



REPORT TO
TIME AND PLACE MANLY LAND PTY LTD

ON
GEOTECHNICAL ASSESSMENT

FOR
PROPOSED MIXED USE DEVELOPMENT

AT
35 TO 43 BELGRAVE STREET, MANLY, NSW

Date: 19 May 2023

Ref: 35999PErpt

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Figure 1: Site Location Plan

Report Explanation Notes

Appendix A: Boreholes Logs 1 & 2 From Our Previous Geotechnical Investigation Report (Ref. 4159) Dated 21 January 1986

1 INTRODUCTION

This report presents the results of a preliminary geotechnical assessment for the proposed mixed use development at 35 to 43 Belgrave Street, Manly, NSW. The location of the site is shown in Figure 1. The assessment was commissioned by Marcus Lewin of Time and Place Pty Ltd by return of a signed 'Acceptance of Proposal' form dated 27 April 2023. The commission was on the basis of our fee proposal, Ref. 'P58247PE' dated 28 February 2023.

Based on the supplied architectural drawings (Project Number. 6693, Drawing Numbers. DA-0103², DA-0111¹, DA-0112¹, DA-0113¹, DA-0114¹ and DA1501²) prepared by SJB Architects, we understand that following demolition, the proposed development will comprise the construction of a five storey building overlying a two level basement. To achieve the finished floor level of the proposed Basement Level 2 at RL0.0m, excavation to a maximum depth of approximately 6m below existing surface levels will be required. The basement will extend to the site boundaries.

The purpose of the assessment was to review available subsurface information from previous JK Geotechnics (JKG) investigations within the subject site and nearby properties, and carry out a walkover inspection of the site. Based on the information obtained, we present our preliminary comments and recommendations on the expected subsurface conditions, site preparation and excavation, retention, hydrogeological issues, footing design, the basement slab, earthquake design parameters, and additional geotechnical input.

This geotechnical assessment was carried out in conjunction with a preliminary site investigation (PSI) by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref. 'E35999BT rpt', for the results of the environmental site assessment.

2 ASSESSMENT PROCEDURE

The preliminary geotechnical assessment is based upon a detailed inspection of the topographic and geological conditions of the site and its immediate environs, correlated with the subsurface conditions encountered during our previous geotechnical investigation within the site.

The results of our previous investigation were presented in our report, Ref. '4159' dated 21 January 1986 (JKG 1986). The fieldwork comprised the drilling and testing of three boreholes (BH1 to BH3) to depths of 4.4m (BH2) and 5.95m (BH1 & BH3) below original surface levels using spiral augering techniques with a truck mounted Edson 3000 drill rig. The boreholes were located at the northern end of the site adjacent to Raglan Street and the borehole logs are presented in Appendix A.

3 SITE OBSERVATIONS

The site is located within relatively flat terrain beyond the toe of an east facing hillside. Manly Beach and Manly Cove are located approximately 200m to the east and 400m to the south-west of the site, respectively.

The site is 'L' shaped in plan being approximately 41m long (north to south) by 33m deep (east to west) and is bound to the north by Raglan Street, to the east by Whistler Street and to the west by Belgrave Street.

At the time of our assessment, the site was occupied by one, two and three storey brick and rendered buildings which generally extended to the site boundaries. Some small internal courtyards were also present within the site. Based on a cursory inspection, the buildings appeared in a relatively good condition with localised cracking and stepped cracking to approximately 5mm wide.

The neighbouring property to the south-east (No. 21 Whistler Street) was vacant and unoccupied, with vegetation and granular soils scattered across the site. Based on publicly available information sourced from the Northern Beaches Council website, a six storey residential building overlying a single basement level is proposed at the site, although no construction activities (i.e. plant, equipment, personnel etc.) were evident during our walkover inspection.

The neighbouring property to the south (No. 33 Belgrave Street) comprised a two and three storey rendered building abutting the common boundary. The building extended from Belgrave Street to Whistler Street and was also in a relatively good condition with localised stepped cracking noted on the northern wall. The neighbouring property did not appear to contain a basement structure.

The ground surface levels across the site boundaries appeared relatively level.

4 EXPECTED SUBSURFACE CONDITIONS

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Quaternary aged deposits comprising fine to medium grained 'marine' sand.

