REPORT TO

JOHN KOLENDA

ON

GEOTECHNICAL INVESTIGATION AND STABILITY ASSESSMENT

FOR

PROPOSED GARAGE AND SEAWALL

AT

15 MONASH CRESCENT, CLONTARF, NSW

Date: 11 March 2022
Ref: 32694YJrpt2

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1 INTRODUCTION

This report presents the results of a geotechnical investigation and stability assessment for the proposed garage and seawall at 15 Monash Crescent, Clontarf, NSW. The location of the site is shown in Figure 1. The report was commissioned by Mr John Kolenda by email on 17 February 2022 and was completed on the basis of our proposal (Ref: P50224YJRev1, dated 16 February 2022).

From the supplied architectural drawings by Mathieson Architects (Mathieson, Ref: Project No. 18007, Drawing Nos. DA.01 and DA.02, Revision A, dated 25 October 2021), we understand that the proposed works include:

- Alterations to the existing garage that will include the demolition of the existing pitched roof and rear portion of the garage building. Following demolition, the garage will be extended further to the south-west to form a double garage and storeroom. The finished floor level of the garage will be RL2.35m while the finished floor level of the storeroom will be RL2.70m. Some minor filling, about 0.1m to 0.2m may be required to achieve design storeroom levels.
- Construction of a new seawall and stairway at the rear of the site. The new seawall will be built on the south-western site boundary and will be located in front of the existing seawall. It is proposed to comprise a core filled blockwork wall supported on a grout injected contiguous pile wall. The gap between the two walls is shown to be backfilled.

The comments and recommendations presented in this report are based on the subsurface conditions encountered at six (6) test locations completed during a previous investigation of the site. Based on this information we have provided comments and recommendations on retention, excavation and hydrogeology, new footings, slabs on grade and earthworks, trafficable slabs on grade and soil aggression. We have also assessed the risk posed by slope instability of the site to both life and property.

A Coastal Engineering Risk Management report that has been prepared by Horton Coastal Engineering Pty Ltd (Horton, dated 19 January 2022) has been provided to us.

2 INVESTIGATION PROCEDURE

The subsurface investigation was carried out on 24 September 2019 and was limited by access constraints to the use of portable hand-held equipment. Prior to the commencement of the fieldwork, the proposed test locations were assessed with reference to ‘Dial Before You Dig’ plans and scanned for the presence of buried services by a specialist subcontractor such that they could be located clear of services. Our stability assessment was carried out on 25 February 2022.

The subsurface investigation comprised:

- Five (5) hand auger boreholes, BH1 to BH5, drilled to depths ranging from 0.55m to 2m. BH4 and BH5 encountered refusal at depths of 0.55m and 0.82m, respectively while the remaining boreholes were drilled to their target depths.
- Six (6) Dynamic Cone Penetration (DCP) tests, DCP1 to DCP6, were completed to depths ranging from 0.58m to 5.0m. DCP4 and DCP5 encountered refusal at depths of 0.51m and 0.72m, respectively while the remaining DCP's were tested to their target depth (DCP2, DCP3 and DCP6) or deeper (DCP1). The DCP tests were carried out adjacent to the boreholes (DCP1 to DCP5) and at one other location (DCP6).
- Completion of a stability assessment.

The borehole and DCP test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface RL's at the test locations were estimated by interpolation between spot heights shown on the survey drawing prepared by Bee & Lethbridge Pty Ltd (Ref: No. 3051, Drawing No. 3051A, dated 18 October 2018). 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 while the DCP tests were used to assess the apparent compaction of the fill and the relative density of the sands. The depth of refusal of the DCP tests can also be used to infer the depth to bedrock, although it should be noted that premature refusal may occur on inclusions in the fill or harder layers within the soils. In this instance we consider that the depth of refusal of the DCP tests does not represent the depth to bedrock but rather the presence of inclusions in the fill.

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. 227009 and summarised in Section 3.3.

Groundwater observations were made during drilling, on completion of drilling, and up to one hour after the completion of drilling. No longer term groundwater monitoring was completed.

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

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located in the flat, littoral zone between the steep clifflines that drop down from the ridgeline to the north and east and Middle Harbour to the south-west.

The site itself is generally flat. At the time of the investigation and stability assessment, the site included a brick garage and entry passage located along the north-eastern site boundary and a two-storey brick residential dwelling positioned over the central portion of the site. Both appeared in good condition when viewed externally. A concrete driveway runs between Monash Crescent and the garage. During our stability assessment we observed translational (hairline) cracking in the concrete driveway. Garden beds and medium height palm trees are present to the south-east and north-west of the driveway. Around the perimeter of the garage and house are garden beds, lawns and tiled pavements.

The site extends to Clontarf Beach which is located to the south-west. Along the south-western boundary the site is supported by an approximately 1.95m to 2.05m high composite retaining wall. The upper 0.45m height of the wall comprises bricks while the lower 1.5m to 1.6m height of the wall comprises sandstone masonry blockwork. The lower portion of the wall was raked back into the site at an angle of about 20° to 30˚ from vertical. A concrete footing was visible over a portion of the wall. Based on the Horton report, the footing is 0.6m thick. The wall appeared in good condition showing no signs of distress in the form of cracking, bulging or outward rotation.

