completion of drilling BH3 and BH4 due to the introduction of water to facilitate wash boring. No longer term groundwater monitoring was carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The fieldwork for the investigation was carried out under the direction of our senior geotechnical engineer (Michael Egan) who set out the boreholes and directed the buried services scan. Our geotechnical engineer (Joanne Lagan) was present full-time on site, logged the encountered subsurface profile and nominated in-situ testing and sampling. The borehole logs (which include field test results and groundwater observations) are attached, together with a glossary of logging terms and symbols used.

No geotechnical laboratory testing was carried out as it was not deemed necessary. A contamination screen of site soils and groundwater was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on a section of the gently sloping South Pacific Ocean foreshore immediately to the west (landward) of Newport Beach and has a western frontage on to Barrenjoey Road. Surface levels over the landward reaches of the site sloped gently down to the south and east at between approximately 1° and 2°.

The existing one and two storey rendered SLSC building was surrounded by concrete paved pathways and garden areas which contained small to large sized bushes and trees. The western portion of the site comprised the south-eastern section of an asphaltic concrete (AC) surfaced carpark which extended beyond the site to the north. Two steel shipping containers were located beside the concrete kerb and gutter that lined the eastern edge of the carpark opposite the SLSC.

To the north of the site, the carpark was generally flanked to the east and west by grass covered and/or densely vegetated areas with large size Norfolk Pine trees scattered throughout. The densely covered dunes on the seaward side of the carpark, which also formed part of the northern site boundary, generally sloped down to the east at a maximum of approximately 10°. The grass covered 'Bert Payne' Park which also contained large trees and a children's playground were located beyond the site to the south and west, respectively.

Based on a cursory inspection from within the site, the existing SLSC building generally appeared in fair condition with localised cracking and step cracking in the order of approximately 5mm width observed. Concrete spalling of the SLSC facade was also noted in the Structural Feasibility Report prepared by Partridge. Subsidence and cracking (maximum approximately 50mm depth/width) of the AC surfaces and concrete paved pathways was recorded, together with localised potholing of the AC surfaces and minor disintegration of the concrete pathway surfaces.

3.2 Subsurface Conditions

Reference to the 1:100,000 geological map of Sydney indicates that the site is underlain by Quaternary age marine or estuarine sands located close to the interface with the silty, sandy and clayey alluvial and estuarine sediments to the west. The boreholes disclosed a generalised subsurface profile comprising marine sands (occasionally silty) to moderate depth overlying alluvial sands and clays. Groundwater seepage was recorded in the boreholes at moderate depth. For further details of the subsurface conditions at each borehole location, reference should be made to the attached borehole logs. A summary of the pertinent subsurface characteristics is presented below:

Paved Surface

Reinforced concrete and AC paved surfaces were encountered in BH1 and BH2 and were respectively 95mm and 30mm thick.

Fill

Fill comprising silty sand (BH1) and silty sandy gravel (BH2) was encountered below the pavements and extended to respective depths of 0.1m and 0.3m.

Marine Sands

Marine sands (occasionally silty) were encountered within the boreholes from beach surface level or the base of the fill and extended to the top of the alluvial deposits at depths of 5.5m (BH1 and BH3), 7.0m (BH2), and 5.85m (BH4). The relative density of the marine sands was typically either loose to medium dense or medium dense, with loose and dense bands encountered in BH2 (1.5m thick) and BH4 (1m thick) at respective depths of 5.5m and 2.5m. The sands contained various portions of shell fragments with fine to coarse grained ironstone gravel encountered in the loose and dense sand bands.

Alluvial Sediments

Alluvial sands and clays were encountered beneath the marine sands in all boreholes. The silty sandy clays, silty clays and sandy clays were assessed to be of low to high plasticity and of stiff to very stiff strength. The relative density of the sands, silty sands and clayey sands was either loose or medium dense. In BH1, a sandy clay/clayey sand band (1.5m thick) of firm to stiff strength and very loose to loose relativity density was encountered at 10.0m depth. In all boreholes except BH4, the clays and sands were interbedded. The alluvial sands contained traces of fine to coarse grained quartz and sandstone gravels, and shell fragments.