The geotechnical investigations within the subject site and surrounding area disclosed a generalised subsurface profile comprising a limited depth of fill over natural fine to medium grained sand. Groundwater was also encountered at moderate depths. A summary of the subsurface conditions encountered/expected within the site is provided below.

Fill

Fill comprising sand/silty sand was encountered from surface level in BH1 & BH2 and below a thin concrete slab in BH3, and was a maximum depth of 0.4m (BH2). The fill contained brick and concrete fragments.

Natural Soils

Marine sand/silty sand was encountered below the fill and extended to the borehole termination/refusal depths of 4.4m (BH2) and 5.95m (BH1 & BH3) below original surface levels. Based on the Standard Penetration Test (SPT) results, the marine sand was initially very loose to loose relative density, increasing to loose and medium dense relative density with depth. In BH2, the refusal of the SPT at 4.4m depth has

been interpreted to represent a band of either dense (or greater relative density) or indurated (i.e. coffee rock) sand, and not bedrock.

The marine sands within the site are expected to extend below the termination/refusal depths of BH1 to BH3. The relative density of the sands below these depths is likely to be variable (due to the nature of marine deposition) and range between very loose to dense and possibly very dense. Clay bands which are expected to be laterally and vertically discontinuous may also be present within the sandy deposits.

Weathered Bedrock

Bedrock was not encountered within the depths of the investigation at the site. However, weathered sandstone and/or interbedded siltstone and sandstone bedrock was encountered at previous nearby investigations at depths in excess of 30m. The depth to the surface of the weathered bedrock generally increased to the south and east; that is, towards Manly Beach and Manly Cove. The bedrock quality also appears to become fairly poor (i.e. soil strength to very low rock strength) as the rock depth increases.

Groundwater

BH2 was 'dry' during and for a short time following drilling, whereas BH1 encountered groundwater seepage at 4.7m depth. On completion of drilling BH3, the sides of the borehole collapsed at 4.9m depth. In sandy soils, the collapse depth is usually an indication of groundwater. Although long-term and continuous groundwater monitoring has not been carried out at the site, it is expected the groundwater levels may fluctuate with tidal levels.

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The proposed development will comprise the construction of a five storey building overlying a two level basement, with the basement extending to the site boundaries. The site is bound by a two and three storey building to the south (No. 33 Belgrave Street) and roadways to the east, north and west. The excavation for the proposed basement will extend through fill and natural marine deposits, and will encounter groundwater. On this basis, the construction is expected to comprise anchored or propped secant pile shoring walls, temporary dewatering likely using spearpoints, excavation of the sandy deposits using conventional excavation equipment, and piled footings to support the overlying structure. Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report.

The following comments and recommendations are preliminary only and must be reviewed, and revised as necessary, following the completion of a site specific geotechnical and hydrogeological investigation to inform the detailed structural design.

5.1 Sydney Water

Based on an available and current 'Before You Dig' drawing (Sequence No. 223795994) provided by Sydney Water Corporation (Sydney Water), a number of Sydney Water assets are located beyond the site as follows:

- Two 150mm and 200mm diameter 'Cast Iron Cement Lined' (CICL) water mains are located below Belgrave Street and Whistler Street;
- A 225mm diameter 'Vitrified Clay' (VC) sewer pipe is also located below Whistler Street; and
- Several water and sewer pipes are located below Raglan Street, although they appear to be setback at least 10m to the north of the site.

As several Sydney Water assets are located within close proximity to the site boundaries, we recommend that a Water Services Coordinator (WSC) be engaged very early in the design process to determine whether the proposed development will be subject to a 'Specialist Engineering Assessment' (SEA), which requires an estimate of pipe deflections using finite element modelling (FEM) software to determine the effect on the Sydney Water assets from the basement excavation. This information is then used as an input, by others, to complete an SEA of the effect of the proposed basement on the Sydney Water infrastructure. The additional geotechnical and hydrogeological investigation must precede the SEA, if required.

5.2 Transport for New South Wales

Belgrave Street is a Transport for New South Wales (TfNSW) asset, and an engineering assessment to check what potential impact the proposed basement will have on the adjacent roadway may be required. Compliance set out in the TfNSW 'Technical Direction: Geotechnology' (Ref. GTD 2020/001, Version No. 1, dated 2 July 2020) should be satisfactorily met using conventional engineering design procedures.

