



**REPORT TO
COURTNEY SMITH**

**ON
GEOTECHNICAL INVESTIGATION AND
STABILITY ASSESMENT**

**FOR
PROPOSED ALTERATIONS AND ADDITIONS**

**AT
11A MONASH CRESCENT, CLONTARF, NSW**

Date: 5 November 2025

Ref: 37642YFrptRev1

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ATTACHMENTS

Table A: Summary of Risk Assessment to Property

Table B: summary of Risk Assessment to Life

EnviroLab Services Certificate of Analysis No. 384873

Borehole Log 1

Dynamic Cone Penetration Test Results Sheet

Figure 1: Site Location Plan

Figure 2: Borehole Location Plan



Vibration Emission Design Goals

Report Explanation Notes

Appendix A: Landslide Risk Management Terminology

Appendix B: Geotechnical Data from Previous Investigations

1 INTRODUCTION

This report presents the results of a geotechnical investigation and stability assessment for the proposed alterations and additions at 11A Monash Crescent, Clontarf, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Courtney Smith and was carried out in accordance with our fee proposal, Ref: P71680YF, dated 23 April 2025.

We understand from the provided architectural drawings prepared by Corben Architects (Drawing List as per the Cover Page, Dwg. DA000, Rev A dated 29 October 2025) that it is proposed to construct minor alterations to the existing residence and construction of new stairs to access an existing small basement, which is located below the front of the existing house. This will primarily result in the realignment of the staircase from the south-eastern wall, where it is currently located, approximately 1.5m further to the north-west, where it will be located more towards the centre of the house. The garage at the front of the property will be demolished and reconstructed to include a double garage, a bin storage area, sauna and wellness area. At the rear of the property the stairs providing access to Clontarf beach will also be realigned and will be straightened such that they run perpendicular to the sea wall. We have been provided with a report prepared by Horton Coastal Engineering dated 24 October 2025 that contains a coastal engineering risk management of the existing seawall.

Realignment of the staircase in the house will also result in the realignment of the stairs providing access to the existing basement. The current proposal is to retain the existing stairs and cover with the new structure including the basement stair opening, although removal of the lower steps of the existing stair will still be required. The new stairs will be located further to the north-west and will step down into the basement, which is at reduced level (RL) 0.43m. The partially demolished existing wall to allow for the construction of the new stair will only be demolished down to RL0.755m, which is above the mean high water spring tide in order to avoid for the need of ongoing dewatering during the works. In the long-term the structure will be a water-tight structure. The proposed alterations and additions will match existing floor and site levels comprising of a proposed Finished Floor Level of RL3.02m for the house, RL2.94m for the garage and rear patio area and RL0.43m for the basement.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at the test locations. Based on these results we have provided comments and recommendations on site preparation, excavation, groundwater, retention systems, footings, slabs-on-grade and soil aggressivity. We have also assessed the risk posed by slope instability of the site to both life and property and provided preliminary comments and recommendations on remediation or replacement of the existing seawall, if required.

2 INVESTIGATION PROCEDURE

2.1 Current Investigation

The fieldwork comprised of the following:

- A site walkover by our Senior Associate Geotechnical Engineer on 4 June 2025 to undertake a slope stability assessment.
- The completion of a subsurface investigation that was carried out on 25 June 2025 and consisted of:
 - One (1) hand auger borehole, BH201, that was drilled to a termination depth of 3.0m;
 - Three (3) Dynamic Cone Penetration (DCP) tests, DCP201 to DCP2033, that were completed to refusal depths ranging from 0.15m and 4.0m.

The borehole and DCP test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. Figure 2 also includes the geotechnical hazards identified during the stability assessment. The approximate surface RLs at the test locations were estimated by interpolation between spot heights shown on the survey drawing prepared by Waterview Surveying Services (Dwg. No. 1776detail 1, Rev A dated 6 September 2023). The surface level datum used was the Australian Height Datum (AHD). The survey drawing was used as a base plan for Figure 2.

The boreholes were drilled to identify the soils present and to assess the depth to groundwater, while the DCP tests were used to assess the apparent compaction of the fill and the relative density of the sands. Groundwater observations were made in the boreholes during and on completion of drilling. No longer term groundwater monitoring was completed.

On completion of testing, selected samples were sent to an external NATA registered laboratory, Envirolab Services Pty Ltd (Envirolab) for pH, sulphate, chloride and resistivity testing. The results of the tests are presented in the attached Envirolab Certificate of Analysis No. 384873 and are also summarised in Section 3.3.

Our geotechnical engineer was on site during the fieldwork and set out the test locations, nominated the testing and sampling and prepared the borehole logs and DCP test results sheets. The borehole logs, which include field test results and groundwater observations, and DCP test results sheets are attached to this report, together with our Report Explanation Notes, which define the logging terms and symbols used and further describe the investigation techniques and their limitations.

2.2 Previous Investigations

JK Geotechnics, formerly Jeffery & Katauskas, have undertaken investigations at the subject site in 2002 and 2003 comprising of:

- Three (3) hand auger borehole drilled to a termination depths between 2.9m and 3.0m below surface levels;
- Two (2) boreholes drilled using a track mounted rig to termination depths of 12.0m and 13.5m below surface levels.
- Two (2) test pits excavated using hand tools to 0.9m below surface levels to expose the existing footings at the time of the investigation.

Unfortunately, the above investigation locations do not have surface levels, however we presume that they were completed at surface levels that are similar to those that currently exist. Notwithstanding this, the geotechnical data is useful to compare with the results of the current investigation.

The relevant geotechnical data from these investigations have been provided in Appendix B.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in the flat, littoral zone between the steep cliff-lines that drop down from the ridgeline to the north and east and Middle Harbour to the south-west. For simplicity, the site description below has adopted a site north and south as Monash Crescent and Middle Harbour, respectively. We note that Clontarf Beach is underwater during high tide.

The site itself is generally flat with dimensions of about 12m (west to east) by 38m (north to south). At the time of the investigation, the site included a three storey cement rendered and weatherboard house. A single storey cement rendered garage is present in the north-eastern corner. The buildings appear in good condition based upon an external inspection and cursory internal inspection. The external areas of the site predominantly comprised tiled pavements, with small garden areas at the northern end of the site adjacent to the existing garage and along the northern end of the eastern boundary. Raised planter beds with low height hedges and a small tree was present within the rear or southern end of the site and in this portion of the site ran along the eastern and western boundaries.

The rear or southern end of the site contains a tiled patio and in-ground pool and was retained by a cement rendered sea wall. This wall was approximately 1.8m high with sandstone blocks visible at base, indicating the wall itself may be a cement rendered sandstone block wall. A stormwater PVC pipe discharges through the eastern end of sea wall into the harbour.

The eastern and western boundary walls comprised of a mixture of cement rendered, sandstone block and timber walls that appeared in good condition. The neighbouring properties contained two storey brick and cement rendered residences that appeared in good condition with no visible defects. The neighbouring

properties appear to have similar surface levels based upon very limited observations from within the subject site, the street and beach frontages.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Map of the Sydney Region indicates that the site is underlain by deep alluvial deposits. Reference should be made to Plate 1 below for a visual representation of the local geology.