Groundwater

Groundwater seepage was encountered whilst auger drilling the boreholes at approximate depths of 3.8m (BH1), 3.2m (BH2), 4.2m (BH3), and 3.5m (BH4). These depths correspond to approximately RL1.5mAHD (BH1), RL2.6mAHD (BH2), RL0.1mAHD (BH3) and RL0.6mAHD (BH4).

No longer term groundwater monitoring was carried out.

4 COMMENTS AND RECOMMENDATIONS

The comments and recommendations which follow assume the proposed buried piled seawall will be constructed and that coastal erosion issues will be appropriately managed by the new seawall. If this is not the case, then the comments and recommendations will need to be reviewed and amended accordingly with due regard to coastal engineering issues.

4.1 Site Preparation

4.1.1 Dilapidation Surveys

Prior to demolition and earthworks commencing, consideration should be given to preparing a detailed dilapidation report on the portions of the SLSC building that will remain. Council may also require that dilapidation survey reports be completed on their assets to the west (i.e. carpark, paved surfaces etc.). The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the client in defending themselves from unfair damage claims and/or assessing damage caused by the contractor.

4.1.2 Demolition and Excavation

The excavation recommendations provided below should be complemented by reference to the NSW Government "Code of Practice Excavation Work" dated January 2020.

The footprint of the proposed SLSC extension and lift pit are indicated on the attached Figure 2. Bulk excavations to a maximum depth of 0.6m will be required to achieve design surface levels over the north-western corner of the proposed SLSC extension and for the lift pit.

Demolition and excavation will require careful detailing and sequencing, particularly with regard to:

- Temporary propping of sections of the SLSC building that will remain (detailed by the structural engineer); and
- The timing of the SLSC development works and the construction of the proposed buried seawall. As the seawall will be providing coastal erosion protection for the existing and redeveloped SLSC building, we assume that the seawall will be constructed either before or contemporaneously with the SLSC redevelopment.

This work will need to be completed using suitably experienced (and insured) contractors.

Prior to earthworks commencing, the footing details for the existing SLSC should be confirmed by review of available 'as-built' structural drawings, if available. The purpose of the review is to confirm the depth and geometry of the existing footings and whether any strengthening and/or underpinning of the footings is required. If the drawings are not available, then during demolition several test pits should be excavated to expose the existing footings to determine the foundation materials and their depth and geometry. This will be of particular importance for existing footings that will be supporting additional loads, are situated close to proposed excavations and/or in close proximity to any excavations associated with the proposed buried seawall. The test pits must be inspected by the geotechnical and structural engineers and include Dynamic Cone Penetration (DCP) testing to confirm the allowable bearing pressure of the foundation sands. The existing footings will be founded in sand and the test pits must not extend significantly beyond the base of the footings. Based on the results of the test pit inspections, the details of the underpinning can be confirmed by the structural engineer, in particular the panel lengths and the sequencing of their excavation (e.g. 'hit 1 miss 2'). Underpins that will be supporting the soil profile will need to be designed in accordance with the advice provided in Section 4.2.2 below, to resist lateral loading.

Due to the presence of loose sands and possibly very loose sands not disclosed by the investigation, we do not recommend the use of rock breakers during demolition due to the potential for transmission of vibrations which could cause damage to the adjacent sections of the SLSC that we understand are supported on high level footings founded in the sands. We recommend that the removal of all concrete floor slabs, footings, paved surfaces and any sections of internal/external walls be completed using a diamond saw followed by removal of the concrete pieces using a bucket attachment to the tracked excavator. Where access is restricted, a hand held demolition saw may be required. When using the saws, the resulting dust should be suppressed by spraying with water. We also note that 'dropping' of large sections of existing structures during demolition should also be avoided in order to prevent the generation of potentially damaging vibrations.

Following removal of the structures mentioned above, any proposed excavations to achieve design surface levels and/or formation of footings will encounter the soil profile. We expect the excavations to be readily completed using bucket attachments to tracked excavators. In addition, due to the presence of loose sands and possibly very loose sands not disclosed by the investigation, we advise that sudden stop/start movements of any tracked equipment should be avoided to reduce transmission of ground vibrations to adjacent sections of the SLSC building.