If required, the assessment will need to include geotechnical and hydrogeological investigations, analyses and assessment of the proposed basement excavation, using FEM or specialist retaining wall software (i.e. Wallap) to assess the deflection and structural actions of the shoring wall. A Ground Movement Monitoring Program may also be required by TfNSW, which outlines the required monitoring (i.e. visual, survey, vibration, inclinometer, groundwater etc.) through the construction process.

5.3 Dilapidation Reports

Following the completion of the additional investigation and prior to any demolition and excavation commencing, we recommend that a detailed dilapidation report be prepared for the neighbouring building to the south. The dilapidation survey report can be used as a benchmark for assessing possible future damage claims arising from the works. As dilapidation survey reports are relied upon for the assessment of potential future damage claims, they must be carried out thoroughly with all defects rigorously described (i.e. defect type, defect location, crack width, crack length etc.) and defects photographed where practical.

The respective owner of the adjoining property should be asked to confirm in writing that the dilapidation survey report on their property presents a fair assessment of the existing conditions. We note that Council may also require that dilapidation reports be prepared for any adjoining Council assets.

5.4 Excavation

All excavation should be carried out with reference to the most recent 'Excavation Work – Code of Practice' by Safe Work Australia.

5.4.1 Demolition and Excavation

Site preparation works will include demolition of the existing buildings and structures, and excavation of any deleterious or contaminated fill. Care should be taken during site preparation works and subsequent bulk excavation not to undermine or remove support from the buildings or site boundaries. This work will need to be completed using suitably experienced (and insured) contractors.

Prior to bulk excavation commencing, the footing details for the adjacent building to the south should be confirmed by available 'as-built' structural drawings, though judging by the age of the building it is likely the drawings will not be available. The purpose of the review is to confirm whether any strengthening or underpinning of the adjacent footings is required. If the drawings are not available, several test pits should be excavated at the base of the wall to attempt to determine the depth and geometry of the footings. Permission from the owners to complete the works should be sought and the test pits should be jointly inspected by the geotechnical and structural engineers. We note that any underpinning solution where only parts of the building are underpinned down to a different, likely stiffer stratum, there is a risk of differential movements occurring in the long term.

The proposed two level basement is expected to extend to a maximum depth of approximately 6m below existing grade, with locally deeper excavations, say in the order of about 1m depth, necessary for the proposed lift over run pit within the central portion of the site. Excavation below existing surface levels will extend through the fill and penetrate the underlying natural sandy soils. Groundwater is also expected to be encountered below approximately 4.5m depth.

We expect that excavation of the soils should be readily achievable using conventional excavation equipment, such as the buckets of tracked excavators. Due to the presence of poorly compacted sandy fill and loose sands, 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 adjoining building and surrounding structures (i.e. roads and buried services). We recommend that removal of any concrete slabs and footings be completed using a diamond saw followed by removal of the concrete pieces using a bucket attachment to the tracked excavator. When using the saw, the resulting dust should be suppressed by spraying with water.

5.4.2 Seepage and Dewatering

No long term groundwater monitoring has been carried out at the site. However, the results of the initial investigation indicate/infer groundwater levels below approximately 4.7m depth. Further long-term groundwater monitoring is therefore recommended to assess the variability in water levels for detailed design of the lateral water pressures on the shoring walls and hydrostatic uplift pressures on the basement floor slab. In addition, the criteria set out in the 'Minimum Requirements for Building Site Groundwater

Investigation and Reporting' (Ref. PUB20/940, Ver02.2210 October 2022) prepared by the NSW Department of Planning, Industry and Environment (DPIE) will also need to be satisfied. Notwithstanding the results of any long term groundwater monitoring, for preliminary analysis consideration should be given to adopting a groundwater level at around RL2.5m (i.e. 3.5m depth) as the groundwater table may rise over the design life of the building (i.e. during flood/high rainfall events, sea level rise etc.).

Where required, groundwater will need to be lowered to at least 1m below the proposed two level basement and lift over-run pit during the construction period. Dewatering must be carefully controlled and monitored to reduce the risk of excessive drawdown occurring outside the basement causing settlement of buried services or adjoining building supported on shallow footings. However, as a shoring wall will be constructed around the basement perimeter, we expect temporary dewatering should not cause excessive drawdown outside the site provided the cut-off is properly designed and constructed. If there is no continuous clay layer into which a cut-off can be constructed, it is likely that substantial embedment depths of the shoring wall piles will be required to limit inflows. Groundwater monitoring wells will need to be installed outside the basement excavation to ensure groundwater levels are not being drawn excessively down during bulk excavation and dewatering activities.