Running along the south-eastern and north-western site boundaries are brick fences. The levels across the boundaries between the site and adjoining properties to the south-east and north-west are about the same, with exception to the south-western 3m of the north-western boundary, where a maximum 1.5m high sandstone block wall supports the site, which follows the alignment of the adjoining stairs down to the beach. To the north-west of the site is a two-storey residential building setback approximately 2m from the boundary that is newly constructed and appeared in good condition when viewed from the site. To the south-east of the site is a medium height hedge and grassed lane way that provides access to the beach. At the south-western end there is a sloping sandstone block sea wall which appeared to be in good condition. To the north-east is Monash Crescent, which is an asphaltic concrete surface road with concrete kerb and gutter that appeared in good condition.

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. The investigation revealed a generalised subsurface profile comprising silty sand fill overlying natural silty sand that was assessed to be of Aeolian origin. A summary of the investigation findings is presented below. For detailed results of materials encountered or DCP test results at particular locations, reference should be made to the attached borehole logs and DCP test results sheets.

Concrete Slabs/Pavements

As discussed above in Section 3.1 the site is currently developed and there are tile (probably overlying concrete) and concrete pavements. The thickness of these pavements was not determined.

Fill

Fill was encountered in all boreholes to depths ranging from 0.3m to at least 0.82m, at which depth BH5 refused whilst still in fill. BH4 also refused within the fill at a depth of 0.55m. The fill is likely to be deeper, particularly behind the existing retaining wall near the southern boundary where it possibly varies up to a depth of about 2.05m. The fill comprised silty sand and included traces of siltstone, sandstone and igneous gravel and roots and root fibres. The fill was assessed to be poorly compacted.

Aeolian Sand

Aeolian silty sand was encountered below the fill and extended to the termination depths (i.e. 2m) in BH1 to BH3. The sands were generally assessed to be very loose, very loose to loose and loose. Some loose to medium dense and medium dense sands were also encountered. Traces of root fibres were encountered in the upper portion of these sands. The density of the sands generally improved with depth and they were assessed to be of medium dense relative density below depths of 2.1m (DCP1), 2.5m (DCP2), 2.2m (DCP3) and 2.8m (DCP6).

Groundwater

Groundwater was not encountered in any of the boreholes during drilling, on completion or up to 1 hour after completion of drilling. We anticipate groundwater would be similar to that of the nearby sea level in Middle Harbour. Further discussions on groundwater are provided in the comments and recommendations sections of this report.

3.3 Laboratory Test Result

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. 227009. Section 4.7 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
BH1	0.4-0.6	Silty Sand	7.2	<10	<10	16,000
BH2	0.4-0.5	Silty Sand	7.6	<10	<10	35,000
BH3	0.35-0.45	Fill: Silty Sand	6.9	<10	<10	27,000

4 STABILITY ASSESSMENT

4.1 Landslide Risk Assessment Criteria

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 Management (Ref: Journal and News of the Australian Geomechanics Society, Volume 42, No 1, March 2007). In this regard an acceptable risk for loss of life of $1x10^{-6}$ has been adopted for new developments for the person most at risk. 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 the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management adopts a risk to property of low to be acceptable for new developments.

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.

4.2 Landslide Risk Assessment

4.2.1 Hazards

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

- **Hazard A** – Failure of the 1.95m to 2.05m high composite seawall retaining the rear yard.
- **Hazard B** - Failure of the 1.5m high sandstone block retaining wall at the south-western end of the north-western boundary.

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 Very Low, which would be considered acceptable in accordance with the criteria given in the reference provided in Section 4.1. 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 7.5 x 10^{-7}. Therefore, this risk is considered acceptable in relation to the criteria given in the reference provided in Section 4.1. As part of the proposed development a new engineered seawall will be constructed, which will improve the above risk level. Recommendations for the new wall are provided in Section 5.2.

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 has been 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 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.

Our assessment was carried out for the existing site and the proposed development shown on the referenced architectural drawings, which does not require excavation, other than possible minor trimming/filling to achieve final levels. 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 and existing and proposed development can achieve the 'Acceptable Risk Management' criteria.

5 COMMENTS AND RECOMMENDATIONS

5.1 Excavation and Hydrogeology

While some minor excavations may be required, the depth of excavation is anticipated to be less than 0.3m. Excavation to these depths will encounter sandy fill and natural sands and may be completed using conventional earthworks equipment (eg. hydraulic excavators). The excavated granular fill and natural soils must be disposed of appropriately.

Whilst groundwater was not encountered in the boreholes (which was to about RL0.5m and RL0.6m), long term groundwater monitoring was not completed. Notwithstanding this, we anticipate that groundwater levels below the site will be tidal and will be effected by sea levels in Middle Harbour. 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 reference should be made to the Horton report. Horton's report should be reviewed in relation to water levels and scour. Horton indicates that the 100 year Average Recurrence Interval (ARI) for present day water levels as at RL1.5m while in 2082 it is expected to be RL1.88m. Wave action will increase water levels further. In front of the wall, scour of the sands is anticipated to occur to a level of RL-0.8m, which is considered to be a reasonable design level.

Considering only minimal temporary excavation is proposed, i.e. less than 0.3m, we do not anticipate groundwater will be encountered during construction.

The existing and surrounding buildings and structures are expected 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 structure.