4.1.3 Seepage

Groundwater seepage was encountered in the landward boreholes at respective depths of 3.8m (BH1) and 3.2m (BH2). On this basis, we do not expect groundwater will be encountered within the relatively shallow excavations. However, some ephemeral seepage inflows may be encountered, particularly after periods of heavy rain. In general, we expect that inflows, if any, to be very small and managed by conventional sump and pump techniques and/or infiltration into the sandy subgrade.

Inspection and monitoring of groundwater seepage during excavation is recommended, so that any unexpected conditions which may be revealed can be incorporated into the drainage design. The site foreman should also monitor tidal levels when carrying out the works.

4.2 Temporary Excavation Support and Retention

4.2.1 Temporary Batters

Temporary batter slopes through the sandy profile no steeper than 1 Vertical (V) in 1.5 Horizontal (H) are considered to be appropriate, though some surficial instability could still occur. Stockpiles of construction materials, excavated materials etc should be kept well clear of the batter crests to avoid surcharging the slopes. These temporary batters are expected to be achievable provided existing footings are founded and/or underpinned below the temporary batter slopes.

Some instability of temporary sand batters may occur at, or below, the level of any groundwater seepage, especially after rain periods, and sand bagging may be required to stabilise the lower portion of these batters.

With regard to the lift pit, if underpinning of adjacent existing footings is not preferred then the sands will need to be supported by an engineer designed retention system installed prior to the start of excavation. A suitable shoring system would be a contiguous bored pile wall formed using hand auger techniques with temporary liners to support the potentially collapsible sands. The shoring piles would need to extend to sufficient depth below the base of the proposed lift pit excavation to satisfy stability considerations. Sealing of 'gaps' between the piles would also be required to prevent loss of sands between piles.

4.2.2 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of piled shoring walls, lift pit walls, conventional retaining walls, underpins supporting a soil profile, or landscape walls:

- For design of conventional walls that will be temporarily propped, backfilled and permanently supported by the structure, and any underpins supporting a soil profile, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient, K_o, of 0.5 for the retained profile, assuming a horizontal backfill surface.
- For temporary cantilever piled walls and where some minor movements of retaining walls may be tolerated (e.g. landscape walls), these may be designed using a triangular lateral earth pressure distribution and a coefficient of 'active' earth pressure, K_a, of 0.35 for the soil profile, assuming a horizontal backfill surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the retained profile.
- Any surcharge affecting the walls (e.g. nearby footings, compaction stresses, construction loads etc.) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven

geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system or foreshore area.

- Piled walls and any underpins supporting a soil profile must be designed as permanently drained and PVC pipes should be installed at nominal 1.2m horizontal spacings just above the adjacent floor level. The end of the pipe penetrating the retained soils must be wrapped in a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The pipes should discharge into the stormwater system or foreshore area.
- The lift pit walls should be designed to withstand full hydrostatic pressures (i.e. tanked) with a design groundwater level equivalent to the surrounding surface level. Alternatively, the potential groundwater pressures may be alleviated by providing the lift pit with drainage and a pump-out system.
- Lateral restraint of the retaining walls and any underpins founded in the soil profile below adjacent surface levels may be provided by the passive pressure of the soil below these levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, using a triangular earth pressure distribution and provided a Factor of Safety of at least 2 is used in order to reduce the high deflections that are associated with achieving a full passive case. Localised excavations in front of the walls e.g. for buried services etc. must also be taken into account in the design.

4.3 Footing Design

Based on the Structural Feasibility Report prepared by Partridge, we understand the performance of the existing footings supporting the SLSC, which are anticipated to be shallow strip footings, is adequate. On this basis, Partridge have indicated in their report that it is structurally preferred for all new footings to match the existing footing details and be founded on similar foundation materials. We also note that HCEPL have indicated a maximum landward groundwater level of RL3.5mAHD adjacent to the proposed buried seawall.