Borehole infiltration testing must also be carried out to assess the permeability of the sandy soils. The rate at which the groundwater can be extracted will be a function of both the mass permeability of the soils and the capacity and number of pumps used. We note that disposal of extracted groundwater will be affected by practical considerations such as whether or not the groundwater can be discharged to the stormwater or sewer system (with or without on-site treatment).

Detailed hydrogeological analyses with finite element or SEEP/W software using the results of the long term groundwater monitoring and borehole infiltration tests will be required to assess the effect of dewatering on the adjacent structures and to optimise the depth of the shoring wall cut-off. At this stage, it is difficult to predict the likely cut-off depths based on the results of the initial investigation, however, a minimum depth in the order of about 6m below the proposed basement level should not be unexpected. The cut-off depths should also be designed to reduce the risk of 'boiling', where the pore water pressures in the soil exceed the weight of the soil resulting in a zone of zero soil shear strength. Due to the inherent limitations of predicting accurate mass soil permeability, and the empirical relation of analysis methods, a Factor of Safety (FOS) of 2 is applied to the case for 'boiling'. This phenomenon can occur very quickly following the failure or inadvertent turning off of even a single dewatering well and so we consider that for these risks alone, the embedment of the cut-off should be sufficient to have an adequate FOS against 'boiling'.

A dewatering licence will also need to be obtained from DPIE for all temporary dewatering activities. As permanent dewatering systems are not likely to be approved, the basement will need to be tanked and designed to withstand hydrostatic uplift pressures. The hydrogeological analysis mentioned above will also form part of the application for the dewatering licence.

5.5 Retention

5.5.1 Temporary Batter and Retention Options

Prior to the commencement of the detailed design, details of the neighbouring proposed development to the south-east (No. 21 Whistler Street) should be sought, as well as a possible commencement date of construction, as the neighbouring basement level will have implications for the basement/building with the subject site, such as the use of soil anchors and/or props.

As the proposed basement will extend to the site boundaries and below the groundwater table, temporary batter slopes will not be feasible. Therefore, the excavation will need to be supported by a retention system such as a grout injected Continuous Flight Auger (CFA) secant pile wall. Double rotary (i.e. cased) CFA piles will need to be adopted to improve the verticality of the piles which is imperative for adequate overlap of adjacent secant piles to ensure a water tight wall. Sheet piles are not recommended due to their potential to cause vibrations during installation that may cause damage to adjacent buried services or buildings. Subject to geotechnical inspection, temporary batter slopes through the soil profile for the proposed lift over-run pit could be formed at no steeper than 1 Vertical (V) in 2 Horizontal (H) provided all surcharge loads (including equipment and site personnel) are kept well clear of the excavation crest.

When using conventional CFA piling techniques, decompression of the sands could occur when drilling below groundwater. A site trial in the centre of the site would need to be undertaken under the direction of a geotechnical engineer to assess the potential for sand decompression. However, these effects could be controlled by the aforementioned double rotary CFA piles.

Any shoring walls which are founded in loose (or greater) relative density sands may be able to support structural loads from the overlying building, subject to the results of the additional geotechnical investigation. Lateral restraint of the shoring walls in the form of soil anchors and/or props will also be required to reduce deflections (i.e. ground movements) occurring outside the basement excavation. If anchors are to be installed, they will extend beyond the site boundaries and permission of the owners and authorities must be obtained before installation. Considering the limited footprint of the site and capacity of soil anchors, we anticipate that corner props/braces may be a more suitable alternative for this site, although there may be some difficulty with the re-entrant corner.

5.5.2 Shoring Design Parameters

The major consideration in the selection of earth pressures for the design of the retention system is the need to limit deformations occurring outside the excavation. For preliminary design of propped or anchored shoring walls, we recommend the use of a rectangular earth pressure distribution of $8H$ (kPa), where H is the retained height in metres. A bulk unit weight of 20kN/m^3 should be adopted for the retained profile.