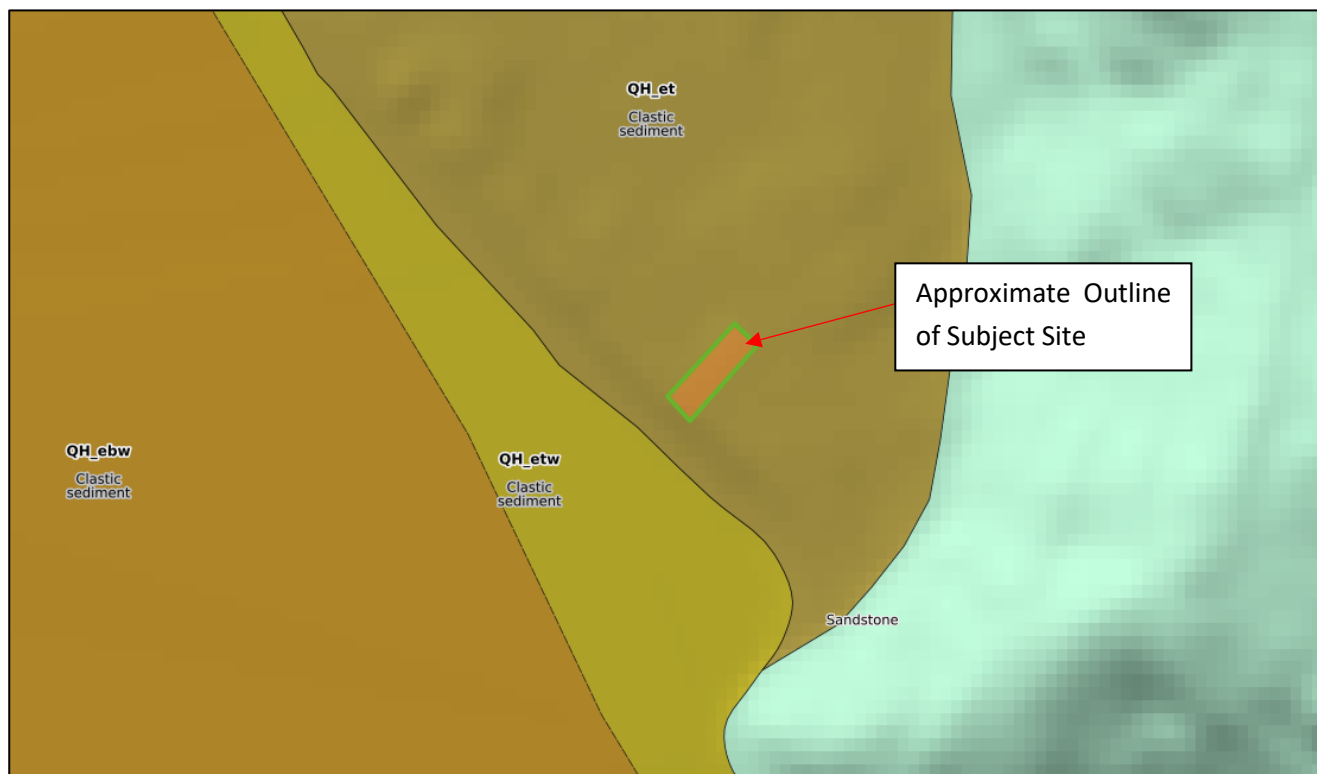


Plate 1 – Geological map, QH_et/etw=Estuarine tidal-delta flat, Sandstone=Hawkesbury Sandstone

The investigation revealed a generalised subsurface profile comprising silty sand fill overlying natural silty sand that was assessed to be of marine origin. A summary of the investigation findings is presented below. For a detailed description of the materials encountered or DCP test results at a particular location, reference should be made to the attached borehole logs and DCP test results sheets. We note the current investigation results appeared to correlate reasonably well with the previous 2002/2003 investigations.

Concrete Slabs/Pavements

As discussed above in Section 3.1, the site is currently developed and sandstone pavers and concrete pavements are present over parts of the site. The thickness of these pavers at the southern end of the site at DCP203 was approximately 100mm, however refusal of the DCP test at 0.15m indicated the potential presence of a second pavement/slab.

Fill

Fill was encountered in BH201 and extended to approximately 1.5m depth. We note the previous 2002/2003 investigations encountered fill to between depths of about 0.7m and 1.0m. The fill is likely to be deeper in parts of the site, particularly behind the existing seawall at the southern end of the site where it possibly extends to depth in excess of about 1.7m based on the wall height. The fill comprised silty sand and included traces of roots, root fibres, igneous/sandstone gravel and concrete fragments. The fill was assessed to be poorly compacted.

Marine Sands

Marine sands were assessed to underlie the fill and extended to at least 4.0m depth based on the DCP tests. We note that, based on our experience primarily within the neighbouring western property (No. 13 Monash Crescent), we expect the sands to extend to at least about 25m depth. The sands were initially of very loose to loose relative density but increased in relative density to medium dense at depths of 2.7m in DCP201 and 2.6m in DCP202, or at about RLRL0.2m and RL0.3m, respectively. In DCP202, the medium dense sands continued to the termination depth at 4m, however in DCP201, dense sands were encountered at approximately 3.5m depth, or at about RL-0.6m. We note that based on the previous 2002/2003 investigations, the dense sands do not appear to be consistently present across the site, however the medium dense sands were.

Groundwater

Groundwater seepage was encountered during drilling in BH201 and approximately 2.5m depth, or at about RL0.4m. The groundwater is expected to be tidal given the sites proximity to Middle Harbour.

Inferred Bedrock

Bedrock was not encountered in this investigation. However, based on nearby geotechnical data, bedrock is expected to be deeper than about RL-22m.

3.3 Laboratory Test Results

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below and are also presented in the attached Envirolab Certificate of Analysis No. 384873. Section 5.7 provides an interpretation of these results with regards to soil aggression on buried concrete and steel structures.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO ₄ (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
201	0.5-0.7	FILL: Silty Sand	6.5	85	<10	3,500
201	1.5-1.7	Silty SAND	6.7	<10	<10	19,000
201	2.5-2.6	SAND	7.1	10	10	10,000

4 RISK ASSESSMENT

4.1 Landslide Risk Assessment Criteria

Based on the Northern Beaches online mapping tool, the site falls within a landslip hazard area 'G3', as shown in Plate 2 below. While a slope stability assessment would typically not be required for this site, as greater than 1m of fill is likely to be present behind the seawall and excavation will extend to depths of greater than 2m, an assessment is required. As such, we have undertaken a slope stability assessment.



Plate 2 – Northern Beaches Council Landslip Hazard Mapping

The assessment of slope stability at the site has been made using the guidelines presented in the Landslide Risk Management Concepts and Guidelines prepared by the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management¹. In this regard an acceptable risk for loss of life of 1×10^{-6} has been adopted for natural slopes for the person most at risk for the proposed development. For loss to property the acceptable risk should be determined by the owner, provided loss to property only affects the owners' property and does not impact on the property of others. As a guide, for new developments the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management adopts a risk to property of low to be acceptable. Where risks posed by slope instability are considered unacceptable, remedial measures should be adopted to reduce the risk posed to an acceptable level.

The assessment has been made on a semi-quantitative basis with quantitative values assigned to qualitative assessments. The qualitative assessments are based on judgements made in the field by the geotechnical engineer and in this regard are subjective and formed in part by the engineers' previous experiences. The range of annual probabilities assigned to the likelihood of events occurring, the recommended vulnerability values and the qualitative risk analysis matrix are presented in Appendix A.

¹ - Journal and News of the Australian Geomechanics Society, Volume 42, No 1, March 2007

4.2 Landslide Risk Assessment

4.2.1 Hazard

Reference should be made to the attached Figure 2, for the approximate location of the potential hazard and Section 3.1 for a more detailed description of the hazard. The following hazards were identified:

- **Hazard A** – Failure of the 1.6m to 1.7m high brick seawall retaining the southern rear area.
- **Hazard B** – Proposed excavation for basement staircase.

4.2.2 Risk Analysis

The attached Table A summarises our qualitative assessment of the potential landslide hazard and of the consequences to the property should the landslide hazard occur. Use has been made of the data presented in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A indicates that the assessed risk to property is Low, which would be considered acceptable in accordance with the criteria given in Reference 1.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk following the completion of the proposed development is about 1×10^{-7} . Therefore, this risk is considered acceptable in relation to the criteria given in Reference 1. Considering that the wall was estimated to have been built at least 50 years ago and is unlikely to comply with current standards, consideration could be made for stabilising or reconstructing the wall during the new development. The suitability of this wall should be assessed by a structural engineer. Where the structural engineer considers that the wall is not suitable for the design life of the house it should either be re-constructed or intervals at which it requires re-inspection nominated. Some preliminary recommendations for support (and replacement) are provided in Section 5.6.

4.3 Risk Assessment

The design project life for this project has been taken as 50 years. This provides the context within which the geotechnical risk assessment should be made. The required 50 years baseline broadly reflects the expectations of the community for the anticipated life of the development and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 50 year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, the geotechnical engineer does not warrant the development for a 50 year period, rather provides a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of the existing retaining wall is based upon a visual appraisal at the time of our inspection. Where the existing retaining wall will not be replaced/stabilised as part of the proposed development, where appropriate the structural engineer must identify the time period at which reassessment of the wall is required. Reference should also be made to the Horton Coastal Engineering report, which is discussed further in Section 5.7 below.

Our assessment was carried out for the existing site and the proposed development shown on the referenced architectural drawings, which does require localised excavation to a depth of about 3m for the basement staircase. In our assessment we have made the following assumptions:

- The proposed development works are as shown on architectural drawings.
- That no activities on the surrounding properties will be undertaken which will increase the risk posed by the subject site.
- That all Council's buried services are, and will be regularly maintained in good condition.

Provided the assumptions above are correct, we consider that our risk analysis has shown that the site in its existing condition and the proposed development can achieve an 'Acceptable Risk Management' criteria.

5 COMMENTS AND RECOMMENDATIONS

5.1 Site Preparation and Excavation

Generally minor excavation (less than 0.3m) will be required, with the exception of the proposed basement and beach access staircases where excavations between about 1.7m and 3.0m are expected. Excavation to these depths will encounter sandy fill and natural sands and may be completed using conventional earthworks equipment (e.g. small hydraulic excavators) or hand tools. We note DCP3 refused at 0.15m indicating the potential presence of another slab or obstructions in the fill, which may provide difficulty in excavating for the proposed beach access stair. Furthermore, deleterious materials may also be present within the footprint of the proposed basement stair excavation. Where excavated material is disposed of offsite, a waste classification will need to be completed, as discussed in Section 6.

The surrounding buildings and structures are likely to be founded on high level footings within the poorer quality sands and will be sensitive to vibration. Consequently, we recommend that tracking of hydraulic excavators or other tracked plant be carried out with caution. Sudden stop-start movements or impacts may result in ground vibration damage to the neighbouring buildings and structures. In this regard caution must be taken during the demolition of the structures with particular care taken not to allow walls and other parts of the structure to drop to and impact the ground or where percussive demolition techniques (such as rock hammers) are used.