The proposed buried seawall will include a piled wall (formed using auger grout injected [CFA] techniques) which may be constructed at the same time as the proposed alterations and additions to the SLSC. If this is the case then pile footings for the alterations and additions could be installed during the piling works for the seawall. Provided the SLSC pile footings were founded below the design scour level, there would be an opportunity to reduce the northern extent of the proposed seawall. A reduction in seawall length may provide budget savings that exceed the additional costs associated with using pile footings to support the SLSC additions rather than high level footings.

Based on the investigation results, it is likely that loose and/or medium dense marine sands will be exposed over the design surface level of the proposed SLSC extension and lift pit excavation. We anticipate the existing strip footings supporting the SLSC to the south of the proposed extension would also be founded in loose (or denser) sands. As described in Section 4.1.2 above, the DCP testing of the sandy soils in test pits exposing existing footings will be required to confirm the allowable bearing pressure and similar testing of the foundation sands at the proposed footing locations will also need to be completed by the geotechnical engineer.

We recommend that all strip and/or pad footings, and any underpins, be founded in the loose (or better) relative density sands.

Strip or pad footings founded in loose (or denser) natural sands (width and embedment depth at least 0.5m) may be designed using an allowable bearing pressure of 100kPa. Predicted total settlements would be in the order of approximately 5mm for these footings.

If the additional geotechnical investigation completed during demolition indicates the presence of very loose sands and/or poorly compacted fill extending to depths in excess of say 1m, then bored piles installed using a pendulum auger attachment to the tracked excavator (or hand auger piles) with temporary liners to support the collapsible soils may be used. Assuming an embedment of 4D (where 'D' is the pile diameter) into loose (or denser) sands below the very loose sands or poorly compacted fill, an allowable end bearing pressure of 150kPa (0.3m diameter) may be assumed for design. Predicted total settlements would be expected to be less than 5mm for piled footings (either bored piers as outlined or deeper CFA piles founded below the design scour level).

We note that the differential settlements between the existing building and new addition would be equivalent to the predicted settlements. However, the settlements would be of an elastic nature, i.e. occur as the building loads are imposed. The design of the proposed additions should be checked with regard to the predicted differential settlements by the structural engineer. If it is assessed that some cracking could occur and which is considered to be unacceptable, then measures such as construction joints will need to be considered. Piling the new addition may also assist in limiting differential settlements, if required.

DCP testing must be carried out by an experienced geotechnical engineer from the base of a representative number of proposed high level footing excavations and at the locations of any bored piles to confirm the allowable bearing pressure.

Excavations for pad or strip footings that extend through sands should be supported with formwork, as vertical cuts will be potentially unstable. The bases of footing excavations in sands should be thoroughly moistened and compacted using a hand held plate compactor (e.g. whacker packer). Qualitative vibration monitoring should be carried out by site personnel to assess vibration levels on nearby buildings. If vibrations are considered to be excessive, then compaction should cease and further advice should be sought.

We recommend the trafficking of any high level footing excavations in sands be kept to a minimum, preferably not at all, and that they be protected by a blinding layer of concrete immediately after geotechnical inspection and DCP testing.

CFA piles (if used) would need to be certified by the piling contractor.

4.4 Floor Slabs

We assume that the new floor slabs will be suspended between footings founded in the marine sands, as is the case for the existing floor slabs that will remain. Whilst slab-on-grade construction is considered feasible, it will be difficult to complete high quality earthworks in accordance with the advice provided in Section 4.5 below on such a small area. It may also require removing and re-compacting any existing fill that may be present that was not disclosed by the investigation. In addition, we note that any on-grade floor slab would also be in contact with a mix of potential variable density natural sands, possibly existing fill and engineered fill locally required to raise site surface levels. This could result in differential deflections, and potential cracking of the floor slabs. All deflections of the new slabs would be differential with respect to the existing sections of the building.

Our recommendation is to suspend the floor slab between footings founded in the marine sands. For floor slabs suspended over soil subgrade areas, the subgrade preparation would comprise the removal of any topsoil and/or any soil containing organics, completion of excavation (where required) and the nominal tracking of 'formwork fill' to the required subgrade level.