The lateral toe restraint of the shoring walls can be calculated using a 'passive' earth pressure coefficient, K_p , of 3.0 for sands of at least medium dense relative density. A factor of safety of at least 2 should be applied

to the calculated resistance due to the large strain necessary to generate the full passive case. Localised excavation in front of the walls, e.g. for buried services, must be considered in the design.

All surcharge loads and hydrostatic pressures should be allowed for in the shoring design. The design must also take into account the groundwater situation where there will be differential water levels on the active and passive sides of the wall.

As a guide, soil anchors may be designed based on a friction angle of 32° for sands of at least medium dense relative density. All anchors should be bonded behind a line drawn up at 45° from the base of the excavation, have a minimum free length of at least 3m and be proof loaded to 1.3 times the maximum working load under the direction of a geotechnical engineer or construction superintendent independent of the anchoring contractor. Lift-off tests should be carried out on 10% of anchors 48 hours following locking off to confirm the anchors are holding their load. The designer of the shoring wall must also predict the deflections of the shoring wall and the structural engineer must use these predictions to make their decision on whether the adjoining building and/or buried services can tolerate these movements. We note that it is normally good practice for anchors to be a specialist design and construct sub-contract to avoid disputes if anchors fail to hold their test load.

Care is required when installing anchors in sands as 'ground loss' could occur, especially when drilling below the groundwater table, causing settlements or differential settlements which may adversely affect adjacent structures. If anchors are to be installed below neighbouring properties or roadways, then permission of the respective owners must be obtained prior to installation.

Further to the above, we expect that the prediction of deflections will be a TfNSW requirement for the proposed western basement wall adjacent to Belgrave Street, as indicated in the TfNSW 'Technical Direction GTD2012/00'. The shoring wall deflections could be analysed using computer-based soil-structure interaction analysis methods (e.g. Wallap or Plaxis), which could result in cost savings compared to a design based on the above simplified earth pressure assumptions. Analysis using soil structure interaction methods can model the actual excavation stages, including progressive anchoring/propping, and outputs include structural actions in the piles, anchor/prop loads, and wall movements. The analysis should be completed by an engineer with a good understanding of soil-structure interaction behaviour, including an understanding of when soil wall friction should and should not be used etc.

5.6 Footings

Based on the anticipated high loading of the proposed structure, there are a number of potential footing options that could be considered. These options comprise piles (CFA or steel screw) founded in the soil profile, piles socketed into bedrock or a piled raft slab. Should a piled raft slab be considered, further advice should be sought from this office. Shallow footings founded within the natural marine sands are unlikely to be feasible for the expected large column loads.

For the footing options described above, additional geotechnical investigations will be required to confirm the results of the previous investigation, assess the nature and strength/relative density of the soils over the

entire footprint of the site and at greater depths including for the presence of any compressive layers (i.e. soft clays/peats), and core drill the bedrock where rock sockets are considered. Once a preliminary footing design has been formulated, we recommend that further advice be sought from this office. The footing options are discussed in detail below.

Piled Footings

The proposed building may be supported using piled footings (single piles or pile groups) founded in the underlying sandy soils. It will be critical that piles are founded at similar depths and on uniform bearing stratum, such that structural loads are not transferred to any underlying soils of variable strength.

Due to the expected variability of the soils and groundwater encountered at moderate depth, we recommend that either CFA or steel screw piles are adopted for this site. Conventional bored piles are not suitable due to the deep and collapsible nature of the marine soils and groundwater seepage. The piles should be installed on a design and construct basis and where this approach is considered, further advice should be sought from specialist piling contractors.

For piles founded in sands of at least medium dense relative density below bulk excavation level, the allowable bearing pressure will be dependent on the pile diameter, embedment depth, and assumed groundwater level. Indicatively, 0.6m diameter piles (CFA or screw piles) founded in medium dense sands at approximately 11m depth (i.e. about 5m of embedment below bulk excavation levels), assuming a groundwater level at approximately 3.5m depth, could be tentatively designed based on an allowable end bearing pressure (ABP) of 800kPa, provided further investigations are carried out to confirm the extent of the foundation material. Predicted settlements for the foundation piles embedded into medium dense sands are expected to be within tolerable limits. Such settlements are expected to be instantaneous on loading provided any underlying lower strength soils or compressible seams are not affected. With regard to screw piles, the contribution of a 'secondary helix' or 'skin friction' should be ignored.