The magnitude of transmitted vibrations that may impact nearby structures will depend on the type and size of plant/equipment used, how it is used, set-backs from structures, experience of operators, etc. Vibration monitoring should be undertaken at the commencement of demolition and during initial tracking of plant/equipment over the soils, 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 to 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 (VEDG) 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 building and structures to the north-west (17 Monash Crescent) while consideration could be given to completing a dilapidation survey on the building and structures to the south-east (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 New Sea Wall Design Concept and Retaining Wall Design

We understand that the preferred sea wall design, as shown on the architectural drawings, comprises a CFA grout injected contiguous pile wall installed below existing beach levels with a core filled block wall constructed on top of this contiguous pile wall. This new wall will be constructed in front of the existing sea wall with the gap between the two then backfilled.

Scour of sand from in front of and beside the wall poses the greatest risk to the satisfactory performance of this wall. In this regard it must be recognised that a contiguous pile wall has gaps between the piles that typically range up to about 50mm. This means that where saturated sands are present behind the wall they will flow through the wall and be lost from behind it. This will result in the settlement of the ground surface and the formation of sinkholes. In addition, as the wall is only proposed to be constructed across the front of the property and returns only a short distance down the north-western and south-eastern sides of the property, where scour extends beyond the ends of the walls there will be no protection and the soils will be readily eroded. Consequently, rather than a contiguous pile wall we recommend that a CFA grout injected secant pile wall be adopted. In addition, this wall must not only be formed along the south-western side of the property but must extend for a sufficient distance along both the north-western and south-eastern sides of the property such that they extend beyond the scour zone. This is likely to require the removal and reconstruction of at least part of the existing walls. Care must be taken that removal of these walls does not result in the undermining of adjoining structures or neighbouring properties. Once the extent of the scour zone is known and the extent of the walls can be determined further advice should be sought from this office.

For the design of the proposed seawall, we recommend that the following earth pressures be adopted:

- A triangular earth pressure distribution and a lateral earth pressure coefficient (k) of 0.35 should be adopted for cantilevered retaining walls. This assumes a horizontal ground surface at the back of the wall.
- A unit weight of $20kN/m^3$ should be adopted for all retained soils.
- The wall should be designed to resist full hydrostatic pressures.
- All applicable surcharge loads such as building loads, stockpiles, traffic loadings, wave loads, etc. should be added to the above earth pressures.

4.4 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.

The design of new footings will depend on the width of the footings, footing embedment depth, relative density of the materials on which they are founded, tolerable settlements, etc. As a preliminary guide, footings with a width and embedment depth of at least 0.5m that are founded in natural sands of very loose relative density may be designed for an allowable bearing pressure (ABP) of 50kPa. The base of the footings should be compacted using a vibratory plate compactor. Maximum total settlements for a strip footing (at the centre) for a 15m length are anticipated to be about 20mm. Differential settlements are anticipated to be roughly half the total settlements. Following the completion of the preliminary footing design this office should be contacted for further advice on the footing dimensions required to carry the design loads and the anticipated settlements.

Prior to pouring concrete all footings should be inspected and tested by the geotechnical engineer to confirm that the design ABP's have been achieved. Testing is anticipated to comprise the completion of a number of DCP tests.

Where the existing garage footings are reused, an assessment of their capacity is required to confirm that they have sufficient capacity to carry the design loads. Where this is the case, we should be contacted for further advice. However, we recommend if the above is proposed, that in the early stages of demolition a number of test pits be excavated at critical locations so that the existing footing details may be inspected by a geotechnical engineer and an assessment of their capacity and any additional settlements they may undergo may be made.

4.5 Site Preparation and Earthworks

The proposed storeroom finished floor level is RL2.70m. Some minor filling, about 0.1m to 0.2m high may be required to achieve final levels. Predicted sea-level changes over the design life of the garage as discussed above and in the Horton report should be considered in the final FFLs adopted.

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

- All grass, topsoil, and any other root affected soils should be stripped from the site.
- Following site stripping the exposed subgrade should be proof rolled with at least six (6) passes of a five (5) 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 causing excessive heaving.

- Where unstable spots are identified they should be excavated down to a sound base and replaced with engineered fill. Where unstable areas are detected then further advice should be obtained from the geotechnical engineers.

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 or ripped sandstone, 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%, while ripped sandstone should be compacted between 98% and 102% of Standard Maximum Dry Density (SMDD) within +/-2% of Standard Optimum Moisture Content (SOMC).

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 $500m^2$ or three tests per visit, whichever requires the most tests. We recommend that at least Level 2 control of fill compaction, as defined in AS3798-2007 (or latest standard at the time of testing), be adhered to on this site. We can complete the abovementioned testing and supervision if required.

4.6 Slabs on Grade

Prior to the placement of slabs on grade or pavements we recommend that the recommendations provided above in Section 4.5 Slabs on Grade and Earthworks are followed.

For the design of slabs on grade that will be trafficked, a modulus of subgrade reaction of 50kPa/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 RTA QA specification 3051 (1994) unbound base material (or equivalent good quality and durable fine crushed rock) which is compacted to at least 100% SMDD. The subbase will provide a more stable working platform, will provide more uniform slab support and will reduce 'pumping' of 'fines' at joints.

4.7 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 non-aggressive to buried steel structures in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009.