Any proposed concrete floor slabs, unless suspended, should be separated from all walls, footings etc (i.e. designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys.

4.5 External Paved Areas and Earthworks

For any proposed external paved areas, slab-on-grade construction is considered feasible, although we reiterate that it will be difficult to complete high quality earthworks over such small site areas. Even if the earthworks are completed in accordance with the following recommendations, it is possible that there will be a significant variation in subgrade conditions, resulting in some degree of differential deflection, and potential cracking of the external paved areas. If this cannot be accepted, the external paved areas should also be suspended from footings founded in loose (or denser) sands.

The following earthworks recommendations should be complemented by reference to AS3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments"

4.5.1 Subgrade Preparation

Prior to construction of any on-grade floor slabs, external paved areas, or placing fill to raise site surface levels, the soil subgrade should be prepared as follows:

- Following removal of existing floor slabs and pavements, and stripping of all topsoil and/or root affected soils, any remaining poorly compacted fill should be removed, as directed by the geotechnical engineer
- Proof roll the soil subgrade with a minimum 2 tonne dead weight smooth drum roller, using the static (non-vibration) mode, following thorough moistening of the sand subgrade. Where access may be restricted, proof rolling may require the use of a hand held plate compactor (e.g. whacker packer)
- To assist with proof rolling, we recommend that a thin layer of road base (75mm thick) be placed over the sand subgrade to improve near surface compaction and prevent shearing during rolling.

- Care should also be taken when using vibrating equipment not to cause damage to adjacent structures. The vibrations should be qualitatively monitored by site personnel and if there is any cause for concern, then proof rolling should cease and further advice sought.

Stripped topsoil and root affected soils should be separately stockpiled for re-use in landscape areas, as such soils are not suitable for re-use as engineered fill. Any deleterious or contaminated existing fill encountered during stripping should be disposed of offsite.

4.5.2 Engineered Fill

For treatment of poor subgrade areas to replace poorly compacted fill and/or locally raise site surface levels, engineered fill should be used.

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size of 40mm. We expect the excavated sands may be used as engineered fill. Engineered fill comprising sands should be placed in layers not exceeding 100mm loose thickness and compacted with the above mentioned rollers to achieve a minimum density index (I_D) of 70%. However, the I_D may be reduced to 65% in soft landscaped areas, where settlement can be tolerated. Engineered fill may also comprise imported crushed or ripped sandstone which should also be compacted using the above mentioned roller in layers not exceeding 100mm loose thickness to a density between 98% and 102% of Standard Maximum Dry Density (SMDD), and within 2% of their Standard Optimum Moisture Content (SOMC). The density may be reduced to 95% of SMDD in soft landscaped areas, where the designer considers that settlements are not critical. Fill comprising the excavated sandstone (suitably crushed) would provide a more stable subgrade. We note that sandy soils are difficult to compact, particularly silty sand which is usually very moisture sensitive. Care must be taken with the compaction with no vibrations being used.

Backfill to conventional retaining walls should also comprise engineered fill comprising the excavated sands. Alternatively, well graded imported granular materials such as demolition rubble would be suitable for this purpose provided it is also free of deleterious substances and has a maximum particle size of 40mm. Imported well graded granular fill should be compacted as described above. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798 (Table 8.1) for the volume of fill involved. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

As an alternative, single sized granular material (or 'no fines' gravel) may be used as backfill to retaining walls and this would also act as the drainage behind the wall and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

4.6 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Dilapidation survey reports.
- Inspection of test pits exposing existing footings and DCP testing of the foundation sands.
- Monitoring of groundwater seepage into excavations.
- Inspection and DCP testing of a representative number of footings.
- Proof rolling of exposed soil subgrade.
- Density testing of engineered fill.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between and below the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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BOREHOLE LOG

Borehole No. 1

1/2

Client: HORTON COASTAL ENGINEERING

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER/WASHBORE **R.L. Surface:** ≈ 5.3m