Piles could also be extended down into the underlying bedrock at depth. Based on our previous investigation to the west of the subject site, bedrock was encountered at depths in excess of 30m. Considering the anticipated poor quality of the rock at these depths, piles drilled into bedrock are unlikely to be an economical solution as they would likely have end bearing pressures similar to those on the sands, and would require very large piling rigs for installation.

Tracked piling rigs may need to be provided with a suitable working platform before they can establish to site. The design of the working platform will need to be based on the loadings and track dimensions supplied by the contractor for the specific equipment proposed. The design of this platform should be based on the methodology outlined in BRE 2004 'Working Platform for Tracked Plant'.

Footing Inspections

We recommend that a geotechnical engineer witness the installation of the initial shoring wall piles, and several subsequent foundation piles (if adopted) thereafter, though the inspections will essentially only be able to deduce that the pile is founded at a level consistent with available geotechnical information obtained from the additional investigation. Therefore, piles should initially be installed adjacent to the subsequent

test locations to assist the geotechnical engineer in confirming the materials in which the piles are founded. The geotechnical load capacity of the piles must be certified by the piling contractor.

5.7 Basement Floor Slab

5.7.1 Basement Slab

For a tanked basement, the basement floor slab or raft slab must be designed for uplift forces due to hydrostatic pressure. The maximum groundwater levels should be evaluated for this purpose, from long-term monitoring and impacts of flooding, climate change etc. As the proposed building is up to five storeys, it is unlikely to provide sufficient self-weight to resist uplift forces. Therefore, the basement floor slab may need to be supported by tensile elements (e.g. soil anchors, tension piles) within the underlying sandy soils, although we reiterate that additional investigations will be necessary to confirm the subsurface material below the basement slab. Tension piles founded in the underlying bedrock anticipated to be at depths in excess of 30m will be uneconomical due to the depth to, and quality of, the underlying rock.

The design of tensile elements should be carried out by a specialist piling contractor. We recommend that soil anchors which are bonded into sand of at least medium dense relative density be tentatively designed using an effective friction angle of 32° . The initial 0.5m of the soil anchors should be ignored due to possible disturbance effects from excavation. Hydrostatic uplift pressures may also be resisted by steel screw piles whereby the helix forms a horizontal anchor plate. Steel screw piles acting in tension should be professionally designed using the friction angle outlined above and a unit weight of 19kN/m^3 (i.e. medium dense sand). We anticipate the working load of a single steel screw pile (0.6m diameter) in tension will be in the order of 100-150kPa. The design of the soil anchors/tension piles must be checked for 'cylindrical shear' and 'cone-liftoff'. Interaction effects with adjacent tensile elements must also be considered in the design.

5.7.2 Subgrade Preparation

The proposed basement should be designed as suspended between piles, and then no particular subgrade preparation would be required apart from levelling and nominal track or tamping with an excavator bucket, i.e. for a level and rigid base to form the slab.

5.8 Earthquake Design Parameters

The following parameters should be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia' (including Amendments 1 & 2).

- Hazard Factor (Z) = 0.09; and
- Site Subsoil Class = D_e

There is a possibility that the measurement of shear wave velocity could reduce the site subsoil class to C_e , and we can arrange for such testing as it would most likely make a significant difference to the building design.

5.9 Further Geotechnical Input

As detailed in the preceding sections of this report, JKG 1986 provides only a limited coverage of the site, and to a maximum depth of about 6m. Due to the variable nature of a 'marine' profile and the complicated soil/structure interaction between the proposed building/basement and soils, a detailed geotechnical investigation will be required to more fully assess the geotechnical conditions required for design, temporary dewatering etc.

The detailed investigation is likely to comprise several Cone Penetration Tests which provide a continuous profile of soil strength/relative density with depth. The investigation must also satisfy the requirements set out in the DPIE document and WaterNSW 'Geotechnical Investigation Reports, Minimum Requirements' (Fact Sheet 070521) document. As outlined in Section 5.4.2, this will involve the installation of groundwater monitoring wells, long-term groundwater level monitoring over a period of at least three months, and borehole infiltration tests to estimate the permeability of the soil profile.

In addition to the above, TfNSW may require geotechnical analysis of the proposed retention system fronting Belgrave Street to confirm the lateral deflections of the wall is within tolerable limits set out in the TfNSW 'Technical Direction: Geotechnology'. Finally, Sydney Water may also require an SEA be completed to confirm that any movements induced within their nearby sewer/water main as a result of the proposed development will fall within the criteria set out in the Sydney Water procedures.

The following additional geotechnical input immediately prior to and during excavation and construction will likely be required:

- Dilapidation survey reports on the neighbouring building to the south, and potentially the Council assets (i.e. roadways) to the north, east and west.
- Review 'as built' drawings of the existing footing details for the adjacent building to the south. If the drawings are unavailable, test pits should be excavated to attempt to expose the footing details which should be inspected by a geotechnical engineer.
- Proof testing of temporary anchors.
- Monitoring of groundwater during construction.
- Geotechnical inspection of pile drilling and raft slab subgrade.

Following demolition of the onsite structures, we will be able to carry out the required investigation and analyses to satisfy the requirements outlined above. However, if permission were granted, we could carry out an initial investigation comprising say two CPT's within the vacant site to the south-east (No. 21 Whistler Street).

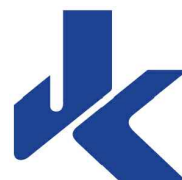
6 GENERAL COMMENTS

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.



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title: SITE LOCATION PLAN	
Location: 40-43 BELGRAVE STREET, MANLY, NSW	
Report No: 35999PE	Figure No: 1
JKGeotechnics	



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

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_0), 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_0).

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 or where only a limited investigation has been completed or 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 N/A

Laboratory Classification Criteria

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

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10} D_{60}}$$

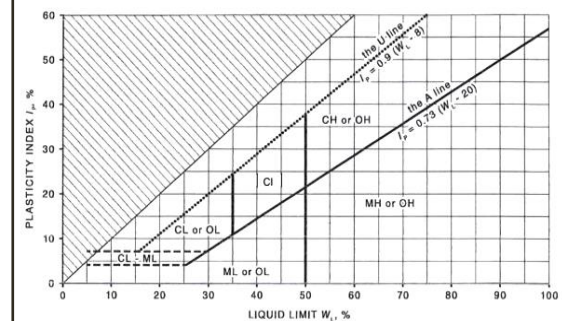
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

- 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.
- 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.
- Clay soils with liquid limits $> 35\%$ and $\leq 50\%$ may be classified as being of medium plasticity.
- The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
fine 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	–	–	–	–

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	Sample taken over depth indicated, for environmental analysis.																	
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.																	
	DB	Bulk disturbed sample taken over depth indicated.																	
	DS	Small disturbed bag sample taken over depth indicated.																	
	ASB	Soil sample taken over depth indicated, for asbestos analysis.																	
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.																	
	SAL	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	Vane shear reading in kPa of undrained shear strength.																	
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).																	
Moisture Condition (Fine Grained Soils) (Coarse Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.																	
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.																	
	w < PL	Moisture content estimated to be less than plastic limit.																	
	w ≈ LL	Moisture content estimated to be near liquid limit.																	
	w > LL	Moisture content estimated to be wet of liquid limit.																	
	D	DRY – runs freely through fingers.																	
	M	MOIST – does not run freely but no free water visible on soil surface.																	
	W	WET – free water visible on soil surface.																	
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.																	
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.																	
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.																	
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.																	
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.																	
	Hd	HARD – unconfined compressive strength > 400kPa.																	
	Fr	FRIABLE – strength not attainable, soil crumbles.																	
	()	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.																	
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE																	
	L	LOOSE																	
	MD	MEDIUM DENSE																	
	D	DENSE																	
	VD	VERY DENSE																	
	()	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.																	
		<table> <tr> <th></th><th>Density Index (I_D) Range (%)</th><th>SPT 'N' Value Range (Blows/300mm)</th></tr> <tr> <td>VL</td><td>≤ 15</td><td>0 – 4</td></tr> <tr> <td>L</td><td>> 15 and ≤ 35</td><td>4 – 10</td></tr> <tr> <td>MD</td><td>> 35 and ≤ 65</td><td>10 – 30</td></tr> <tr> <td>D</td><td>> 65 and ≤ 85</td><td>30 – 50</td></tr> <tr> <td>VD</td><td>> 85</td><td>> 50</td></tr> </table>		Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)	VL	≤ 15	0 – 4	L	> 15 and ≤ 35	4 – 10	MD	> 35 and ≤ 65	10 – 30	D	> 65 and ≤ 85	30 – 50	VD	> 85
	Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)																	
VL	≤ 15	0 – 4																	
L	> 15 and ≤ 35	4 – 10																	
MD	> 35 and ≤ 65	10 – 30																	
D	> 65 and ≤ 85	30 – 50																	
VD	> 85	> 50																	
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

Term		Abbreviation		Definition
Residual Soil		RS		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.
Extremely Weathered		XW		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.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	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.
Moderately Weathered		MW		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.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

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 $Is_{(50)}$ (MPa)	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
	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	P	Planar
	C	Curved
	Un	Undulating
	St	Stepped
	Ir	Irregular
	Vr	Very rough
	R	Rough
	S	Smooth
	Po	Polished
	Sl	Slickensided
	Ca	Calcite
	Cb	Carbonaceous
	Clay	Clay
	Fe	Iron
	Qz	Quartz
	Py	Pyrite
	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
	mm.t	Defect thickness measured in millimetres



APPENDIX A

**Borehole Logs 1 to 3 From Our Previous Geotechnical
Investigation Report (Ref. 4159) Dated 21 January 1986**

BOREHOLE LOG

Client: HUGHES TRUEMAN-LUDLOW PTY. LTD.
 Project: PROPOSED TWO TO THREE STOREY DEVELOPMENT
 Location: RAGLAN STREET, MANLY, N.S.W.

Job No. 4159
 Date: 15.1.86

Method: SPIRAL AUGER
 EDSON 3000

R.L. Surface:
 Datum:

Groundwater record	Samples	FIELD TESTS	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
						FILL: Silty sand, with pieces of concrete	M	VL		
	DS	N=5 3, 3, 2	1		SM	SILTY SAND: Fine to medium grained, brown. — Becoming light brown.	M	L		
					SC	— Grading to sand, fine to medium grained, trace of silt, orange brown.		VL		
			2							
	DS	N=4 2, 2, 2	3							
						As above, but medium grained no silt.		to L		
	DS	N=16 5, 7, 9	4					MD		
			5				W			
WHILE BOREHOLE AT 5.95m	DS	N=17 6, 8, 9	6							
						END OF BOREHOLE AT 5.95m				

BOREHOLE LOG

Client: HUGHES TRUEMAN-LUDLOW PTY. LTD. Project: PROPOSED TWO TO THREE STOREY DEVELOPMENT Location: RAGLAN STREET, MANLY, N.S.W.															
Job No. 4159 Date: 15.1.86		Method: SPIRAL AUGER EDSON 3000		R.L. Surface: Datum:											
Groundwater record	Samples	FIELD TESTS	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks					
DRY ON COMPLETION						FILL: Sand and bricks	D								
	DS	N = 4 2, 2, 2	1		SP	SAND: Fine to medium grained, orange brown — — — — — Becoming light brown	M	VL							
			2												
	DS	N = 10 3, 5, 5	3												
			4												
DS	N > 24 7, 14, 10/60 BOUNCING														
			5			END OF BOREHOLE AT 4.4m									
			6												

BOREHOLE LOG

Client: HUGHES TRUEMAN-LUDLOW PTY. LTD.
 Project: PROPOSED TWO TO THREE STOREY DEVELOPMENT
 Location: RAGLAN STREET, MANLY, N.S.W.

Job No. 4159
 Date: 15.1.86

Method: SPIRAL AUGER
 EDSON 3000

R.L. Surface:
 Datum:

Groundwater record	Samples	FIELD TESTS	Depth (m.)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition	Consistency/ Rel. Density	Hand Penetrometer Readings kPa.	Remarks
						CONCRETE OVER FILL: silty sand				
					SM	SILTY SAND: Fine to medium grained, grey brown	M	VL		
	DS	N=4 1,2,2	1			SAND: Fine to medium grained, orange brown trace of silt				
			2							
	DS	N=4 2,2,2	3			As above, no silt				
			4					MD		
	DS	N=27 4,13,14	5				W			
										
	DS	N=11 3,5,6	6			----- Becoming grey				
										
						END OF BOREHOLE AT 5.95m				

BOREHOLE
 CAVED
 IN TO
 HERE
 ON
 COMPLETION