The magnitude of transmitted vibrations generated will depend on the type and size of plant/equipment used and how it is used, the experience of operator, etc. The sensitivity of nearby structures to these generated vibrations will depend on the magnitude of the vibrations, the sensitivity of the structure itself to vibration and the set-back of the structures from the source of the vibration. Vibration monitoring should, as a minimum, be undertaken at the commencement of demolition and during initial tracking of plant/equipment to confirm that potentially damaging transmitted vibrations are not occurring. Whether further monitoring will be required will depend on the results of that monitoring. If concerns are raised that transmitted vibrations are potentially damaging nearby structures, works should cease until an assessment can be made by the geotechnical and structural engineer or vibration specialist. A set of Vibration Emission Design Goals are attached for guidance, although it should be noted that these goals only consider the impact of the vibrations on the structure itself. They do not consider the potential induced settlement of the sand below structures that may occur as a consequence of transmitted vibrations. This potential impact must be considered in addition to the potential impact of vibrations on the structure itself.

Prior to commencing construction, we recommend that detailed dilapidation surveys be carried out on the neighbouring buildings and structures to the east (No. 11 Monash Crescent) and west (No. 13 Monash Crescent). The owners of the respective properties should be provided with a copy of the reports and asked to confirm, in writing that the dilapidation reports present a fair and accurate record of the existing condition of the adjoining structures. The dilapidation reports may then be used as a benchmark against which to assess possible future damage claims as a result of the works. In this way, the builder is protected from spurious claims of construction related damage for damage that existed prior to the commencement of works.

5.2 Basement Staircase

We understand that as part of the proposed works a new staircase to the basement will be constructed. This will require infilling of the existing basement wall at the location of the existing stair, including covering of the stair at ground floor level, and demolition of another section of wall to allow the new staircase to provide access to the basement, which has a finished floor level of RL0.43m. The demolished section of the basement wall will only extend down to RL0.755m with a plinth remaining extending above the mean high water spring tide as discussed below.

Groundwater was encountered in BH1 at about RL0.4m, however we anticipate that groundwater levels below the site will be tidal given its proximity to Middle Harbour and the presence of highly permeable sands. Consequently, consideration should be given to the following sea levels when assessing potential groundwater levels that may impact the site both during construction and for the long term design of the structure:

- Mean sea level: RL0.067m
- Mean high water spring tides: RL0.696m
- Highest recorded tide (May 1974): RL1.475m

In addition, consideration should also be given to the potential impact of storm surges and climate change related sea level rises, for which expert advice should be obtained. As a preliminary guide, global sea levels are anticipated to increase by 45cm to 88cm by 2090 (Ref: CoastAdapt, Information Manual 2, National Climate Change, 2016), although it is likely that local variations will occur along the coastline.

Based on the above groundwater levels, we assume the existing basement is a water-tight 'tanked' structure and that it is not being maintained in a dry state by ongoing pumping and dewatering. The construction of the new staircase will compromise the water-tightness of the structure. Therefore the construction must be undertaken carefully and methodically to prevent flooding of the existing basement and to ensure the long-term water-tightness of the structure. Whilst the most appropriate construction methods and sequencing will require input from appropriate contractors and the structural engineer, the following construction options may be considered with sequencing anticipated to incorporate the following:

- Construct a watertight retaining wall around the proposed staircase excavation. While this may comprise a conventional secant pile wall, jet grout wall or similar, it must be noted that the most suitable wall type will depend on the equipment that can be established to site considering the access constraints as it is proposed to keep most of the house. It must also be noted that should a secant pile wall be adopted, a gap will be left between the end of the secant pile wall and the basement wall and that this must be sealed to make the wall watertight. In this regard, jet grouting or similar would need to be completed to form this seal.
- A contiguous pile wall by itself is not suitable for this site due to the presence of sand and groundwater and the gaps that exist between the piles which allow water to flow between the piles should levels extend above the mean high water spring tide level. It may be possible to infill gaps between contiguous piles in a 'dry' state during times outside of high tide, however this would require careful planning and there is risk that inflow of groundwater and sand could still occur, albeit it is expected to be of a minor extent. Regardless, this could impact surrounding structures by causing surface settlement. Otherwise, jet grouting or similar could be used to fill the gaps between contiguous piles and where this approach is adopted, it could be suitable for this site. Sheet piles would not be suitable due to the vibrations during installation that would detrimentally impact the existing house.
- Irrespective of the type of wall adopted, it must extend a sufficient depth below Bulk Excavation Level (BEL) to satisfy stability criteria, limit deflections such that the existing structure is not adversely impacted and prevent boiling or liquefaction of the sands. Internal propping of the wall is unlikely to be required but if necessary would limit wall deflections, as discussed further below. Temporary anchors are unlikely to be feasible as they will likely extend beyond the site boundaries and would need to be installed in quite confined conditions;
- Excavate for the proposed stair;
- To maintain a 'dry' excavation, the demolished section of the existing basement wall will be demolished to RL0.755m with a plinth remaining extending above the mean high water spring tides. As such, we anticipate the excavation and basement will mostly stay 'dry' during the works, although occasionally the high tide level may extend higher than the mean level, given the highest recorded high tide is at RL1.475m, resulting in inflow into the basement. However, where this occurs, it will be transient only, not occur during every high tide and is expected to be relatively small volumes that

should be easily managed by a conventional sump and pump system, but should be further assessed as design progresses. Regardless, the adoption of the plinth will allow for the excavation and construction to occur in a 'dry' state the majority of the time.

- Construct the proposed stair, with careful attention to the connection between the proposed stair and the existing basement wall to ensure water-tightness. The stair should be designed for appropriate hydrostatic uplift pressures based on the tidal ranges and any other relevant standards or requirements.
- The existing staircase will not be demolished and will be left in the ground with staircase entrance sealed and stairs within the basement demolished. It is assumed that the whole of the basement footprint has been constructed at RL0.43m. This must be confirmed at an early stage as should the basement need to be lowered in its southern corner the construction methodology discussed above will also need to be adopted for this portion of the site.

For the design of the watertight retaining wall, where a piled wall will be adopted, we recommend that Continuous Flight Auger (CFA) piles be adopted. Bored piles are not suitable for this site. Consideration could also be given piles formed using jet grouting techniques. Irrespective of the approach adopted, further advice will be required from specialist contractors on the feasibility of establishing equipment to site and the benefits, drawbacks and risks associated with the various options considered.

For cantilevered retaining walls supporting soil materials comprising of very loose to loose sands, we recommend that walls can be designed based on a triangular earth pressure distribution on the basis of an active earth pressure co-efficient (K_a) of 0.40 and a passive earth pressure co-efficient (K_p) of 2.56 (although as appreciable deflections are required to mobilise full passive pressure we recommend that a FOS of 2 be adopted and a K_p of 1.3 be adopted), where some wall movements are tolerable and assuming a horizontal backfill surface. If deflections need to be limited to reduce the risk of damaging nearby structures, an 'at rest' earth pressure coefficient (K_o) of 0.56 should be adopted. For the passive earth pressure, any localised excavations in front of the wall must be taken into consideration.

A bulk unit weight of 20kN/m^3 above the groundwater level and 10kN/m^3 below groundwater level should be adopted for the soil profile on the active side while a bulk unit weight of 8kN/m^3 should be adopted on the passive side for soils below the water table. Surcharge loads and hydrostatic pressures are additional to the above earth pressure recommendations. We recommend adopting a hydrostatic level 0.5m higher than the highest recorded tide as noted above to include some redundancy to account for climatic variations or storm surges.

If there is preference for a propped wall due to excessive deflections and the risk of damage to nearby structures, services or pavements, the walls may be designed based on a trapezoidal earth pressure distribution of $8H\text{ kPa}$, where H is the retained height of soils. Appropriate surcharge loads (such as adjoining buildings, traffic, sloping backfill, footing loads etc) are additional to the above earth pressures and should be allowed for in the design. The additional earth pressures from surcharge loads may be calculated using an 'at rest' earth pressure coefficient of 0.56.

We note there will be a number of existing footings immediately behind or within the zone of influence of the proposed stair excavation. Due to the difficulty and destructive nature of the works, we were unable to excavate test pits to expose the existing footings. In the worst case, the footings are high level footings founded on the upper very loose to loose sands. If available, as-built drawings for the existing house should be obtained, as well as investigation of the existing footings once access is available. As mentioned above, internal propping may be required in order to limit wall deflections. Regardless, we recommend test pits are excavated prior to expose the existing founding conditions to check the adopted retaining wall is suitable for the conditions. We recommend these additional works be undertaken as early as possible within the design phase. Given these additional investigation may be difficult to complete, the works could be undertaken at commencement of construction, noting however that this may lead to site delays as the design is re-assessed based on the additional data obtained.

5.3 Footings

Due to the presence of apparently uncontrolled fill that extends to depths in excess of 0.8m, the site classifies as a 'Class P' site in accordance with AS2870-2011. However, where footings are founded below the fill on the natural sands the structure may be designed in accordance with the recommendations for a Class A site.

We expect new footings will be required for the proposed garage, basement stair and rear beach access stair. We expect that piles will be required to accommodate the structural loads, particularly when considering the potential risk posed by scour during storm events. CFA piles or screw piles are considered suitable for this site. Where scour is not a consideration, pad and strip footings or a stiffened raft slab could be adopted.

Where piles are adopted the allowable bearing pressure (ABP) will depend on the pile diameter, founding depth, density and effective unit weight of the sand, number of piles and spacing of piles in pile groups, etc. Notwithstanding this, to provide some guidance on potential ABP's that may be achieved, a 0.5m diameter pile founded at a depth of 3.5m in sand of medium relative density may be designed for an ABP of 700kPa. An allowable skin resistance of 10kPa (compression) and 5kPa (tension) where piles have a minimum depth of 3.5m. Where screw piles are adopted no skin resistance may be adopted. Where piles are proposed to be installed to shallower depths, the ABP and shaft adhesion will be lower and further advice must be sought from this office. Total settlements are expected to be less than 10mm.

The design of pad and strip footings or stiffened raft slabs will depend on the width of the footing, the footing embedment depth, the relative density of the materials on which they are founded, tolerable settlements, etc. As a preliminary guide, footings of at least 0.5m width and an embedment depth of 0.8m founded in natural sands of very loose relative density may be designed for an allowable bearing pressure (ABP) of 60kPa. Maximum total settlements for a strip footing with a width of no greater than 0.5m and a length no longer 23m are anticipated to be about 5mm to 15mm. Differential settlements are anticipated to be roughly half the total settlements. Following the completion of the conceptual footing design this office should be contacted for further advice on footing design.

Pile drilling should be inspected by a geotechnical engineer to confirm drill depths have been satisfied and that the materials encountered are those that are expected, although we note screw piles are typically

installed on a design and construct basis and should be certified by the contractor. Should pad/strip footings be adopted, all footings should be inspected and tested by the geotechnical engineer to confirm that the design ABP's have been achieved prior to placing steel and pouring concrete. Testing is anticipated to comprise the completion of a number of DCP tests.

5.4 Suspended Floors, Slabs-On-Grade Construction and Earthworks

Should the proposed basement staircase and garage be fully suspended and supported on piles, no specific subgrade preparation is required other than the stripping of any grass and topsoil.

Where it is proposed to adopt slabs on grade for the garage or basement staircase, there is the risk that some differential settlement and cracking of the floor slabs may occur due to the presence of fill of variable depth and compaction. If this option is adopted the following subgrade preparation is recommended to help reduce the risk of poor performance of the slabs.

Prior to the placement of engineered fill or slabs-on-grade, we recommend the following site and subgrade preparation be completed:

- All grass, topsoil, and any other root affected soils should be stripped from site.
- Following site stripping, the exposed subgrade should be proof rolled with at least six passes of a five tonne minimum deadweight smooth drum roller. The final pass of proof rolling should be carried out in the presence of an experienced geotechnical engineer or geotechnician. The purpose of proof rolling is to increase the near surface density of the subgrade and to identify any soft or unstable areas. It may be necessary to use a confining layer of gravel (such as a DGB) at the surface to reduce the risk of shearing of near surface soils and the subsequent "boggling" of the roller.
- Where unstable spots are identified they should be excavated down to a sound base and replaced with engineered fill. Further advice will be required from the geotechnical engineer should unstable areas be encountered.
- Care must be taken rolling under vibration does not cause the generation of damaging transmitted vibrations. Where vibratory rolling is completed, vibration monitoring must be completed for the duration of the works. Reference should be made to *Section 5.1 Site Preparation and Excavation* for further advice on vibration monitoring.

Alternatively, where greater certainty of performance is required, all fill should be removed, a natural subgrade exposed, the subgrade preparation detailed above closely followed and engineered fill, as detailed below, placed to achieve design subgrade levels. It should be noted that where this approach is adopted fill may require removal to depths in the order of about 1.5m. Consequently, care must be taken that removal of fill to these depths does not result in the loss of material from below adjoining properties or structures and that support is provided to adjoining footings at all times. Where space allows, temporary batters may be formed at no steeper than 1Vertical (V):1.75 Horizontal (H), although some slumping of the batters may still occur, particularly during rainfall periods. Where structures are located within three times the proposed excavation height or space does not allow for the formation of batters, temporary support must be installed

prior to the commencement of excavation. If consideration is given to this approach further advice must be sought from this office.

Where slabs-on-grade are adopted they must be separated from all walls, columns, footings, etc, to permit relative movement. Joints in the on-grade floor slab should incorporate dowels or keys to allow transfer of shear forces but not bending moments.

Engineered Fill

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as the existing sands on site, which are free from deleterious substances and have a maximum particle size not exceeding one third the loose layer thickness. Fill should be compacted in layers of approximately 200mm loose thickness, although layer thickness may be varied depending on the size of compaction equipment adopted provided the full layer thickness is compacted to the required density. Where sand is used as engineered fill it should be compacted to a minimum density index of 75%. Should alternative materials be proposed for use as engineered fill, further advice should be sought from this office on the suitability and earthworks specification required for these materials.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m² or three tests per visit, whichever requires the most tests. We recommend that Level 1 compaction control be undertaken where the fill will support structures or at least Level 2 compaction control where it will not. Reference should be made to AS3798-2007 (or latest standard at the time of testing) and the requirements of this standard adhered to. We can complete the abovementioned testing and supervision if required.

5.5 Trafficable Slab-On-Grade

Prior to the placement of trafficable slabs-on-grade or pavements, we recommend that the recommendations provided above in Section 5.4 be closely followed.

For the design of slabs on grade that will be trafficked, a modulus of subgrade reaction of 40kPa/mm (based on a 760mm diameter plate) may be adopted. The concrete on-grade floor slab should be separated from all walls, columns, footings, etc., to permit relative movement. Joints in the concrete on-grade floor slab should incorporate dowels or keys. The slab should have a sub-base layer of at least 100mm thickness of crushed rock to TfNSW QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% Standard Maximum Dry Density (SMDD). The subbase will provide a more stable working platform, will provide more uniform slab support and will reduce 'pumping' of 'fines' at joints.

5.6 Existing Sea Wall at Rear Patio

The existing seawall is understood to be about 50 years old and shows no signs of distress in the form of cracking, outward rotation or bowing. Notwithstanding this, with climate change the wall may be subjected to a more aggressive energy environment which may result in additional scour and loss of support below the wall. This has been further assessed in the Horton Coastal Engineering report who discuss that wave overtopping does not pose an issue for the seawall, however the seawall stability should be checked by a structural engineer based on an estimated scour level at RL-0.8m. Should this pose an issue, and as it will be less disruptive and better access to the wall will be available during the works on site, it would be an opportune time to rebuild or strengthen the wall such that it satisfies current codes for the life of the proposed site development and the structural engineer is satisfied by the wall stability based on the above scour level. Prior to rebuilding or strengthening the wall we recommend that further investigation be completed to determine the dimensions of the wall, in particular the reduced level of the toe of the wall. The scope of any further investigation should be determined in consultation with the structural/coastal engineer assessing the walls suitability.

Based on our observations, the wall appears in good condition with no obvious signs of instability. However, where it is determined by the structural engineer that the wall has an unsatisfactory design life, a number of options exist. Should the structural engineer be satisfied by the wall stability, one option would be to develop an inspection program whereby at defined intervals the seawall is inspected jointly by the structural and coastal engineers, and perhaps by a geotechnical engineer if deemed required, who can then advise on the wall condition and whether any works on the walls are warranted.

Where there are concerns regarding the strength or stability of the wall and it is simply proposed to remediate the existing wall, consideration could be given to the use of shotcrete, mesh and soil nails. However, it should be noted that this will not result in the deepening of the toe of the wall. Should scour below the toe of the wall be a potential mechanism for failure, the toe of the wall must also be deepened or protected in some manner. Deepening is likely to be difficult, although jet grouting could be considered although further advice will be required from specialist contractors on whether such an approach will achieve the desired effect.

A second option would be, where CFA or jet grout piles are installed as part of the works, a secant or jet grout pile wall could be installed behind the existing wall. While this approach will not prevent the failure of the existing wall it will prevent the loss of materials from behind the wall. Consideration must also be given to how far the secant pile wall will need to be extended along the eastern and western sides of the property to provide certainty that erosion behind the wall will not occur.

Alternatively, consideration could be given to demolishing and rebuilding the wall. However, while this approach would allow the new wall to be constructed on the site boundary, demolition of the existing wall will result in the loss of material from behind the wall (which will require battering) and potentially from below the properties to both the east and west that currently rely on the existing seawall to prevent materials from being lost at the end of their seawalls. Consequently, further investigation will be required to determine

whether temporary shoring will be required along the eastern and western site boundaries and, if required, this must be installed prior to the demolition of the existing seawall.

5.7 Soil Aggression

The results of the pH, chloride, sulphate and resistivity tests indicate that the soils pose a 'Mild' aggression environment to buried concrete structures and are 'Mild' to buried steel structures in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009. Consideration must also be given to the direct exposure to sea water that some elements may be subjected to and in this case the exposure classification for concrete elements is 'Severe' in accordance with Table 6.4.2(A) of AS2159-2009 while for steel elements it is 'Very severe' in accordance with Table 6.5.2(A) of AS2159-2009.

5.8 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:

- Investigation of existing footings.
- Review of proposed basement staircase design.
- Dilapidation surveys of No. 11 Monash Crescent, and of No. 13 Monash Crescent, if considered necessary.
- Vibration monitoring during demolition, tracking of machinery and vibratory compaction.
- Geotechnical review of footing design, where required.
- Geotechnical advice regarding stabilising the existing sea wall.
- Proof roll of subgrade prior to the placement of engineered fill and slabs on grade.
- Inspection of all footings or piling drilling/final depths to confirm that the design ABP's have been achieved.
- Density testing of all fill placed as engineered fill.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project. In the event that any of the advice presented in this report is 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications

and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

The subsurface conditions between 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.

TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A	B
	Existing Sea Wall	Basement Staircase Retaining Wall
Assessed Likelihood	Unlikely	Unlikely
Assessed Consequence	Minor	Medium
Risk	Low	Low
Comments	-	

***Property Value Assumed to be \$7-\$8 million (Ref: www.onthefhouse.com.au, 14 January 2021)**

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A	B
	Existing Sea Wall	Basement Staircase Retaining Wall
Assessed Likelihood	Unlikely	Unlikely
Indicative Annual Probability	10^{-4}	10^{-4}
Duration of Use of area Affected (Temporal Probability)	Above, 0.25hr/day (0.0104) Below, 1hr/day x 3 months per year (sitting/laying) + 1 minute/day (walking) (0.0111)	6hrs/day, 6 days a week (0.21)
Probability of not Evacuating Area Affected	Above, 0.8 Below, 0.5	Above, 0.5 Below 1.0
Spatial Probability	Above and Below, 3m length fails, 3m/12m (0.25)	0.5
Vulnerability to Life if Failure Occurs Whilst Person Present	Above, 0.1 (Ride Down) Below, 0.5 (Crushed)	Above, 0.1 (Ride Down) Below, 0.9 (Buried)
Risk for Person most at Risk	Above, 2.08×10^{-8} Below, 6.94×10^{-8}	Above, 5.25×10^{-7} Below, 9.45×10^{-6}
(Total Risk)	9.0×10^{-8}	1.0×10^{-7}

CERTIFICATE OF ANALYSIS 384873

Client Details

Client	JK Geotechnics
Attention	Mohammad Mahmoud
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	<u>37642YF, 11A Moash Crescent, Clontarf, NSW</u>
Number of Samples	3 Soil
Date samples received	02/07/2025
Date completed instructions received	02/07/2025

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	09/07/2025
Date of Issue	09/07/2025
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. Tests not covered by NATA are denoted with *	

Results Approved By

Nick Sarlamis, Assistant Operation Manager

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		384873-1	384873-2	384873-3
Your Reference	UNITS	BH1	BH1	BH1
Depth		0.5-0.7	1.5-1.7	2.5-2.6
Date Sampled		02/07/2025	02/07/2025	02/07/2025
Type of sample		Soil	Soil	Soil
Date prepared	-	04/07/2025	04/07/2025	04/07/2025
Date analysed	-	04/07/2025	04/07/2025	04/07/2025
pH 1:5 soil:water	pH Units	6.5	6.7	7.1
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	85	<10	10
Resistivity in soil*	ohm m	35	190	100

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate			Spike Recovery %		
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			04/07/2025	[NT]	[NT]	[NT]	[NT]	04/07/2025	[NT]
Date analysed	-			04/07/2025	[NT]	[NT]	[NT]	[NT]	04/07/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	89	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	97	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Air volumes are typically provided by customers (often as flow rate(s) and sampling time(s) and/or simply volumes) sampled or exposure times (determines 'volume' passive badges are exposed to)). Hence in such circumstances the volume measurement is inevitably not covered by Envirolab's NATA accreditation. An exception may occur where Envirolab Newcastle does the sampling where accreditation exists for certain types of sampling and hence volume determination(s). Note air volumes are often used to determine concentrations for dust and/or analyses on filters, sorbents and in impingers. For canister sampling, the air volume is covered by Envirolab's NATA accreditation.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

For Dust Deposit Gauge (DDG) analysis the sampling, sampling period and funnel exposure area do not fall under Envirolab's NATA accreditation (unless the Newcastle laboratory where responsible for the sampling), hence the annotation on the DDG units of reporting.

Urine Analysis - The BEI values listed are taken from the 2022 edition of "TLVs and BEIs Threshold Limits" by ACGIH.

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BOREHOLE LOG



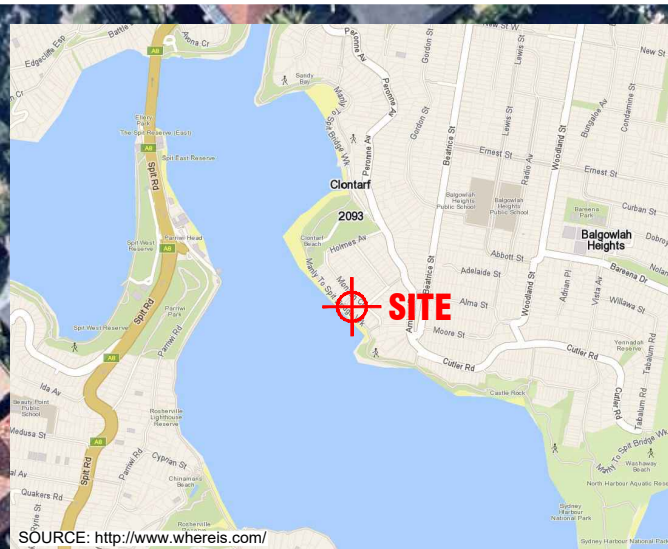
Borehole No.
201

1/1

Client: COURTNEY SMITH												
Project: PROPOSED ALTERATIONS AND ADDITIONS												
Location: 11A MONASH CRESCENT, CLONTARF, NSW												
Job No.: 37642YF												
Method: HAND AUGER												
R.L. Surface: ≈ 2.9m												
Date: 25/6/25												
Datum: AHD												
Plant Type: -												
Logged/Checked by: M.M./O.F.												
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB									
DRY ON COMPLETION				REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, with clay fines, trace of fine to coarse grained sandstone, concrete fragments and root fibres.	M			APPEARS POORLY COMPACTED
					0.5		FILL: Silty clay, low to medium plasticity, dark brown, trace of fine to coarse grained sandstone gravel, with fine grained sand.	w>PL				
					1		FILL: Silty sand, fine to medium grained, dark grey and brown.	M				
					1.5	SM	Silty SAND: fine to medium grained, yellow brown.	M		MARINE / ESTUARINE		
					2							
					2.5		SP	SAND: fine to medium grained, yellow brown, trace of shell fragments and silt.	W			
					3			END OF BOREHOLE AT 3.0m				
					3.5							

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	COURTNEY SMITH						
Project:	PROPOSED ALTERATIONS AND ADDITIONS						
Location:	11A MONASH CRESCENT, CLONTARF, NSW						
Job No.	37642YF			Hammer Weight & Drop: 9kg/510mm			
Date:	25-6-25			Rod Diameter: 16mm			
Tested By:	M.M.			Point Diameter: 20mm			
Test Location	201	202	203	Test Location	201	202	
Surface RL	≈2.9m	≈2.9m	≈3.0m	Surface RL	≈2.9m	≈2.9m	
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	DRILLED	3000-3100	10	6	
100 - 200			8/50mm	3100-3200	9	7	
200 - 300		↓	REFUSAL	3200-3300	12	8	
300 - 400		1		3300-3400	12	11	
400 - 500	↓	↓		3400-3500	12	12	
500 - 600	1	1		3500-3600	13	12	
600 - 700	2	↓		3600-3700	16	12	
700 - 800	1	2		3700-3800	13	12	
800 - 900	1	2		3800-3900	15	9	
900 - 1000	1	2		3900-4000	16	9	
1000 - 1100	1	1		4000-4100	REFUSAL	REFUSAL	
1100 - 1200	1	↓		4100-4200			
1200 - 1300	1	1		4200-4300			
1300 - 1400	1	↓		4300-4400			
1400 - 1500	1	1		4400-4500			
1500 - 1600	1	↓		4500-4600			
1600 - 1700	2	1		4600-4700			
1700 - 1800	1	1		4700-4800			
1800 - 1900	2	1		4800-4900			
1900 - 2000	1	1		4900-5000			
2000 - 2100	1	1		5000-5100			
2100 - 2200	2	2		5100-5200			
2200 - 2300	1	3		5200-5300			
2300 - 2400	2	4		5300-5400			
2400 - 2500	3	3		5400-5500			
2500 - 2600	4	3		5500-5600			
2600 - 2700	5	6		5600-5700			
2700 - 2800	6	8		5700-5800			
2800 - 2900	6	7		5800-5900			
2900 - 3000	7	7		5900-6000			
Remarks:	1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013) 2. Usually 8 blows per 20mm is taken as refusal 3. Datum of levels is AHD 4. DCP1 Seepage at 3.4m						



SOURCE: <http://www.wherereis.com/>



Nearmap

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title:

SITE LOCATION PLAN

Location:

11A MONASH CRESCENT, CLONTARF, NSW

Report No:

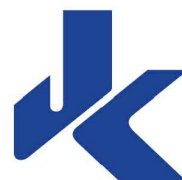
37642YF

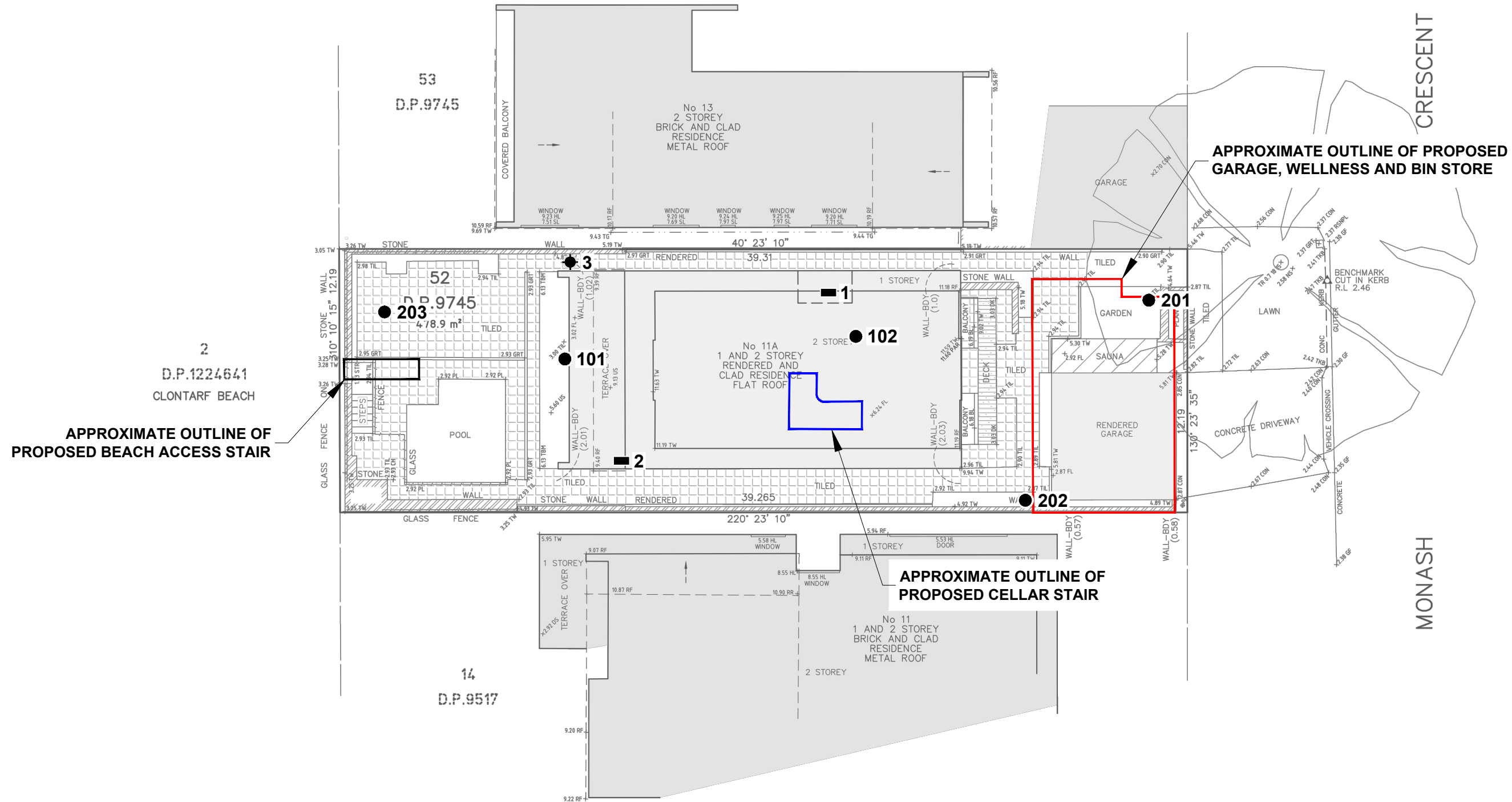
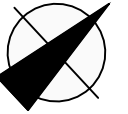
Figure No:

1

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

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- LEGEND**
- BOREHOLE
 - ⊕ BOREHOLE AND DCP TEST
 - TEST PIT

- NOTES:**
1. TEST LOCATIONS 1, 2 AND 3 ARE FROM OUR PREVIOUS 2002 INVESTIGATION (APPROXIMATE).
 2. TEST LOCATIONS 101 AND 102 AND ARE FROM OUR PREVIOUS 2003 INVESTIGATION (APPROXIMATE).
 3. BOREHOLES 201, 202 AND 203 ARE FROM OUR CURRENT GEOTECHNICAL INVESTIGATION.

 This plan should be read in conjunction with the JK Geotechnics report.	Title: INVESTIGATION LOCATION PLAN		
	Location: 11A MONASH CRESCENT, CLONTARF, NSW		
	Report No: 37642YF	Figure No: 2	
	JKGeotechnics		

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite ‘safe’, depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are ‘safe limits’, up to which no damage due to vibration effects has been observed for the particular class of building. ‘Damage’ is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the ‘safe limits’, then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the ‘safe limits’ are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

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:

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

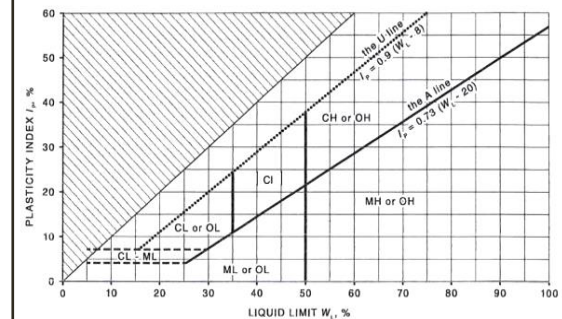
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

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
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	–	–	–	–

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

**LANDSLIDE RISK
MANAGEMENT
TERMINOLOGY**

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
Consequence	The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
Hazard	A condition with the potential for causing an undesirable consequence (the landslide). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.
Individual Risk to Life	The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
Landslide Intensity	A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. These are two main interpretations: (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability (continued)	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

NOTE: Reference should be made to Figure A1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

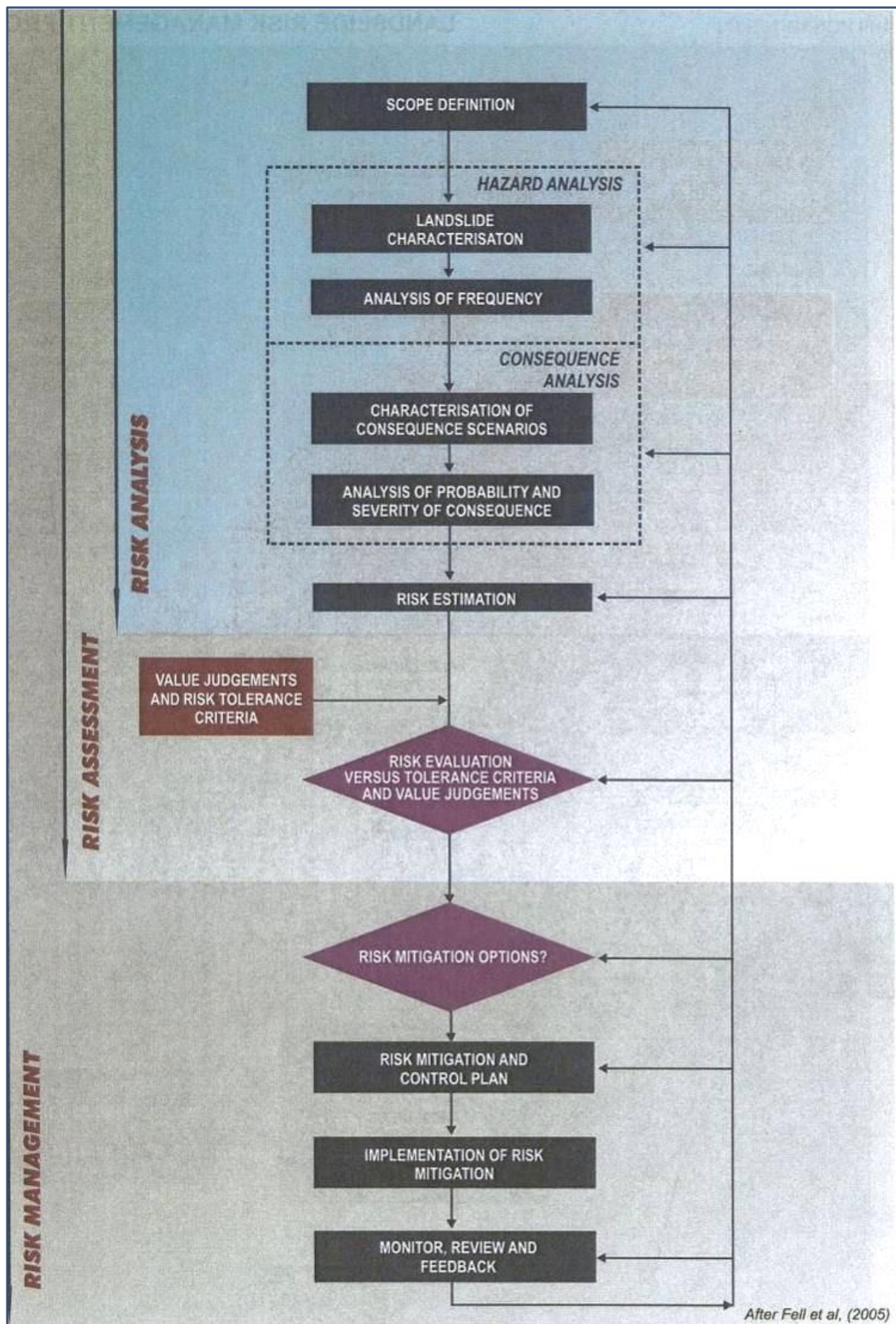


FIGURE A1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10 ⁻¹	5×10 ⁻²	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10 ⁻²		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10 ⁻³	5×10 ⁻³	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10 ⁻⁴	5×10 ⁻⁴	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10 ⁻⁵	5×10 ⁻⁵	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10 ⁻⁶	5×10 ⁻²	1,000,000 years	200,000 years	The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%		Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	40%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%	1%	Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not *vice versa*.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (continued)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	1: CATASTROPHIC 200%	2: MAJOR 60%	3: MEDIUM 20%	4: MINOR 5%	5: INSIGNIFICANT 0.5%
A – ALMOST CERTAIN	10 ⁻¹	VH	VH	VH	H	M or L (5)
B – LIKELY	10 ⁻²	VH	VH	H	M	L
C – POSSIBLE	10 ⁻³	VH	H	M	M	VL
D – UNLIKELY	10 ⁻⁴	H	M	L	L	VL
E – RARE	10 ⁻⁵	M	L	L	VL	VL
F – BARELY CREDIBLE	10 ⁻⁶	L	VL	VL	VL	VL

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book “Guideline Document Landslide Hazards” published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board’s website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both “potential” and “actual” landslides must be taken very seriously. They present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with serious consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can “run-out” from above, “regress” from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else’s land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

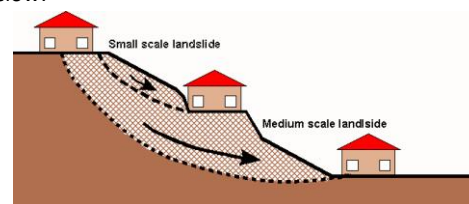


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

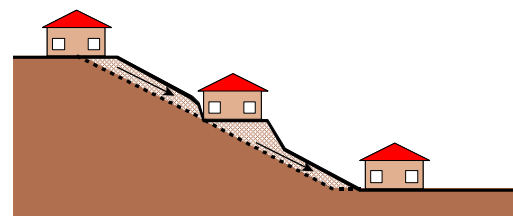


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

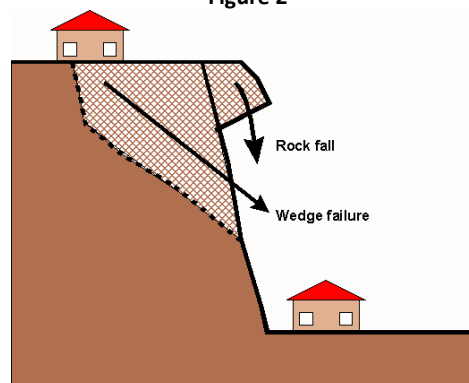


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

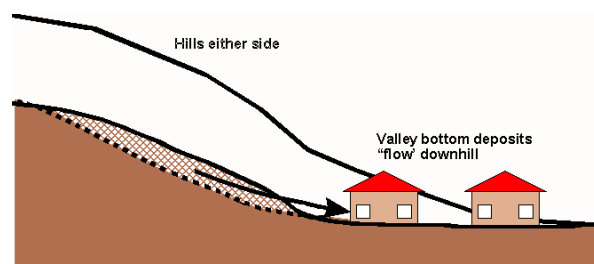


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the [Australian Geomechanics Society](#), a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific **"landslide hazard zones"**. Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment

for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of the repairs and temporary loss of use if the landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2 – LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable risk level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	Usually acceptable to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	Acceptable. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	Activity/Event Leading to Death (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding, ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

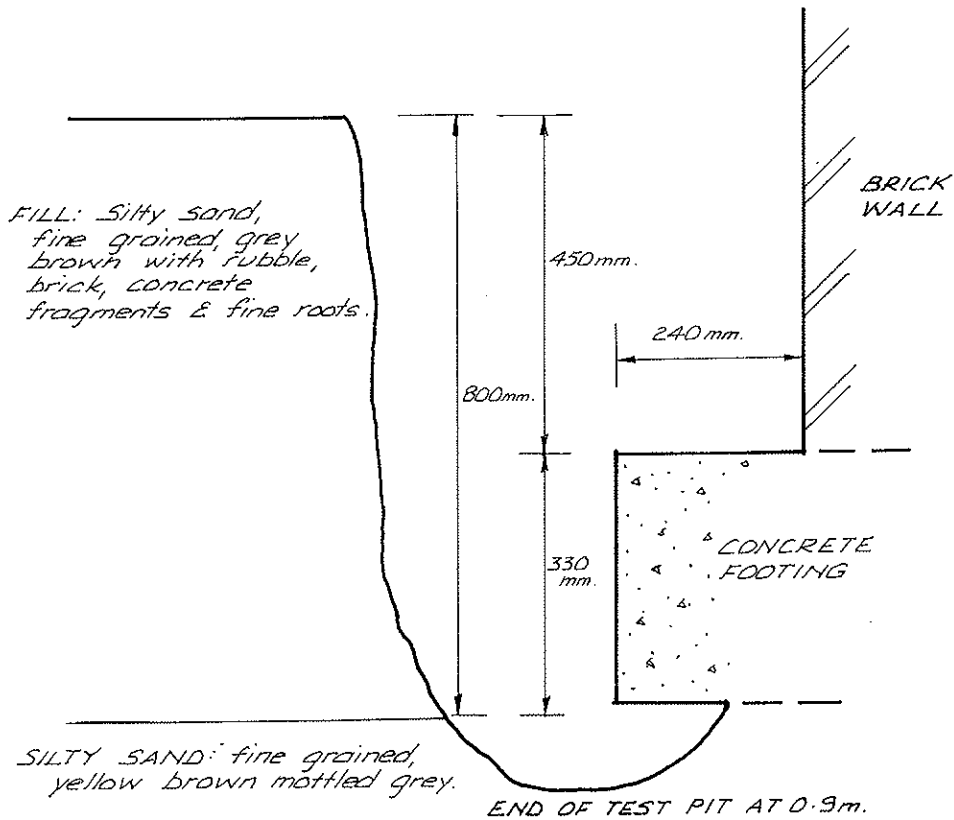
More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.



APPENDIX B



TEST PIT 1

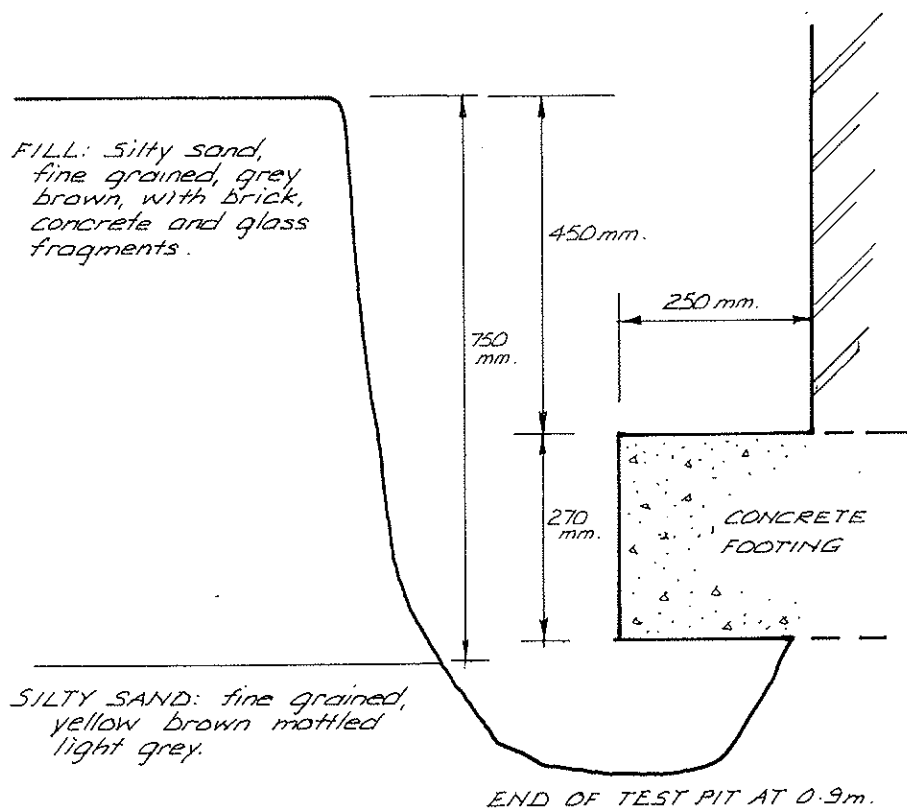
SCALE



Jeffery & Katauskas Pty Ltd



Report No. 16894V Figure No. 2



TEST PIT 2

SCALE

0 0.5m

Jeffery & Katauskas Pty Ltd



Report No. 16894V Figure No. 3



Borehole No.

1 1/1

BOREHOLE LOG

Client: MR AND MRS F PARTRIDGE
Project: PROPOSED ADDITIONS TO HOUSE
Location: 11A MONASH CRESCENT, CLONTARF. NSW

Job No. 16894V **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 19-6-02 **Datum:**
Logged/Checked by: Y.N./y

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
						0			REFER TO TEST PIT 1 SECTION				
						0.5							
						1		SM	SILTY SAND: fine grained, yellow brown mottled grey.	M	VL	-	COMMENCE HAND AUGERING
						1.5					L		
						2					MD		
						2.5				W			
						3			END OF BOREHOLE AT 3.0m				
						3.5							

AFTER 30 MINS



Borehole No.

2 1/1

BOREHOLE LOG

Client: MR AND MRS F PARTRIDGE
Project: PROPOSED ADDITIONS TO HOUSE
Location: 11A MONASH CRESCENT, CLONTARF. NSW

Job No. 16894V **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 19-6-02 **Datum:**
Logged/Checked by: Y.N. *YN*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/Weathering	Strength/Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB	DS									
						0			REFER TO TEST PIT 2 SECTION				
						0.5							
					REFER TO DCP TEST RESULTS	1		SM	SILTY SAND: fine grained, yellow brown mottled light grey.	M	L		COMMENCE HAND AUGERING
						1.5							
						2							
						2.5			as above, but with ironstone gravel.	W	MD		
						3			END OF TEST PIT AT 2.9m				CONTINUOUS CAVE-IN
						3.5							

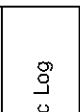
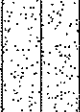
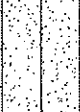
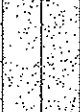
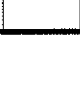
Borehole No.

3 1/1

BOREHOLE LOG

Client: MR AND MRS F PARTRIDGE
Project: PROPOSED ADDITIONS TO HOUSE
Location: 11A MONASH CRESCENT, CLONTARF. NSW

Job No. 16894V **Method:** HAND AUGER **R.L. Surface:** N/A
Date: 19-6-02 **Datum:**
Logged/Checked by: Y.N./*Y*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	USO	DB	DS									
					REFER TO D C P TEST RESULTS	0			FILL: Silty sand, fine grained, grey brown, with brick and sandstone fragments.				APPEARS POORLY COMPACTED
						0.5							
						1		SM	SILTY SAND: fine grained, light yellow brown mottled light grey.	M	VL	-	-
						1.5							
						2							
						2.5			as above, but with cemented silty sand bands.				
						2.5			SILTY SAND: coarse grained, yellow brown and light grey, with shells.	W			
						3			END OF BOREHOLE AT 3.0m				
						3.5							



DYNAMIC CONE PENETRATION TEST RESULTS

Client:		MR AND MRS F PARTRIDGE					
Project:		PROPOSED ADDITIONS TO HOUSE					
Location:		11A MONASH CRESCENT, CLONTARF. NSW					
Job No.		16894V		Hammer Weight & Drop: 9kg/510mm			
Date:		19-6-02		Rod Diameter: 16mm			
Tested By:		Y.N.		Point Diameter: 20mm			
Number of Blows per 100mm Penetration							
Test Location				Test Location			
Depth (mm)	1	2	3	Depth (mm)	1		3
0 - 100	1	1	1	3000-3100	8		2
100 - 200		1	1	3100-3200	9		5
200 - 300				3200-3300	9		6
300 - 400	1	3	2	3300-3400	10		9
400 - 500		2		3400-3500	10		9
500 - 600	1	3		3500-3600	9		9
600 - 700		2		3600-3700	8		8
700 - 800		2		3700-3800	7		8
800 - 900	1	2		3800-3900	8		9
900 - 1000	1	2		3900-4000	9		9
1000 - 1100	2	2		4000-4100			
1100 - 1200	4	3		4100-4200			
1200 - 1300	4	3		4200-4300			
1300 - 1400	3	2		4300-4400			
1400 - 1500	3	2		4400-4500			
1500 - 1600	3	3		4500-4600			
1600 - 1700	3	2		4600-4700			
1700 - 1800	3	2		4700-4800			
1800 - 1900	3	3		4800-4900			
1900 - 2000	6	2		4900-5000			
2000 - 2100	4	2		5000-5100			
2100 - 2200	4	3		5100-5200			
2200 - 2300	4	3		5200-5300			
2300 - 2400	7	5		5300-5400			
2400 - 2500	6	7		5400-5500			
2500 - 2600	7	7		5500-5600			
2600 - 2700	6	7		5600-5700			
2700 - 2800	7	5		5700-5800			
2800 - 2900	7	5		5800-5900			
2900 - 3000	7	6		5900-6000			
Remarks:		1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2. 2. Usually 8 blows per 20mm is taken as refusal					



Borehole No.

101

1/2

BOREHOLE LOG

Client: MR & MRS PARTRIDGE
Project: PROPOSED NEW HOUSE
Location: 11A MONASH CRESCENT, CLONTARF, NSW

Job No. 16894V1

Method: SPIRAL AUGER
JK250

R.L. Surface: N/A

Date: 21-8-03

Datum:

Logged/Checked by: N.S. *[Signature]*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB	DS									
DRY ON COMPLETION						0			FILL: Sand, fine to medium grained, yellow, with a trace of silt.	D-M			APPEARS POORLY COMPACTED
					N = 4 2,2,2	1		SP	SAND: fine to medium grained, yellow, with a trace of silt.	M	VL	-	POSSIBLY FILL
ON COMPLETION					N = 5 2,2,3	2			SAND: fine to medium grained, yellow.	M	VL-L		
					Nc = 6 8 8	3			SAND: medium grained, yellow brown, with shell fragments.	W	MD		
					Nc = 5 8 9	4							
					Nc = 9 10 10	5							
						6							
						7							



Borehole No.

101

2/2

BOREHOLE LOG

Client: MR & MRS PARTRIDGE													
Project: PROPOSED NEW HOUSE													
Location: 11A MONASH CRESCENT, CLONTARF, NSW													
Job No. 16894V1			Method: SPIRAL AUGER JK250				R.L. Surface: N/A						
Date: 21-8-03			Datum:										
Logged/Checked by: N.S./ <i>[Signature]</i>													
Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
					Nc = 3 5 9	8			SAND: medium grained, yellow brown, with shell fragments.		MD		
					Nc = 4 5 7	9							
						10							
					Nc = 7 10 11	11							
					Nc = 12 13 19								
					Nc = 24 16 18								
					Nc = 19								
								12					
									END OF BOREHOLE AT 12.0m				
						13							
						14							



Borehole No.

102

1/2

BOREHOLE LOG

Client: MR & MRS PARTRIDGE
Project: PROPOSED NEW HOUSE
Location: 11A MONASH CRESCENT, CLONTARF, NSW

Job No. 16894V1

Method: SPIRAL AUGER
JK250

R.L. Surface: N/A

Date: 21-8-03

Datum:

Logged/Checked by: N.S./y

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB	DS									
DRY ON COMPLETION						0			FILL: Silty sand, fine to medium grained, grey brown, with brick fragments.	M			APPEARS POORLY COMPACTED
					N = 7 3,3,4			SM	SILTY SAND: fine to medium grained, pale brown grey.	M	L		POSSIBLY FILL
ON COMPLETION						1		SP	SAND: fine to medium grained, yellow to light brown, with a trace of silt.	M	VL		
					N = 5 3,3,2	2							
					5								
ON COMPLETION						3			SAND: medium grained, yellow brown, with shell fragments.	W	MD		
					N = 15 6,8,7	4							
						5					VL		
					N = 4 2,2,2	6							
						7					L		
					N = 9 3,4,5								



Borehole No.

102

2/2

BOREHOLE LOG

Client: MR & MRS PARTRIDGE
Project: PROPOSED NEW HOUSE
Location: 11A MONASH CRESCENT, CLONTARF, NSW

Job No. 16894V1

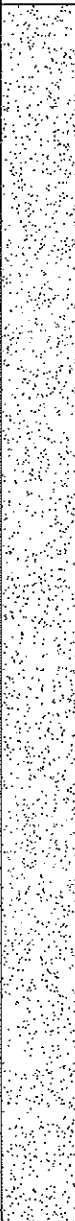
Method: SPIRAL AUGER
JK250

R.L. Surface: N/A

Date: 21-8-03

Datum:

Logged/Checked by: N.S./*nz*

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks			
	ES	US	DB	DS												
					N = 6 2,2,4	8		SP	SAND: medium grained, yellow brown, with shell fragments.		L					
						9										
						10					MD					
					Nc = <table><tr><td>5</td></tr><tr><td>9</td></tr><tr><td>11</td></tr></table>	5		9	11	11						
5																
9																
11																
						12										
						13										
						14			END OF BOREHOLE AT 13.5m							