Date: 8/8/19 **Datum:** AHD

Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			CONCRETE: 95mm.t.				6mm DIA. REINFORCEMENT 60mm TOP COVER
					-	FILL: Silty sand, fine to medium grained, dark brown.	M			
		N = 10 4,4,6			SP	SAND: fine to medium grained, light brown.	M	L-MD		MARINE
			1							
		N = 13 4,5,8						MD		
			2							
						SAND: fine to medium grained, light orange brown, with shell fragments.				
		N = 14 5,7,7	3							
					SM	Silty SAND: fine to medium grained, dark grey and grey.				
						as above, but light orange brown, with shell fragments.				
▶							W			
			4							
		N = 27 13,16,11								COMMENCE WASHBORING
			5							
					CL	Silty sandy CLAY: low plasticity, light grey, fine to medium grained sand.	w>PL	St-VSt		ALLUVIAL
		N = 8 2,4,4	6						290 200 310	
			7							

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BOREHOLE LOG

Borehole No.
1
2/2

Client: HORTON COASTAL ENGINEERING
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER/WASHBORE **R.L. Surface:** ≈ 5.3m
Date: 8/8/19 **Datum:** AHD
Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		N = 10 3,4,6			CI	Silty sandy CLAY: medium plasticity, light grey, fine to medium grained sand.	w>PL	VSt	380 290 360	ALLUVIAL
			8							
		N = 17 7,5,12	9		SC	Clayey SAND: fine to medium grained, light grey.	W	MD		
		N = 4 3,2,2	10		CL/SC	Sandy CLAY/Clayey SAND: low plasticity, fine to medium grained, light grey and grey.	w>PL/ W	F-St/ VL-L	60 80 100	
			11							
		N = 8 3,3,5	12		CI	Silty CLAY: medium plasticity, light grey and grey, with fine to coarse grained sand.	w>PL	St-VSt	190 380 210	
						END OF BOREHOLE AT 12.15m				
			13							
			14							

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BOREHOLE LOG

Borehole No.
2

Client: HORTON COASTAL ENGINEERING
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER/WASHBORE **R.L. Surface:** ≈ 5.8m
Date: 9/8/19 **Datum:** AHD
Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		-	ASPHALTIC CONCRETE: 30mm.t.	D			
					SP	FILL: Silty sandy gravel, fine to coarse grained igneous, grey, fine to coarse grained sand.	M	MD		MARINE
		N = 13 5,6,7				SAND: fine to medium grained, light orange brown, trace of shell fragments.				
			1							
		N = 10 5,5,5						L-MD		
			2							
		N = 10 3,4,6	3							
▶										COMMENCE WASHBORING
			4			SAND: fine to medium grained, light orange brown, with shell fragments.	W	MD		
		N = 26 6,14,12								
			5							
						as above, but with fine to coarse grained ironstone gravel.		L		
		N = 8 9,6,2	6							
			7							

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BOREHOLE LOG

Borehole No. **2**

Client: HORTON COASTAL ENGINEERING

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER/WASHBORE **R.L. Surface:** ≈ 5.8m

Date: 9/8/19 **Datum:** AHD

Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SC	Clayey SAND: fine to medium grained, light grey.	W	L		ALLUVIAL
	ES, DS	N = 9 4,5,4								
			8							
					CH	Silty CLAY: high plasticity, grey, trace of fine to medium grained sand.	w>PL	VSt		
	ES, DS	N = 14 3,5,9	9						270 370 280	
					SC	Clayey SAND: fine to medium grained, light grey and grey.	W	L		
			10							
	ES, DS	N = 9 8,4,5								
			11							
	ES, DS	N = 6 1,3,3	12							
						END OF BOREHOLE AT 12.15m				
			13							
			14							

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BOREHOLE LOG

Borehole No.
3

Client: HORTON COASTAL ENGINEERING
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 4.3m
Date: 8/8/19 **Datum:** AHD
Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		SP	SAND: fine to medium grained, orange brown, trace of shell fragments.	M	L-MD		MARINE
		N = 13 4,5,8								
			1							
		N = 9 4,4,5								
			2							
		N = 10 4,5,5	3							
			4							
▶		N = 20 5,8,12				SAND: fine to medium grained, light orange brown, with shell fragments.	W	MD		
			5							
					SC	Clayey SAND: fine to medium grained, grey brown and red brown.	W	L		ALLUVIAL
		N = 7 2,2,5	6							
			7		CI	Silty sandy CLAY: as below.	w>PL	VSt		