4.8 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:

- Dilapidation survey of 17 and 13 Monash Crescent if considered necessary.
- Vibration monitoring during demolition and tracking of machinery.
- Geotechnical review of seawall design, and in particular with reference to the estimated scour levels and extent.
- Review of preliminary footing design.
- Test pit excavations where existing footings will be reused.
- Proof roll of subgrade.
- Inspection of all footings prior to pouring 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 construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs 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 will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 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) should be expected. We strongly recommend that this issue 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**
	Failure of Seawall	Failure of Retaining Wall
Assessed Likelihood	Possible	Unlikely
Assessed Consequence	Insignificant	Insignificant
Risk	Very Low	Very Low
Comments	-	-

***Property Value Assumed to be $11.5 million (Ref: www.onthehouse.com.au, 7 March 2022)**

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A** Failure of Seawall	**B** Failure of Retaining Wall
Assessed Likelihood	Possible	Unlikely
Indicative Annual Probability	10^{-3}	10^{-4}
Duration of Use of area Affected (Temporal Probability)	Above, 15 minutes/day (0.0104) Below, 1hr/day x 3 months per year (sitting/laying) + 1 minute/day (walking) (0.0111)	Above, 5 minutes/day (0.00347) Below, 10 seconds/day (walking) (1.15×10^{-4})
Probability of not Evacuating Area Affected	Above, 0.8 Below, 0.5	Above, 0.8 Below, 0.5
Spatial Probability	Above and Below, 4m length fails, 4m/18m (0.2)	Above and Below, 3m length fails, 3m/3m (1.0)
Vulnerability to Life if Failure Occurs Whilst Person Present	Above, 0.01 (Ride Down) Below, 0.5 (Crushed)	Above, 0.01 (Ride Down) Below, 0.5 (Crushed)
Risk for Person most at Risk	Above, 2.08×10^{-8} Below, 6.94×10^{-7}	Above, 2.78×10^{-9} Below, 2.88×10^{-9}
(Total Risk)	7.5×10^{-7}	

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CERTIFICATE OF ANALYSIS 227009

Client Details

Client	JK Geotechnics
Attention	Kartik Singh
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**32694YJ, Clontarf**
Number of Samples	3 Soil
Date samples received	26/09/2019
Date completed instructions received	26/09/2019

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	03/10/2019
Date of Issue	02/10/2019

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Results Approved By
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Authorised By

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Misc Inorg - Soil				
Our Reference		227009-1	227009-2	227009-3
Your Reference	UNITS	BH1	BH2	BH4
Depth		0.4-0.6	0.4-0.5	0.35-0.45
Date Sampled		25/09/2019	25/09/2019	25/09/2019
Type of sample		Soil	Soil	Soil
Date prepared	-	30/09/2019	30/09/2019	30/09/2019
Date analysed	-	30/09/2019	30/09/2019	30/09/2019
pH 1:5 soil:water	pH Units	7.2	7.6	6.9
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	<10
Resistivity in soil*	ohm m	160	350	270

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. 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	227009-2
Date prepared	-			30/09/2019	1	30/09/2019	30/09/2019		30/09/2019	30/09/2019
Date analysed	-			30/09/2019	1	30/09/2019	30/09/2019		30/09/2019	30/09/2019
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	7.2	7.2	0	103	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	96	92
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	101	106
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	160	180	12	[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.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

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.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-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.

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BOREHOLE LOG

Borehole No.
1
1/1

Client: JOHN KOLENDA
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 15 MONASH CRESCENT, CLONTARF, NSW

Job No.: 32694YJ **Method:** HAND AUGER **R.L. Surface:** ≈ 2.5m
Date: 24/9/2019 **Datum:** AHD
Plant Type: **Logged/Checked by:** K.K.S./J.M.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION AND 1HR AFTER		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, gray, trace of roots and root fibres.	M			GRASS COVER
					SM	Silty SAND: fine to medium grained, brown, trace of root fibres.	M	L		APPEARS POORLY COMPACTED
						as above, but without root fibres.		L-MD		AEOLIAN
			1			Silty SAND: fine to medium grained, yellow brown.		L		
			2			END OF BOREHOLE AT 2.0m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
2
1/1

Client: JOHN KOLENDA
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 15 MONASH CRESCENT, CLONTARF, NSW

Job No.: 32694YJ **Method:** HAND AUGER **R.L. Surface:** ≈ 2.5m
Date: 24/9/2019 **Datum:** AHD
Plant Type: **Logged/Checked by:** K.K.S./J.M.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		REFER TO DCP TEST RESULTS	0			FILL: Silty clayey sand, fine to medium grained, dark brown, trace of roots and root fibres, glass fragments and fine to medium grained sub angular sandstone gravel.	M			GARDEN BED
					SM	Silty SAND: fine to medium grained, brown, trace of root fibres.	M	L-MD		APPEARS POORLY COMPACTED
								MD		AEOLIAN
			1			Silty SAND: fine to medium grained, yellow brown.		L		
			2			END OF BOREHOLE AT 2.0m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. 3

1/1

Client: JOHN KOLENDA

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 15 MONASH CRESCENT, CLONTARF, NSW

Job No.: 32694YJ **Method:** HAND AUGER **R.L. Surface:** ≈ 2.6m

Date: 24/9/2019 **Datum:** AHD

Plant Type: **Logged/Checked by:** K.K.S./J.M.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPL -ETION AND 30MINS AFTER		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown and grey, trace of fine to medium grained sub angular igneous gravel, roots and root fibres.	M			GARDEN BED APPEARS POORLY COMPACTED
					SM	Silty SAND: fine to medium grained, grey, trace of root fibres.	M	VL		AEOLIAN
			1			as above, but without root fibres.		L VL-L		
						Silty SAND: fine to medium grained, yellow brown.				
			2			END OF BOREHOLE AT 2.0m				
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
4
1/1

Client: JOHN KOLENDA
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 15 MONASH CRESCENT, CLONTARF, NSW

Job No.: 32694YJ **Method:** HAND AUGER **R.L. Surface:** ≈ 2.4m
Date: 24/9/2019 **Datum:** AHD
Plant Type: **Logged/Checked by:** K.K.S./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, trace of roots and root fibres, and fine to medium grained sub angular sandstone gravel.	M			GRASS COVER APPEARS POORLY COMPACTED
									FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained sub angular siltstone gravel.				HAND AUGER REFUSAL INFILL, POSSIBLY ON A SANDSTONE BOULDER
						1			END OF BOREHOLE AT 0.55m				
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
5
1/1

Client: JOHN KOLENDA
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 15 MONASH CRESCENT, CLONTARF, NSW

Job No.: 32694YJ **Method:** HAND AUGER **R.L. Surface:** ≈ 2.4m
Date: 24/9/2019 **Datum:** AHD
Plant Type: **Logged/Checked by:** K.K.S./J.M.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, grey, trace of roots and root fibres.	M			GRASS COVER
									FILL: Silty sand, fine to medium grained, brown, trace of fine to coarse grained sub angular sandstone gravel.				APPEARS POORLY COMPACTED
						1			END OF BOREHOLE AT 0.82m				HAND AUGER REFUSAL IN FILL
						2							
						3							
						4							
						5							
						6							
						7							

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	JOHN KOLENDA
Project:	PROPOSED ALTERATIONS AND ADDITIONS
Location:	15 MONASH CRESCENT, CLONTARF, NSW

Job No.	32694YJ	Hammer Weight & Drop: 9kg/510mm
Date:	24-9-19	Rod Diameter: 16mm
Tested By:	K.K.S.	Point Diameter: 20mm

Test Location	1	2	3	Test Location	1	2	3
Surface RL	~2.5m	~2.5m	~2.6m	Surface RL	~2.5m	~2.5m	~2.6m
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	1	3000-3100	5	7	4
100 - 200	1		1	3100-3200	6	7	6
200 - 300	1			3200-3300	7	8	9
300 - 400	1	2		3300-3400	6	8	8
400 - 500	2	3		3400-3500	7	10	9
500 - 600	1	4	1	3500-3600	8	9	9
600 - 700	4	4	1	3600-3700	11	9	10
700 - 800	2	5	1	3700-3800	10	10	9
800 - 900	4	4	2	3800-3900	10	11	10
900 - 1000	3	3	2	3900-4000	11	10	10
1000 - 1100	3	2	1	4000-4100	13	END	END
1100 - 1200	4	2	1	4100-4200	14		
1200 - 1300	3	3	1	4200-4300	14		
1300 - 1400	2	2	2	4300-4400	15		
1400 - 1500	2	1	2	4400-4500	14		
1500 - 1600	2	2	1	4500-4600	12		
1600 - 1700	2	2	1	4600-4700	12		
1700 - 1800	2	3	1	4700-4800	11		
1800 - 1900	3	2	1	4800-4900	14		
1900 - 2000	2	3	2	4900-5000	13		
2000 - 2100	3	4	2	5000-5100	END		
2100 - 2200	5	3	3	5100-5200			
2200 - 2300	6	3	4	5200-5300			
2300 - 2400	5	2	4	5300-5400			
2400 - 2500	5	2	4	5400-5500			
2500 - 2600	4	6	3	5500-5600			
2600 - 2700	3	8	4	5600-5700			
2700 - 2800	5	7	3	5700-5800			
2800 - 2900	5	8	3	5800-5900			
2900 - 3000	6	7	4	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

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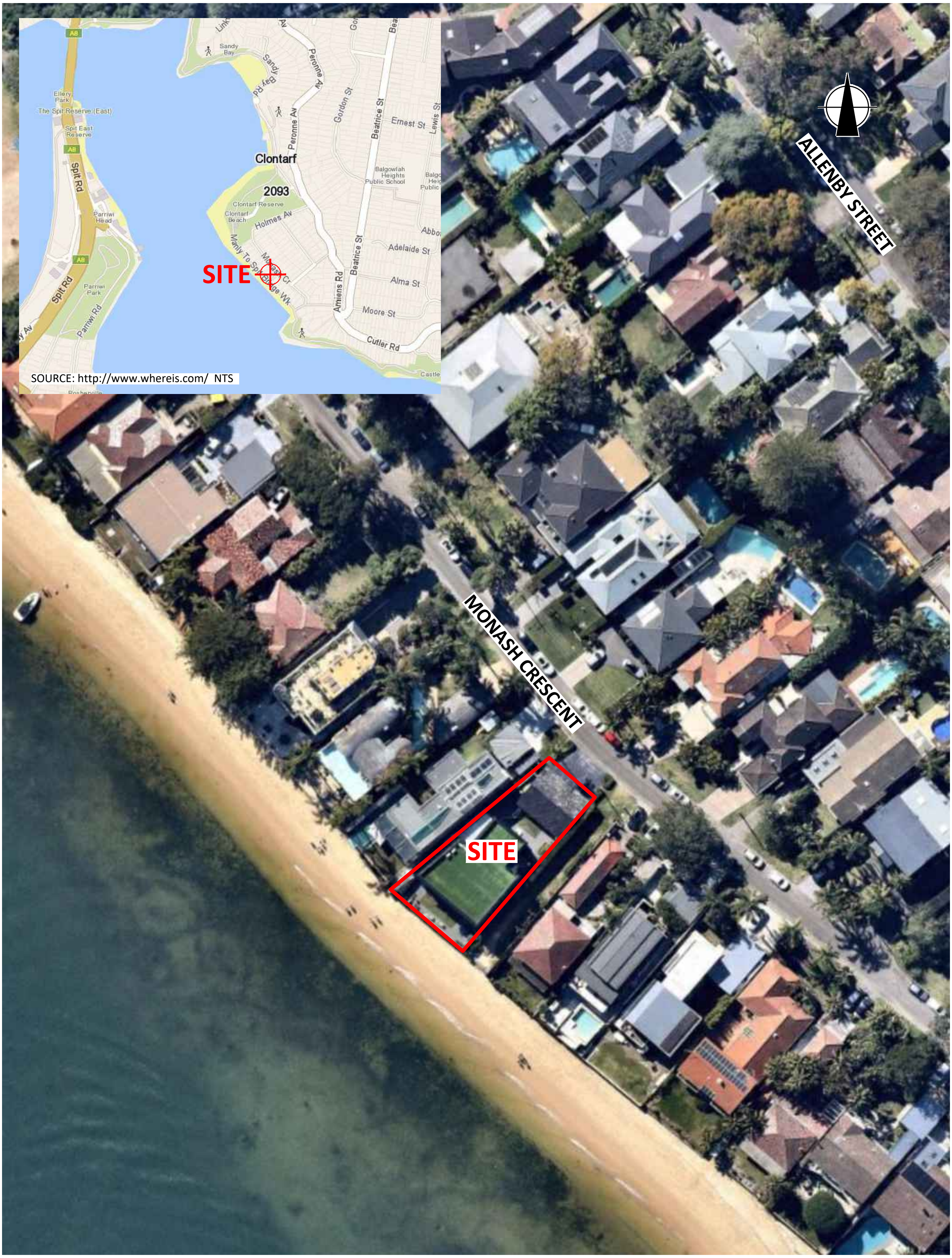
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	JOHN KOLENDA		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	15 MONASH CRESCENT, CLONTARF, NSW		
Job No.	32694YJ	Hammer Weight & Drop: 9kg/510mm	
Date:	24-9-19	Rod Diameter: 16mm	
Tested By:	K.K.S.	Point Diameter: 20mm	

Test Location	4	5	6	Test Location	4	5	6
Surface RL	~2.4m	~2.4m	~2.4m	Surface RL	~2.4m	~2.4m	~2.4m
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	1	3000-3100			7
100 - 200	1	2	1	3100-3200			8
200 - 300	1	1	2	3200-3300			8
300 - 400	1	1	2	3300-3400			6
400 - 500	11	8	3	3400-3500			7
500 - 600	8/10mm	9	3	3500-3600			7
600 - 700	REFUSAL	3	4	3600-3700			7
700 - 800		20/20mm	4	3700-3800			8
800 - 900		REFUSAL	4	3800-3900			9
900 - 1000			4	3900-4000			8
1000 - 1100			5	4000-4100			END
1100 - 1200			3	4100-4200			
1200 - 1300			3	4200-4300			
1300 - 1400			2	4300-4400			
1400 - 1500			2	4400-4500			
1500 - 1600			1	4500-4600			
1600 - 1700			1	4600-4700			
1700 - 1800			1	4700-4800			
1800 - 1900			1	4800-4900			
1900 - 2000			1	4900-5000			
2000 - 2100			1	5000-5100			
2100 - 2200			2	5100-5200			
2200 - 2300			2	5200-5300			
2300 - 2400			1	5300-5400			
2400 - 2500			3	5400-5500			
2500 - 2600			3	5500-5600			
2600 - 2700			3	5600-5700			
2700 - 2800			3	5700-5800			
2800 - 2900			5	5800-5900			
2900 - 3000			8	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



0 10 20 30 40 50

SCALE 1:1000 @A4 m

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

Title:	**SITE LOCATION PLAN**
Location:	15 MONASH CRESCENT CLONTARF, NSW
Report No:	32694YJrpt2
Figure:	1

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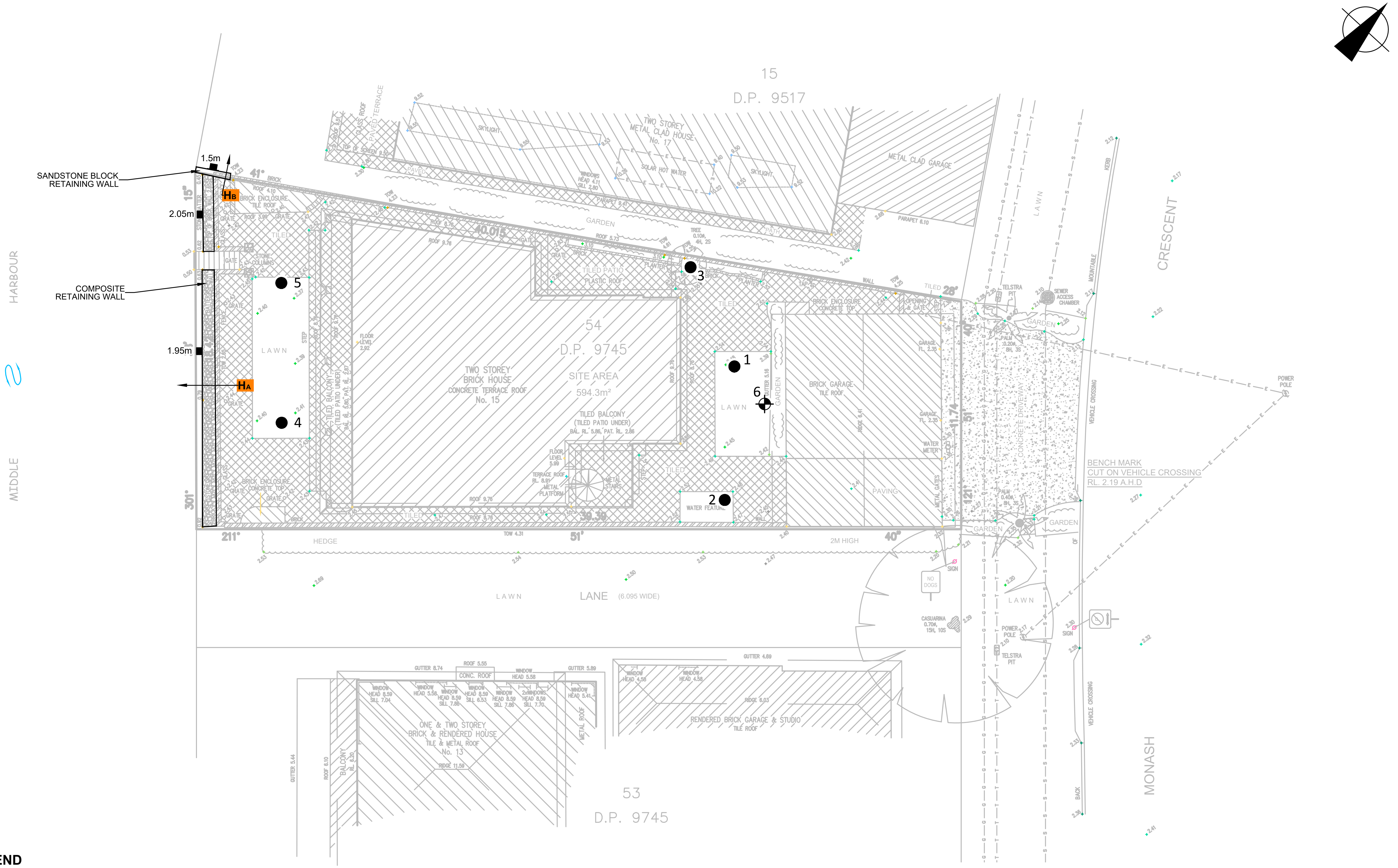
PLOT DATE: 3/03/2022 3:54:54 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32694Y) CLONTARF\CAD\32694YJRPT2.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST
- HA2 GEOTECHNICAL HAZARD
- HEIGHT OF WALL

0 2 4 6 8 10
SCALE 1:200 @A3 m

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

Title:	INVESTIGATION LOCATION PLAN INCLUDING GEOTECHNICAL HAZARDS
Location:	15 MONASH CRESCENT CLONTARF, NSW
Report No:	32694YJrpt2
Figure:	1

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PLOT DATE: 3/03/2022 3:55:06 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32694YJ CLONTARF\CAD\32694YJRPT2.DWG

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_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

Where major civil or structural developments are proposed 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	

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

Laboratory Classification Criteria

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

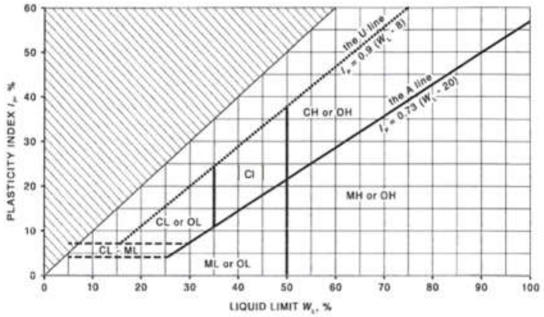
$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\, D_{60}}$$

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

NOTES:

1. For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
2. Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.
3. Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.
4. The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**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
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

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.

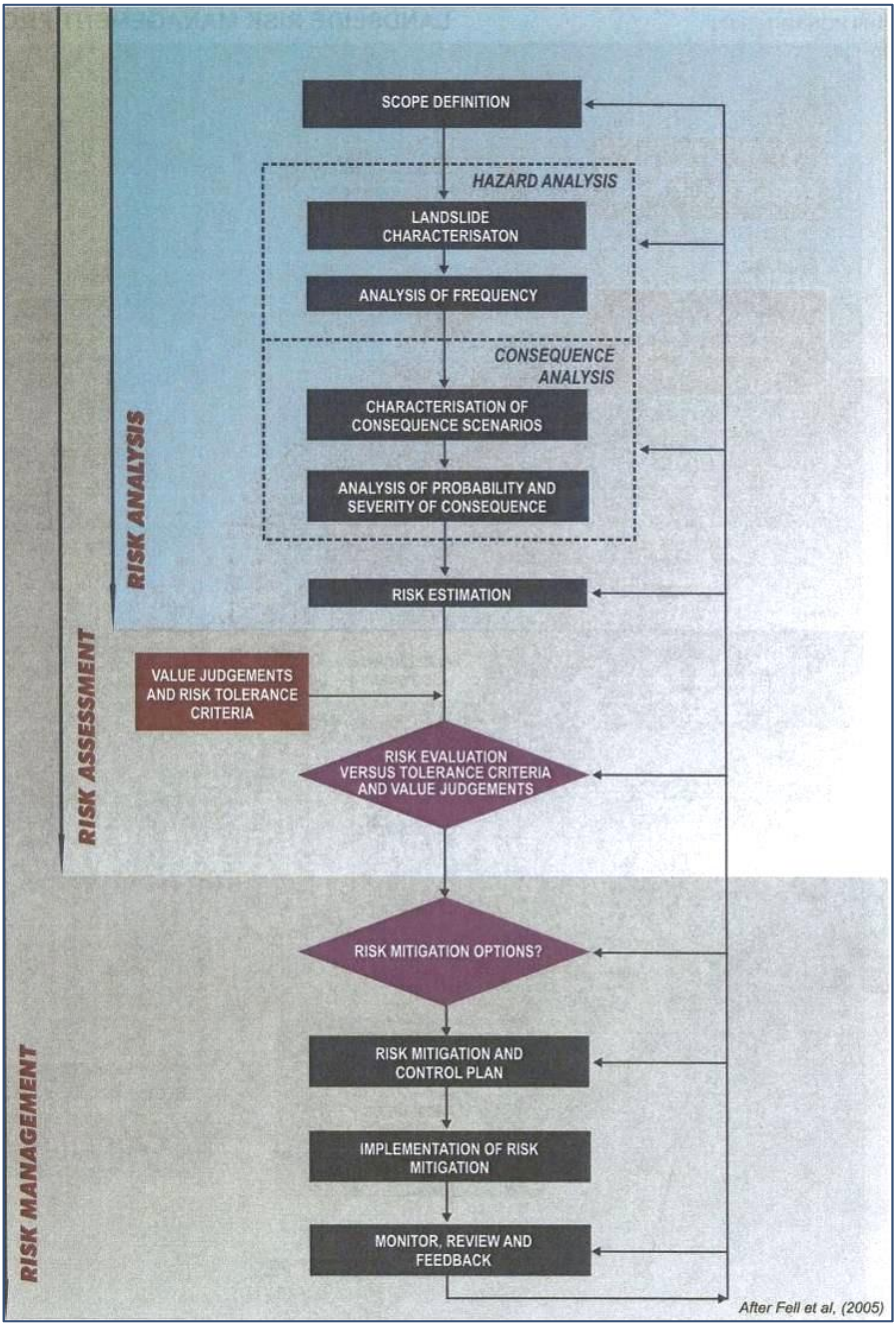


FIGURE A1: Flowchart for Landslide Risk Management.

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}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-2}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 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%	40%	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%	10%	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%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		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. The 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 series 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.

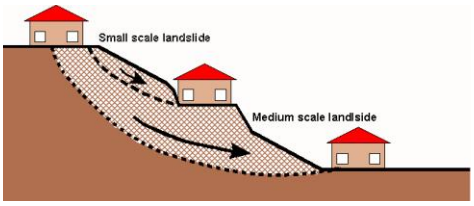


Figure 1

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.

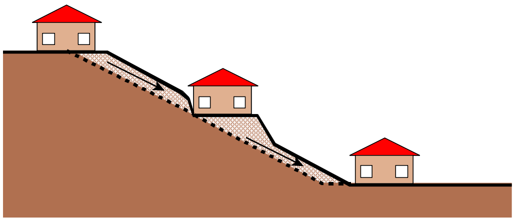


Figure 2

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.

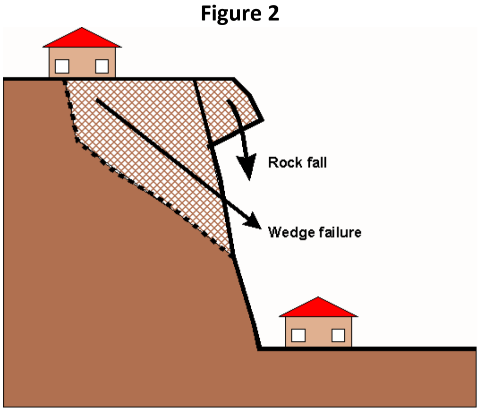


Figure 3

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.

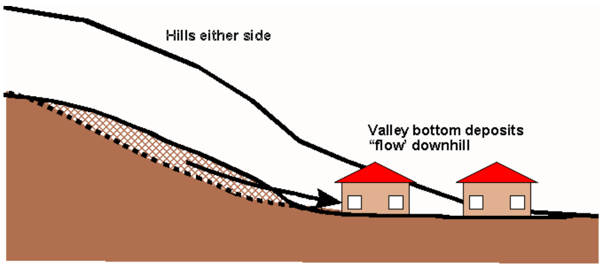


Figure 4

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:

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