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BOREHOLE LOG

Borehole No. 3

Client: HORTON COASTAL ENGINEERING

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 4.3m

Date: 8/8/19 **Datum:** AHD

Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		N = 10 4,4,6	8		CI	Silty sandy CLAY: medium plasticity, grey, fine to medium grained sand.	w>PL	VSt	240 220 220	ALLUVIAL
		N = 26 4,10,16	9		SP	SAND: fine to medium grained, light grey, trace of medium to coarse grained quartz and sandstone gravel, clay and silt fines.	W	MD		
			10 11 12 13 14			END OF BOREHOLE AT 9.15m				

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BOREHOLE LOG

Borehole No. 4

Client: HORTON COASTAL ENGINEERING

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 4.1m

Date: 9/8/19 **Datum:** AHD

Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0	SP	SAND: fine to medium grained, light brown.	M	MD		MARINE
	N = 12 4,6,6							
		1		as above, but light orange brown, trace of shell fragments.				
	N = 13 6,6,7							
		2						
				as above, but trace of medium to coarse grained ironstone gravel.		D		
	N = 35 7,16,19	3						
▶				SAND: fine to medium grained, light orange brown, with shell fragments.	W	MD		
		4						
	N = 13 10,8,5							
		5						
				as above, but trace of clay and silt fines.				
	N = 14 4,7,7	6	SM	Silty SAND: fine to coarse grained, brown and grey, with clay fines, trace of fine to medium grained quartz and sandstone gravel, and shell fragments.				ALLUVIAL
			SP	SAND: fine to medium grained, grey, with shell fragments, clay and silt fines.				
		7						

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BOREHOLE LOG

Borehole No.
4

Client: HORTON COASTAL ENGINEERING
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: NEWPORT SLSC, 394 BARRENJOEY ROAD, NEWPORT, NSW

Job No.: 32537RE **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 4.1m
Date: 9/8/19 **Datum:** AHD
Plant Type: JK308 **Logged/Checked by:** J.L./M.E.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■	N = 21 3,9,12			SP	SAND: fine to medium grained, grey, with shell fragments, clay and silt fines.	W	MD		ALLUVIAL
	■					8							
	■			■	N = 15 6,7,8	9							
									END OF BOREHOLE AT 9.15m				
						10							
						11							
						12							
						13							
						14							

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	NEWPORT SLSC, 394 BARRENJOEY ROAD NEWPORT,NSW
Report No: 32537RE	Figure: 1

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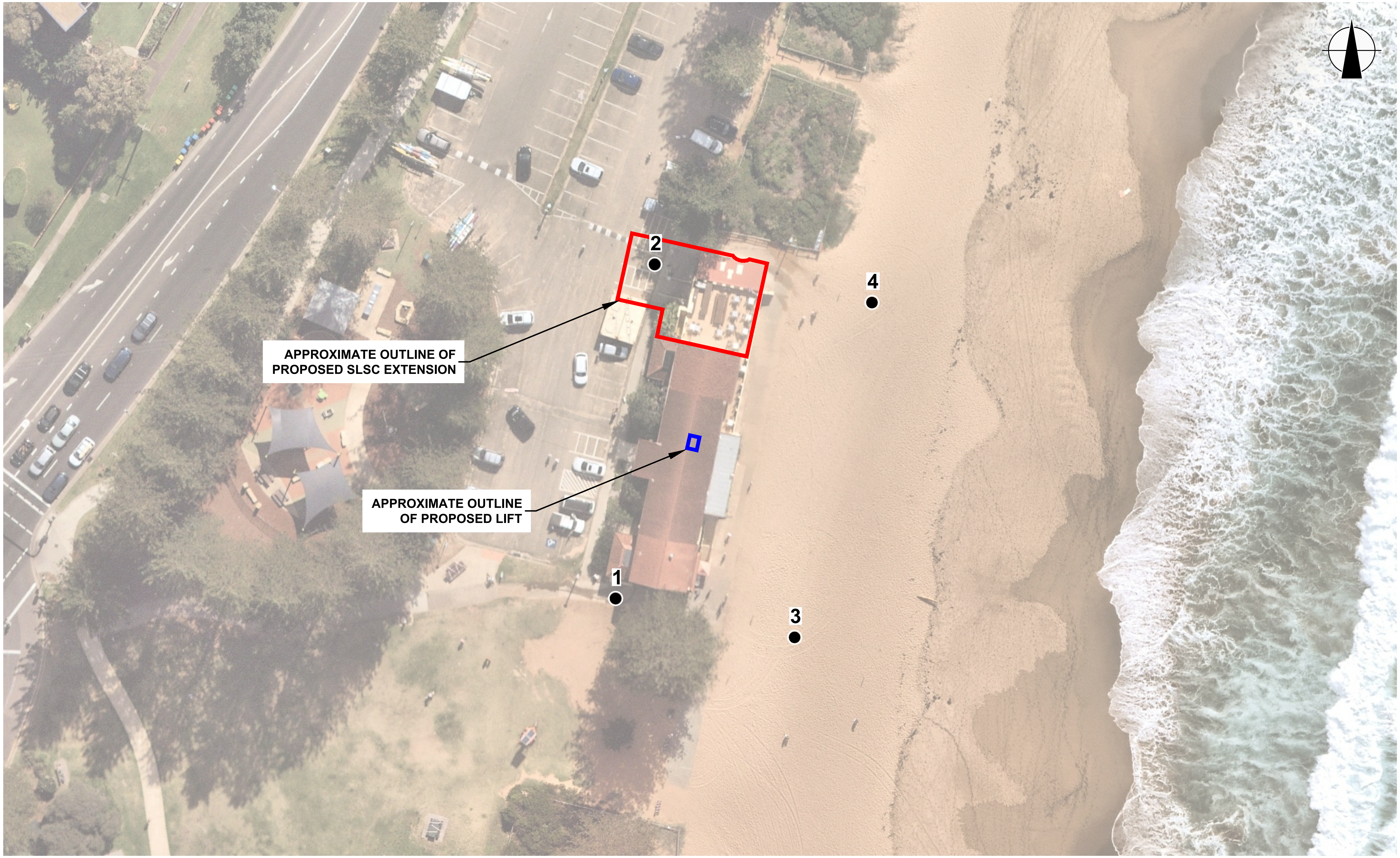
PLOT DATE: 21/08/2020 10:56:39 AM DWG FILE: Y:\32000'S\32537RE NEWPORT\CAD\32537RE.DWG



LEGEND

● BOREHOLE

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 5 10 15 20 25

SCALE 1:500 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**BOREHOLE LOCATION PLAN**
Location:	NEWPORT SLSC 394 BARRENJOEY ROAD, NEWPORT, NSW
Report No: 32537RE	Figure: 2

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PLOT DATE: 25/08/2020 9:36:55 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32537RE NEWPORT\CAD\32537RE.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

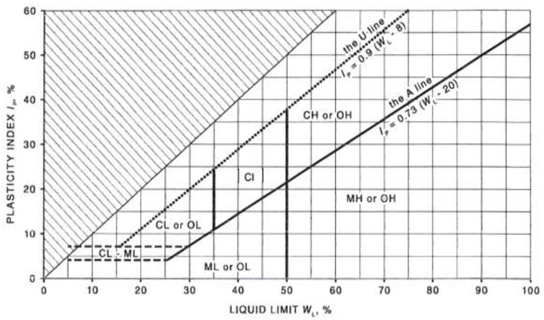
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE: ≤ 15 / 0 – 4
	L	LOOSE: > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE: > 35 and ≤ 65 / 10 – 30
	D	DENSE: > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE: > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as: RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock. EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock. ALLUVIAL – soil deposited by creeks and rivers. ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents. MARINE – soil deposited in a marine environment. AEOLIAN – soil carried and deposited by wind. COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits. LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength: Point Load Strength Index $I_{s(50)}$ (MPa)	Guide to Strength: Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres