REPORT TO

SANJEEV LOURA

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Date: 17 November 2021

Ref: 33510PDrpt Rev1

JKGeotechnics

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33510PDrpt	Final Report	25 September 2020
33510PDrpt Rev1	Final Report – Updated Architectural Drawings	17 November 2021

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<u>ATTACHMENTS</u>

Cone Penetrometer Test Results 1 to 3

Figure 1: Site Location Plan

Figure 2: Investigation Location Plan

Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed residential development at 45 to 49 Warriewood Road, Warriewood, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Sanjeev Loura by email dated 8 September 2020. The commission was on the basis of our proposal (Ref: P52596HD) dated 4 September 2020.

We have been provided with the following information:

- Relevant architectural drawings (Drawing Nos. A04^{06}, A05^{06}, A12a^{05}, A12a-a^{01}, A12a-b^{01}, A12b^{05}, A12b-a^{01}, A12b-b^{01} dated 21 May 2021) Prepared by Archidrome.
- Site survey plan (Drawing Name: 15843detail, Sheet 1, dated 7 December 2016) prepared by C.M.S. Surveyors Pty Ltd.
- Addendum geotechnical advice report (Ref: CA/20/126-2707, dated 23 April 2020) prepared by NG Child & Associates.

Based on the provided information we understand the proposed development will include demolition of the existing structure on site and construction of two three storey buildings over a single level basement car park. Excavation to a maximum depth of about 4.2m will be required to achieve the basement bulk excavation level.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions and use it as a basis for comments and recommendations on excavation conditions, drainage, excavation support, retention design parameters, footing design, subgrade preparation, engineered fill, floor slabs and external pavements, and landslide risk management.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 9 September 2020 and comprised three Cone Penetration Tests (CPT1 to CPT3) completed to refusal depths of 9.57m, 12.01m and 19.22m below the existing surface levels, using our 24 tonne truck mounted CPT rig.

The test locations are show on the attached investigation location plan (Figure 2). The surface RLs at the test locations were estimated by interpolation between spot levels indicated on the survey plan. The survey plan has been over laid on a Nearmap aerial photo which forms the basis of Figure 2. The datum is the Australian Height Datum (AHD).

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

The CPT involved continuously pushing a testing probe with a 35mm diameter conical tip into the subsoil profile using the hydraulic rams of a ballasted truck mounted rig. Measurements were made during testing of the end resistance of the cone tip and the frictional resistance of a separate 134mm long sleeve located directly behind the cone. The testing was carried out using a piezocone, which measures pore water pressures within the soils and provides an indication of the depth of the groundwater.

We note that the CPT does not recover soil samples and the strength/relative density and composition of the subsurface materials were assessed by interpretation of the CPT results using published and in-house correlations.

Groundwater observations were also made following extraction of the CPT rods. Groundwater levels were measured in the existing monitoring wells (MW1 to MW3, shown on the attached Figure 2) installed by NG Child and Associates.

Our geotechnical engineer (Ms. Joanne Lagan) was present on a full-time basis during the fieldwork and set out the test locations, directed the buried services scan, and operated the computerised data collection system during testing. The CPT results (including an interpreted subsoil profile and pore water pressure profile) are attached, together with a glossary of logging terms and symbols used.

Contamination screening of the site soils and groundwater were outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on the north-eastern bank of a gully feature extended north-west through the local area. The site is roughly rectangular in plan shape being approximately 300m long (south-west to north-east) by 75m (south-east to north-west). The site generally slopes down to the south-west at between about 2° and 5° steepest at the north-eastern end of the site and flattening out towards Narrabeen Creek which forms the south-western site boundary. The site has a north-eastern frontage onto Warriewood Road.

At the time of fieldwork, the north-eastern half of the site was densely vegetated with several single level fibro clad buildings and sheds scattered throughout. Multiple rows of dilapidated glass houses extended down slope and were separated by a gravel access road, which extended from Warriewood Road down through the centre of the site. The remaining south-western portion of the site was densely vegetated.

The neighbouring properties to the north-west comprised one and three storey rendered and brick residential buildings, set back at least 1m from the common boundary. The ground surface level across the common boundary was similar, with the exception of the south-western end where a masonry block retaining wall was supporting the neighbouring property up to approximately 1.5m height. The masonry block retaining wall generally appeared to be in good condition.

The neighbouring properties to the south-east contained two storey residential buildings currently under construction. Observations were limited due to site access, but where visible the neighbouring buildings appeared to be set back at least 1m from the common boundary. A masonry block retaining wall located along the north-east portion of the common boundary was supporting the neighbouring site up to about 1.8m height and generally appeared to be in good condition.

3.2 Subsurface Conditions

Reference to the 1:100,000 Geological Series Sheet for Sydney indicates the site is underlain by deep alluvial deposits comprising silty clay and silty sand, although the site is located close to the geological boundary with Hawkesbury Sandstone. The subsurface conditions encountered at the test locations comprised a limited thickness of surficial fill overlying alluvial sands and clays. For detailed subsurface conditions at each test location, reference should be made to the attached CPT results sheets. A summary of some of the more pertinent subsurface information is outlined below.

Fill

A limited thickness of surficial fill comprising fine to coarse grained gravel was encountered from surface of each CPT test.

Alluvial Soils

Alluvial soils comprising predominantly silty clay, but with layers of silty sand of variable thicknesses at varying depths, were indicated below the fill at each investigation location down to the CPT refusal depths. At the CPT locations, the silty sand was assessed to range from very loose to medium dense, whilst the silty clays were assessed to range from firm to hard strength.

Inferred Weathered Bedrock

Weathered bedrock was inferred at the CPT refusal depths of 9.57m (CPT1), 12.01m (CPT2) and 19.22m (CPT3) however as the CPT cannot penetrate rock, the presence of rock cannot be confirmed. On this basis the bedrock has been assessed to step down to the south-west.

Groundwater

Groundwater was indicated at about 0.6m depth in CPT1 and CPT 2 and at about 0.5m depth in CPT3. A summary of the groundwater levels measured during the fieldwork are presented below.

Location	Approximate Surface RL (mAHD)	Approximate Depth (m)	Approximate Groundwater RL (mAHD)
CPT1	8.3	0.6	7.7
CPT2	4.8	0.6	4.2
CPT3	3.3	0.5	2.8
MW1	9.5	2.6	6.9
MW2	4.8	0.8	4.0
MW3	3.3	0.5	2.8

4 Stability Assessment

The site is located within the former Pittwater Local Government Area (LGA) and excavation for the proposed development will extend to depths greater than 1.5m, therefore, we understand that the site is subject to the Pittwater Council 'Geotechnical Risk Management Policy'. The proposed development site is located outside all Hazard Zones as identified on the 'Pittwater Geotechnical Hazard Map'.

The site is located on a south-westerly facing hillside that slopes between about 2° and 5° where general hillside instability is inconceivable. We therefore consider the only remaining hazards to be the existing retaining walls and proposed basement retaining walls/shoring system.

We have carried out a qualitative assessment of the risk to property associated with potential collapse of retaining walls, in accordance with AGS 2007c. We have adopted a likelihood of 'Rare'. We further consider that the consequences to property and noting the nature of the proposed development, to be 'Medium' should any instability occur. Based on the above, the risk to property is Low which would be considered 'Acceptable', in accordance with the recommendations provided in AGS 2007c.

Given the very low likelihood, and the expected very low temporal probability (the probability of a person being very close to the retaining wall when it failed), we expect the risk to life will be less than 10^{-6}, which would also be considered 'Acceptable' in accordance with the recommendations provided in AGS 2007c.

In preparing our recommendations, we have also assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all buried services within and surrounding the site are, and will be, regularly maintained to remain in good condition.

The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A.

We consider that our risk analysis has shown that the proposed development can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy provided that the recommendations given in Sections 5 and 6 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

5 COMMENTS AND RECOMMENDATIONS

5.1 Geotechnical Issues

Based on the results of our investigations, we consider that the following principal geotechnical issues will need to be carefully considered in the design and construction of the proposed development:

- The groundwater level is relatively shallow and seepage is expected to occur.
- Bulk excavation will extend below the groundwater level and tanking of the basement will be required if permission cannot be obtained to drain the basement.
- The alluvial soils are of variable relative density and strength, and bedrock was inferred at depths up to about 19.2m below the existing surface levels.

The above issues are discussed in more detail in the following sections of the report.

5.2 Excavation

The following recommendations should be read in conjunction with the NSW Government 'Code of Practice Excavation Work' dated January 2020.

5.2.1 Excavation Methods

Construction of the proposed residential development will require demolition and of the existing buildings, structures and driveways, and removal of vegetation. Following this, all top soil and root affected soil, and any deleterious or contaminated existing fill should be stripped. Stripped topsoil and/or root effected soils should be stockpiled separately as it is not considered suitable for reuse as engineered fill but may be reused for landscaping purposes. Guidelines on offsite disposal of soils are provided in Section 5 below.

Excavation for the proposed development will extend to a maximum depth of about 4.2m below the existing surface levels with additional localised excavations to about 1.5m for the proposed lift pits. The excavations will extend through the alluvial soil profile which can be readily excavated using the bucket attachment on a large hydraulic excavator.

5.2.2 Seepage

Groundwater seepage was indicated in the CPT test and measured in the existing NG monitoring wells at relatively shallow depths.

In order to facilitate excavation of the basement level in the 'dry', we recommend that basement shoring be formed as perimeter cut-off walls founded in the underlying low permeable silty clay or weathered bedrock below bulk excavation level (BEL).

We recommend that perimeter secant pile walls be constructed for the proposed basement excavation. as a result, we expect that groundwater infiltration into the excavation would be of a limited volume and be satisfactorily controlled by conventional sump and pump drainage techniques.

We recommend seepage analysis be undertaken for the proposed basement. If the groundwater inflows are relatively low, it may be possible to drain the basement using the collected water for garden reuse, and watering/irrigation of the low end of the site.

Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed can be timeously addressed.

5.3 Excavation Support and Retention Design

5.3.1 Excavation Support

Temporary batter slopes through the soil profile may not be feasible due to the expected shallow groundwater levels and the proposed excavation depth. Temporary batters no steeper than 1 Vertical (V) in 1.5 Horizontal (H) in the alluvial soil above any groundwater levels are considered appropriate.

Due to the requirement to form a cut-off wall to facilitate excavation, a full depth engineered retention system installed prior to commencement of excavation will be required. The majority of the piled wall retention system could likely be cantilevered from a toe socket, though where the depth of excavation exceeds about 2m, it will likely require internal propping or anchoring in order to reduce deflections. This will need to be assessed by the structural engineer, in particular with regard to the presence of movement sensitive services within the existing easement. Internal propping may restrict construction activities and would need to be carefully sequenced. Alternatively, the shoring system could be designed with an internal stabilising berm, which remains in place until the ground floor slab has been cast.

The piled wall retention system will need to comprise grout injected (CFA) secant piled walls. The piles would need to extend to sufficient depth below BEL to satisfy stability considerations and provide an effective cut-off for the groundwater. In this regard the secant piles will need to be socketed into the underlying bedrock or embedded within the low permeability silty clays.

Further advice from the piling contractor should be sought in regard to the most suitable piling rig to form the rock sockets. However, we have no information on bedrock strength, and so following demolition, when access for a drill rig is feasible, additional cored boreholes should be drilled to provide this additional information.

The proposed basement may need to be tanked if permission cannot be gained for a drained basement and further advice is provided in Sections 5.3.2 and 5.3.3 below.

5.3.2 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of the piled wall shoring system and any landscape walls:

- Cantilever shoring walls should be designed for a triangular earth pressure distribution and an active lateral earth pressure coefficient (Ka) of 0.35.
- Where the shoring wall will be propped or anchored, and in the final case for the basement walls which will be braced by the building, the design should be checked using a uniform/rectangular earth pressure distribution of 6H kPa, where H is the depth of excavation in metres.
- The above recommendations are for near level backfill surfaces behind shoring.
- Where some minor movements of retaining walls may be tolerated (e.g. landscape walls), they may be designed using a triangular lateral earth pressure distribution and a coefficient of 'active' earth pressure, (k_a), of 0.3.5 for the soil, assuming a horizontal backfill surface.
- An effective bulk unit weight of 20kN/m^3 and 10kN/m^3 should be adopted for the retained profile above and below groundwater level, respectively.
- Any surcharge affecting the walls (e.g. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.
- Due to the elevated groundwater levels encountered in the investigation, the proposed basement will need to be permanently tanked if permission for a drained basement cannot be obtained. The tanked retention system must be designed to withstand full hydrostatic pressures. A groundwater level equivalent to the surrounding ground surface levels or 100yr flood level if it is higher than the surface levels should be adopted for design. Reference should be made to Section 5.3.3 below for further comments in this regard.
- Toe restraint for landscape walls and for retention piles founded in the soil profile may be provided by the passive pressure of the soil below adjacent surface levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections associated with achieving a full passive case. Localised excavations in front of the walls e.g. for buried services etc must be taken into account in the design, along with an allowance for over excavation of 10% of the shoring height.
- For retention piles socketed into the sandstone bedrock below bulk excavation level, an allowable lateral stress of 200kPa may be adopted for sandstone of at least low strength, with the upper 0.3m of socket ignored to allow for potential fracturing of the top of the socket.
- Rock anchors (if used) to resist hydrostatic uplift pressures should be bonded into sandstone bedrock of at least low strength and provisionally designed for an allowable bond strength of 150kPa. Anchors should be designed with a minimum bond length of 3m. these will need to be installed as permanent anchors, with all appropriate corrosion protect etc.

5.3.3 Tanking (If Required)

If the basement is to be tanked, the design should incorporate water pressures associated with the higher of the ground surface level, or the 100yr flood level, with a pressure relief system to protect the slab should even higher pressures occur.

The, uplift pressures acting on the basement slab would need to be resisted by ground anchors or tension piles designed in accordance with the advice provided in Section 5.4.2 above and Section 5.5 below, respectively, if the self-weight of the building does not provide sufficient resistance. In this regard, we recommend suspending the basement floor slab between piled footings. The piles would assist with resisting uplift pressures as well as supporting vertical structural loads.

In order to permanently tank or waterproof the basement if tanking id required, the structural design must take into account the lateral and uplift pressures associated with this system. Care is required with the tanking details particularly at internal footing locations and at the perimeter pile wall-floor slab connection. In this regard, we note that there are practical difficulties in providing a waterproof seal between floor slabs and the undulating face of secant pile walls. Provision would need to be made for sealing any gaps with a proprietary waterproof sealant.

The basement with secant pile walls has the potential to create a localised damming effect on groundwater flow which we have assessed to be generally down to the south-west. In order to satisfactorily manage this impact, we recommend the following:

- On the upstream (north-eastern) side of the proposed basement, provide a cut-off drain with an invert level set at 0.5m below the adjacent surface levels.
- The drainage pipe should be directed around the building and discharge to the stormwater system or as diffuse surface flow in the lower portion of the site.

5.4 Footing Design

Foundation piles and perimeter shoring piles founded at least 4 pile diameters below BEL into alluvial silty clay of hard strength and silty sand of at least medium density (identified in the CPT logs at depths between about 6.4m and 14m) or weathered bedrock may be designed for an allowable bearing pressure of 600kPa. Piles embedded in sands of medium dense relative density or hard strength clays may adopt an allowable shaft adhesion of 15kPa (in compression) and 5kPa (in tension). Piles socketed into weathered bedrock may adopt an allowable shaft adhesion of 60kPa (in compression) and 30kPa (in tension). Due to the significant variation in founding depths, it will be necessary to undertake significant additional investigation post demolition to confirm appropriate target depths for the piles in each area. This could be let as part of a design and construct piling package, with the contractor certifying the load capacity of the piles.

Higher bearing pressures for piles founded in the underlying bedrock may be feasible, however, we have no information on bedrock strength, and additional cored boreholes would need to be drilled to provide this additional information.

Based on the encountered high groundwater level and the collapsible nature of the sometimes sandy soils, conventional bored piles will not be suitable, though auger grout injected piles (CFA) or steel screw piles are considered to be suitable. A copy of the CPT test results should be provided to the piling contractor so the appropriate size piling rig is mobilised to site.

5.5 Basement Floor Slab

As noted in Section 5.3.3 above, we recommend that the basement floor slab be suspended between piles. The floor slab may need to be designed to resist uplift pressures if tanking is required, and ground anchors or tension piles may also be required if the self-weight of the building does not provide sufficient resistance.

For floor slabs suspended over soil subgrade areas, the subgrade preparation would comprise the removal of any topsoil and/or any soil containing organics, completion of the bulk excavation and the nominal tracking of 'formwork fill' to the required subgrade level.

5.6 External Paved Areas and Earthworks

The on-grade-construction of external pavements is considered to be feasible provide the sandy and clayey subgrade. We assume the pavement will be rigid/concrete design as it will not be feasible to compact roadbase materials over soils with such a high groundwater level.

Care must be taken not to disturb the subgrade in the proposed pavement area. The top soil and any deleterious or soft subgrade should be removed, and the surface drains installed on both sides of the subgrade. The excavation could then extend down to the design subgrade, and the surface inspected by a geotechnical engineer to confirm a suitable subgrade has been reached.

A thick lean mix concrete subbase should then be placed on the prepared subgrade, with a subsoil drain down each side extending at least 0.3m below the subgrade level.

5.7 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:

- Seepage analysis for basement to further assess whether a drained basement is feasible.
- Monitoring of groundwater seepage into the excavation.
- Geotechnical inspection during initial pile installation.
- Additional investigation to provide better target depths for the piled footings

6 RISK MANAGMENT

We consider that the proposed development may proceed provided the following specific design, construction and maintenance recommendations are adopted to maintain and reduce the present risk of instability of the site and to control future risks. These recommendations address geotechnical issues only and other conditions may be required to address other aspects.

6.1 Conditions Recommended to Establish the Design Parameters

6.1.1 Refer to Section 5 above.

6.2 Conditions Recommended to the Detailed Design to be Undertaken for the Construction Certificate

6.2.1 All structural design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.2 All hydraulic design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.3 All landscape design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.4 An excavation/retention methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation techniques, the proposed excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points.

6.2.5 The excavation/retention methodology must be reviewed and approved by the geotechnical engineer.

6.3 Conditions Recommended During the Construction Period

6.3.1 The approved excavation/retention methodology must be followed.

6.3.2 Proposed material to be used for backfilling behind retaining walls must be approved by the geotechnical engineer prior to placement.

6.3.3 Compaction density of the backfill material must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer. Backfill material should be compacted to a minimum density of 98% Standard Maximum Dry Density (SMDD) otherwise backfill with free draining durable single size granular material such as ‘Blue Metal’.

6.3.4 The geotechnical engineer must inspect all subsurface drains prior to backfilling.

6.3.5 An ‘as-built’ drawing of all buried services at the site must be prepared (including all pipe diameters, pipe depths, pipe types, inlet pits, inspection pits, etc).

6.3.6 The geotechnical engineer must confirm that the proposed development has been completed in accordance with the geotechnical reports.

We note that all above Conditions must be complied with. Where this has not been done, it may not be possible for Form 3, which is required for the Occupation Certificate to be signed.

7 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. 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 and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

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

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

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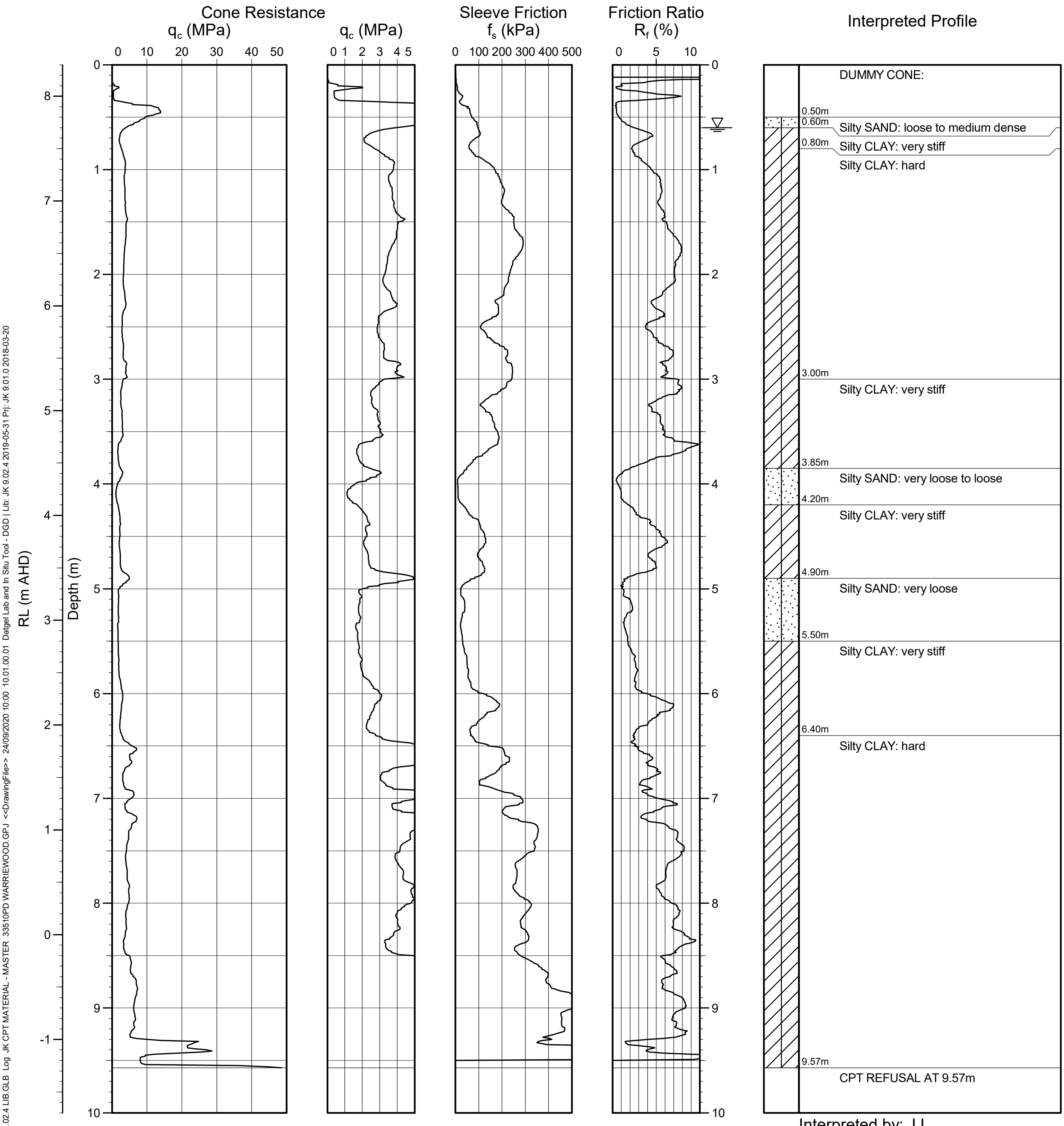
CONE PENETROMETER TEST RESULTS

CPT No. 1

1 / 1

Client:	SANJEEV K LOURA
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Job No.: 33510PD	**R.L. Surface:** ~8.3 m	**Data File:** 33510PD Warriewood
Date: 11/9/20	**Datum:** AHD	**Operator:** B.J.



Interpreted by: J.L.
Checked by: D.S.

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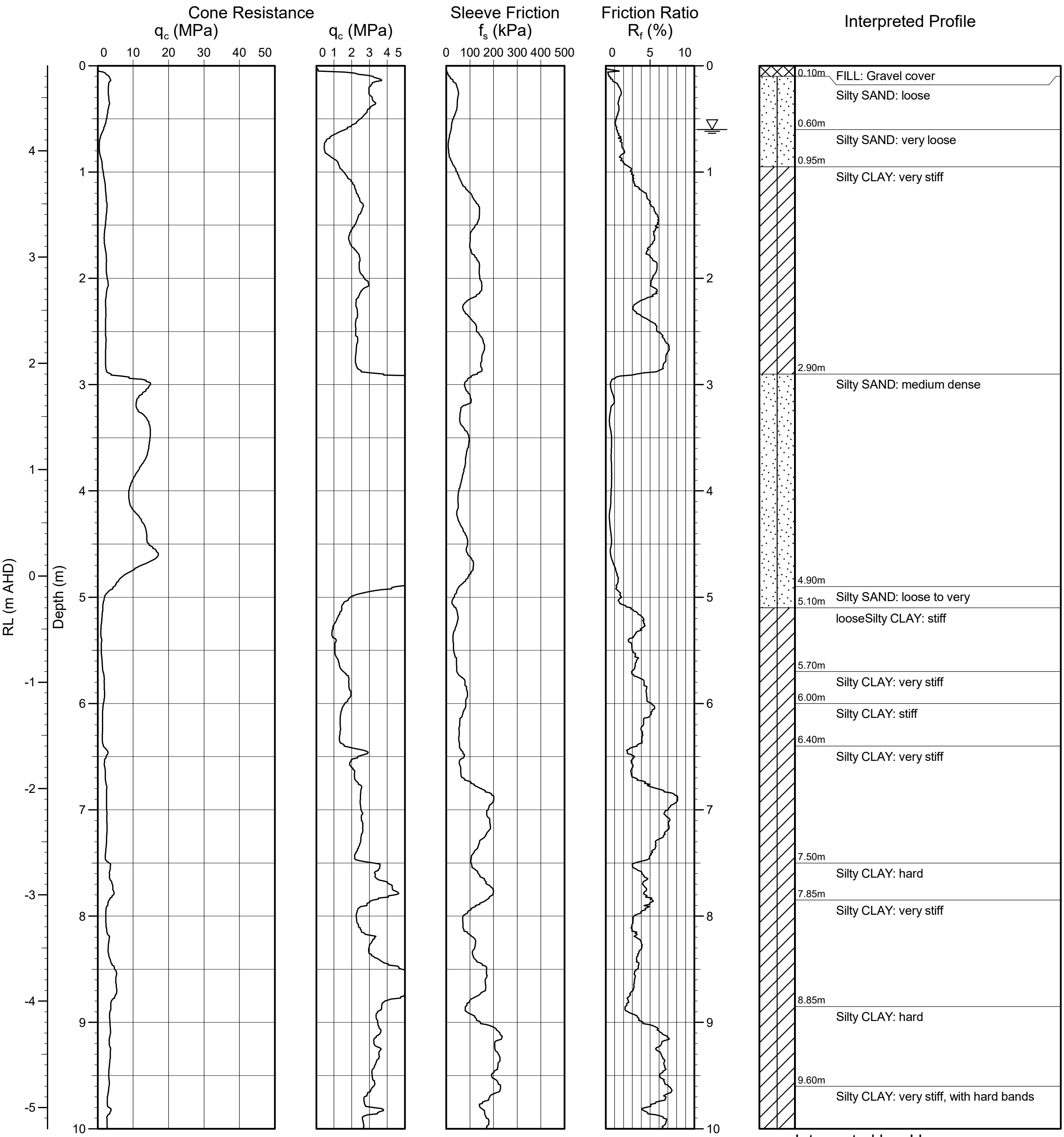
CONE PENETROMETER TEST RESULTS

CPT No. 2

1 / 2

Client:	SANJEEV K LOURA
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Job No.: 33510PD	**R.L. Surface:** ~4.8 m	**Data File:** 33510PD Warriewood
Date: 11/9/20	**Datum:** AHD	**Operator:** B.J.



Interpreted by: J.L.
Checked by: D.S.

JKGeotechnics

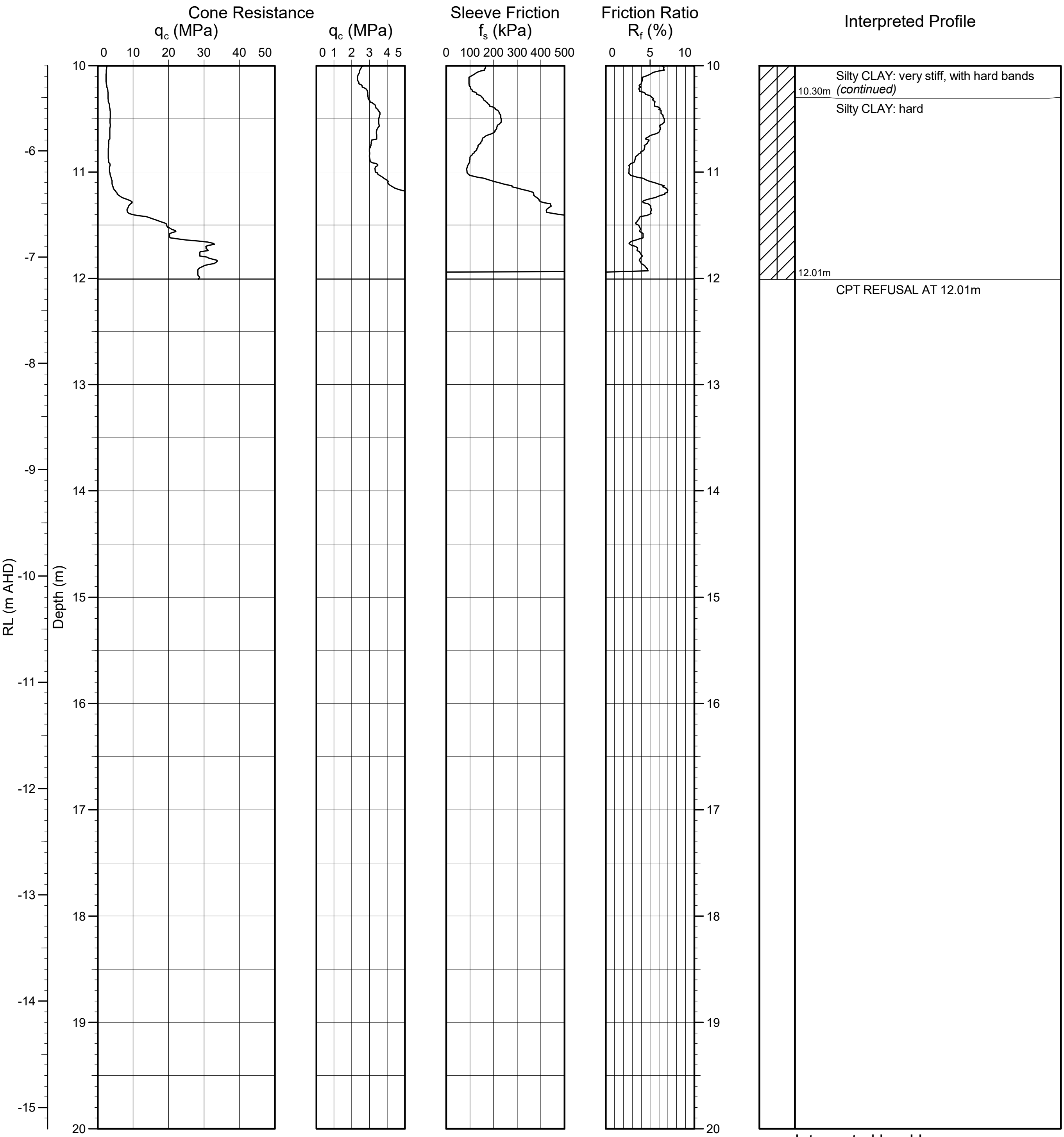
CONE PENETROMETER TEST RESULTS

CPT No. 2

2 / 2

Client:	SANJEEV K LOURA
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Job No.: 33510PD	**R.L. Surface:** ~4.8 m	**Data File:** 33510PD Warriewood
Date: 11/9/20	**Datum:** AHD	**Operator:** B.J.



Interpreted by: J.L.
Checked by: D.S.

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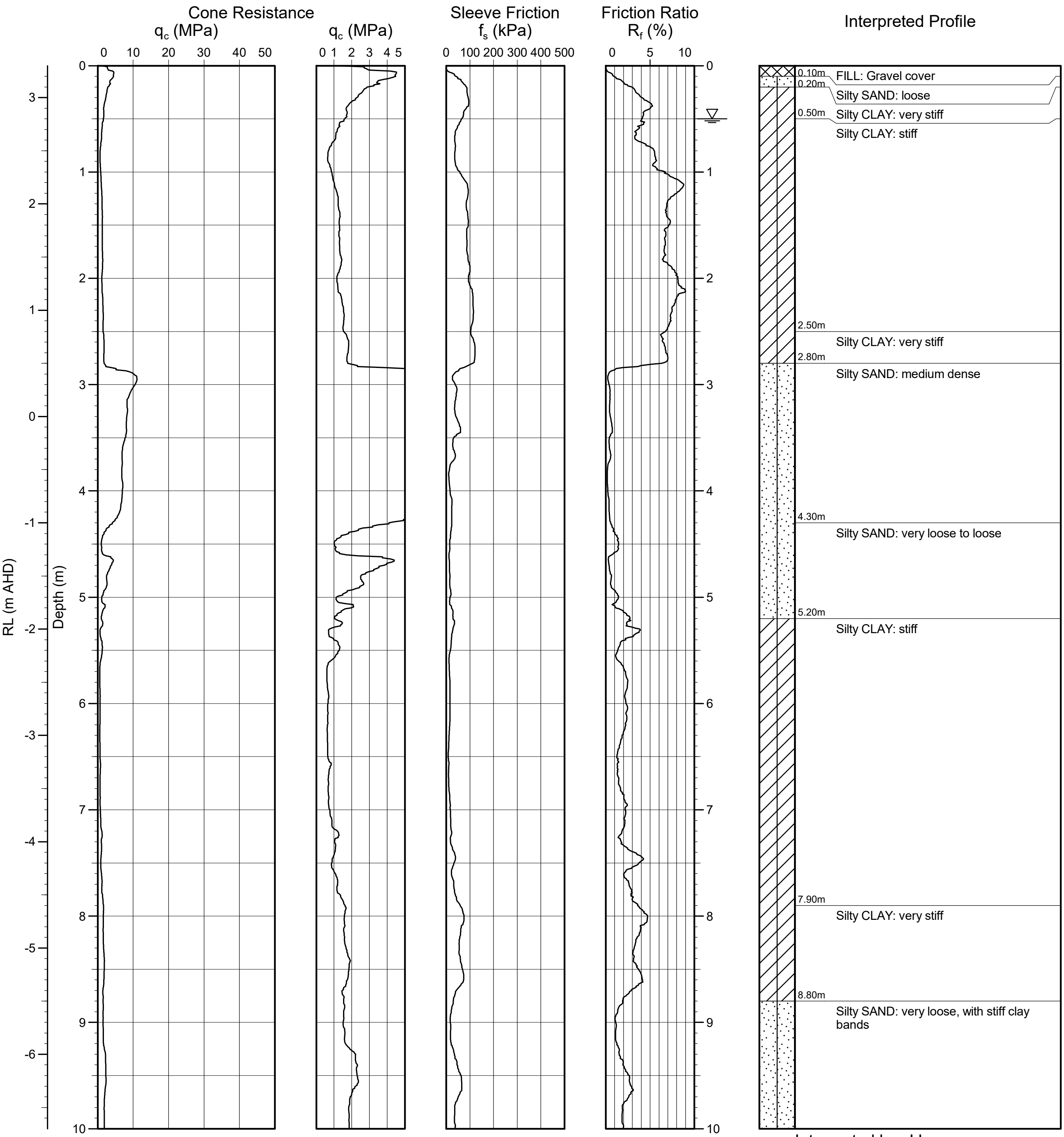
CONE PENETROMETER TEST RESULTS

CPT No. 3

1 / 2

Client:	SANJEEV K LOURA
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Job No.: 33510PD	**R.L. Surface:** ~3.3 m	**Data File:** 33510PD Warriewood
Date: 11/9/20	**Datum:** AHD	**Operator:** B.J.



Interpreted by: J.L.
Checked by: D.S.

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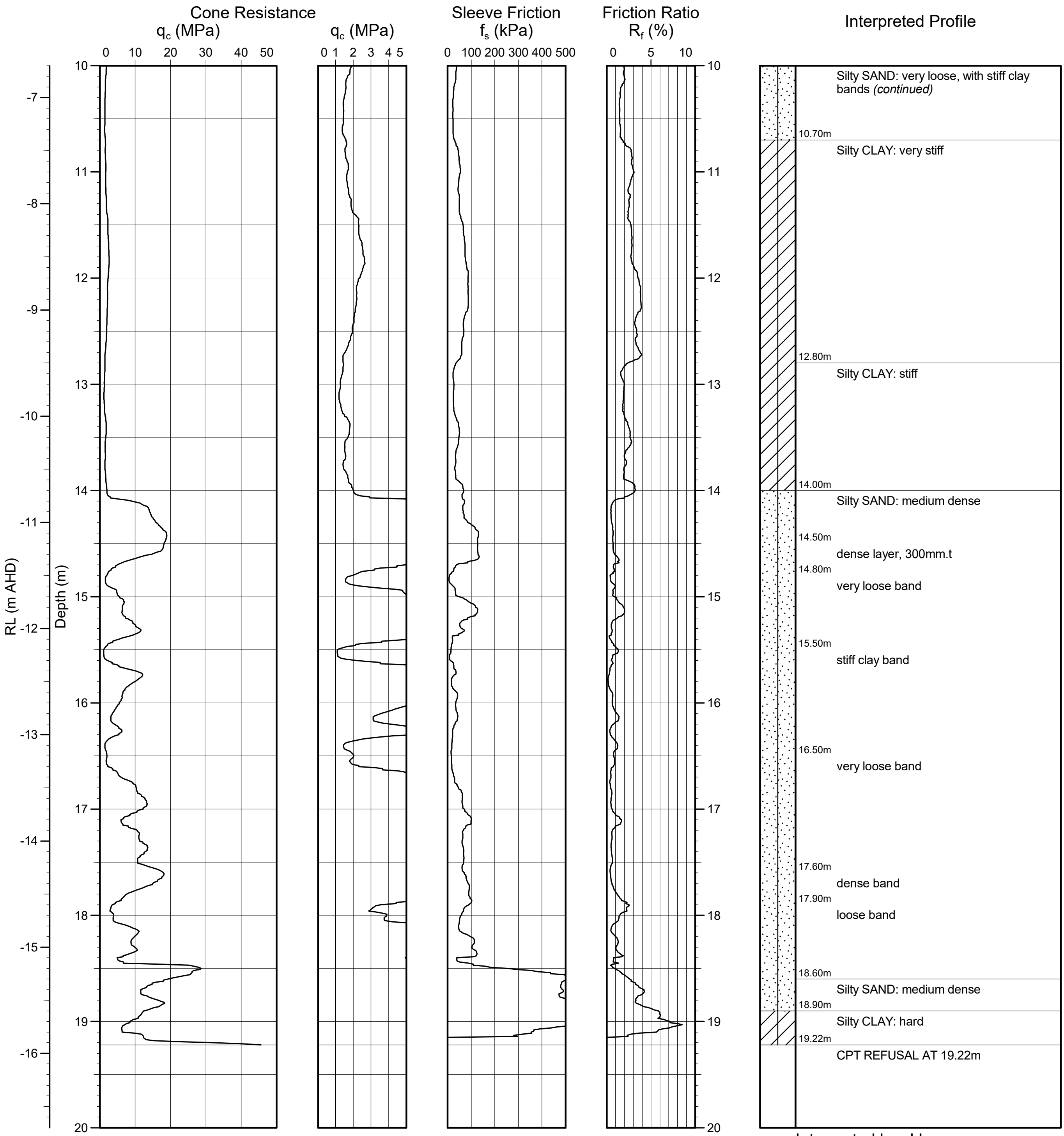
CONE PENETROMETER TEST RESULTS

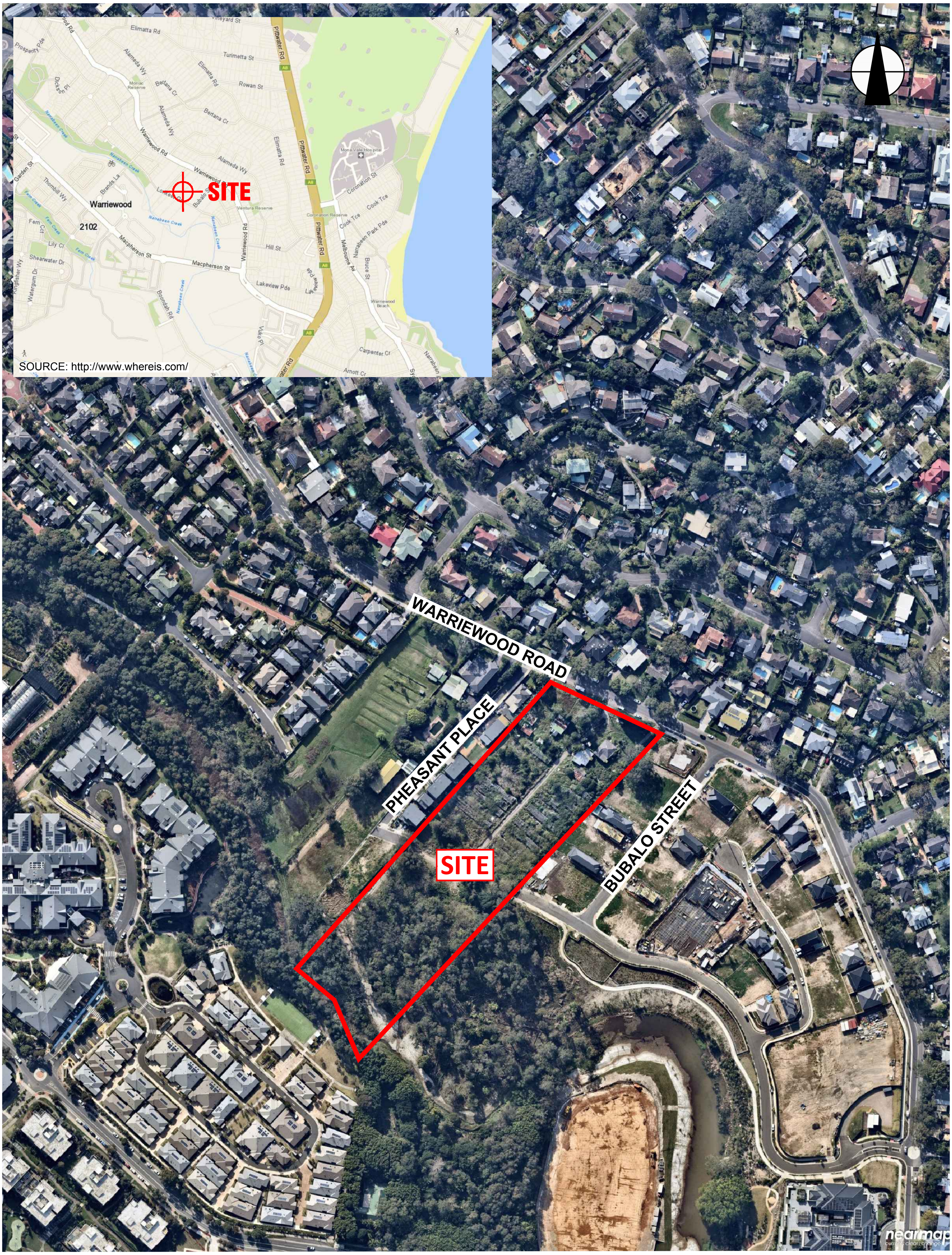
CPT No.
3
2 / 2

Client: SANJEEV K LOURA
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Job No.: 33510PD **R.L. Surface:** ~3.3 m **Data File:** 33510PD Warriewood
Date: 11/9/20 **Datum:** AHD **Operator:** B.J.



Interpreted by: J.L.
Checked by: D.S.



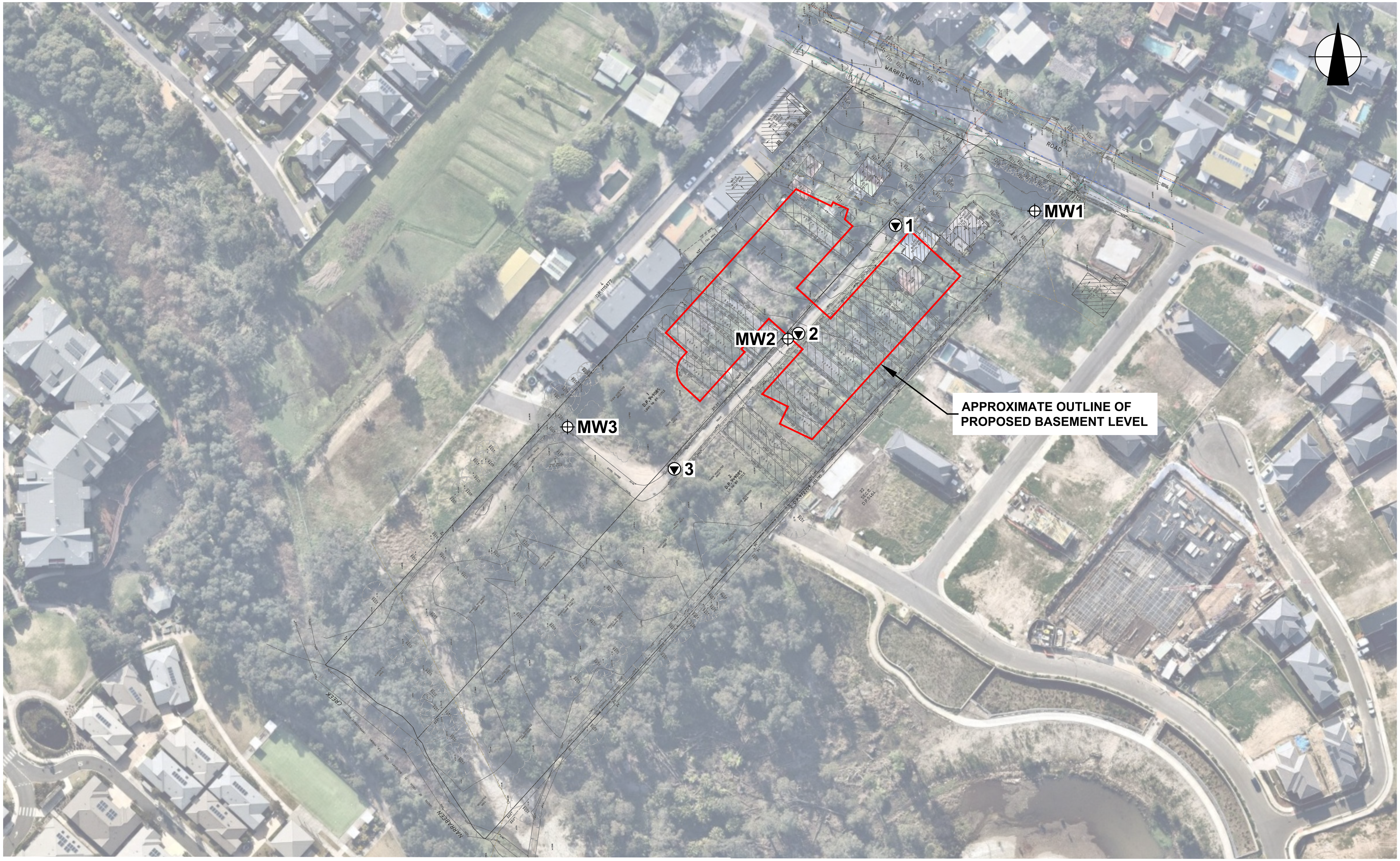
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW
Report No: 33510PD	Figure No: 1

JKGeotechnics

PLOT DATE: 23/09/2020 10:35:34 AM DWG FILE: Y:\33000's\33510PD WARRIEWOOD\CAD\33510PD.DWG



LEGEND

- CONE PENETROMETER TEST
- MONITORING WELL

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 12 24 36 48 60

SCALE 1:1200 @A3 METRES

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

Title:

INVESTIGATION LOCATION PLAN

Location: 45-49 WARRIEWOOD ROAD, WARRIEWOOD, NSW

Report No: 33510PD

Figure No: 2

JKGeotechnics

PLOT DATE: 23/09/2020 11:20:40 AM DWG FILE: Y:\33000'S\33510PD WARRIEWOOD\CAD\33510PD.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

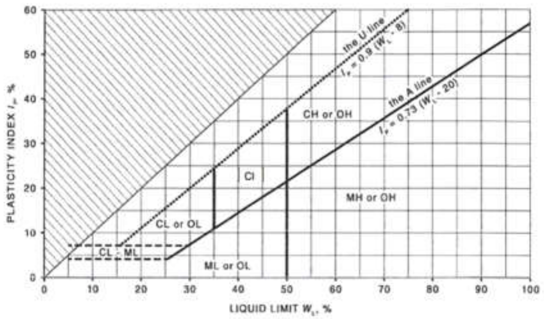
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼ C ▶	Standing water level. Time delay following completion of drilling/excavation may be shown. 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) (Coarse Grained Soils)	*w* > PL *w* ≈ PL *w* < PL *w* ≈ LL *w* > LL D M W	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. 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 $\leq$ 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.

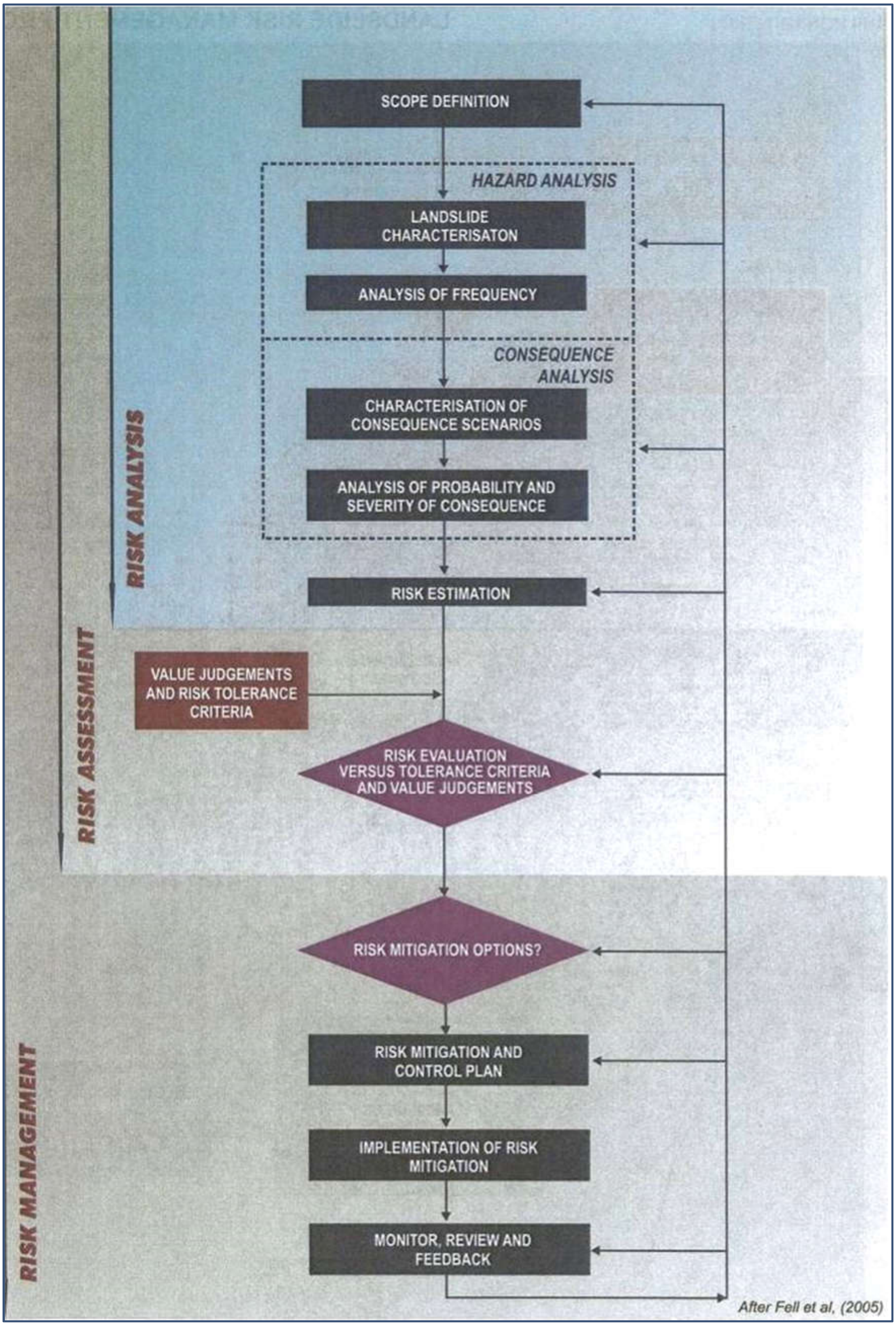


FIGURE A1: Flowchart for Landslide Risk Management.

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.

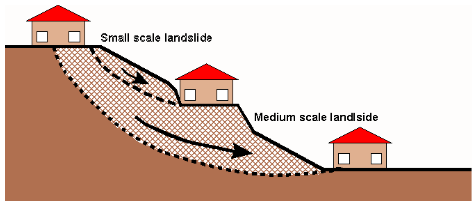


Figure 1

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.

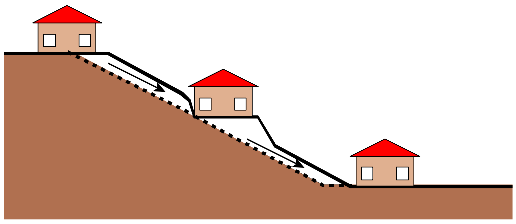


Figure 2

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.

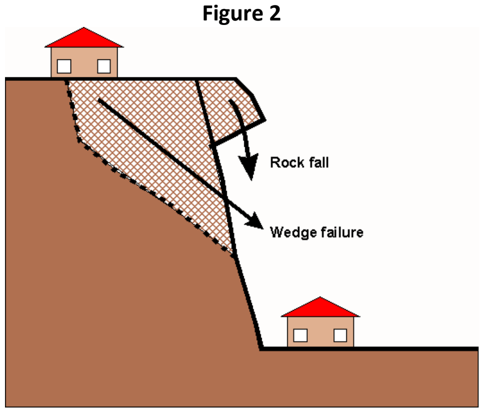


Figure 3

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.

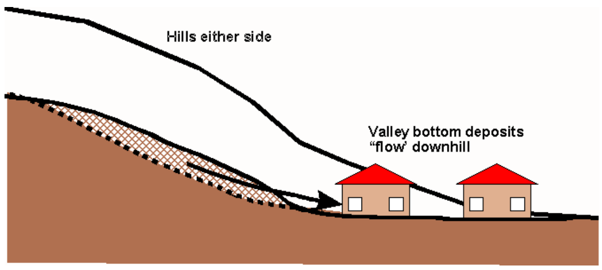


Figure 4

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:

- 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.

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for Sanjeev Loura

Name of Applicant

Address of site 45 to 49 Warriewood Road Warriewood NSW

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I Peter Wright *on behalf of* *JK Geotechnics Pty Ltd.*
(Insert Name) (Trading or Company Name)

on this the 25 September 2020, I certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million.
*we/*I have:

Please mark appropriate box

☐ Prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☒ ~~I~~ *Are*/am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ Have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. *We*/I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ Have examined the site and the proposed development/alteration in detail and *are/*am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my/*our* report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.

☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Investigation

Report Date: 17 November 2021 *Report Ref No: 33510PDrpt Rev1*
:
Author: *David Schwarzer*

Author's Company/Organisation: *JK Geotechnics Pty Ltd*

Documentation which relate to or are relied upon in report preparation:

Architectural drawings (Drawing Nos. A04^{06}, A05^{06}, A12a^{05}, A12a-a^{01}, A12a-b^{01}, A12b^{05}, A12b-a^{01}, A12b-b^{01} dated 21 May 2021)) Prepared by Archidrome.
Survey plan (Drawing Name: 15843detail, Sheet 1, dated 7 December 2016) prepared by C.M.S. Surveyors Pty Ltd.
Addendum geotechnical advice report (Ref: CA/20/126-2707, dated 23 April 2020) prepared by NG Child & Associates.

~~I am~~ *We are* aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ~~ensuring~~ *confirming* that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk, *as discussed in the Report*.

Signature P. Wright.

Name Peter Wright

Chartered Professional Status MIEAust; CPEng

Membership No. 446230

Company: *JK Geotechnics Pty Ltd.*

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements for Geotechnical Risk Management Report for Development Application

Development Application for Sanjeev Loura	
Name of Applicant	
Address of site 45 to 49 Warriewood Road Warriewood NSW	

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Investigation	
Report Date: 17 November 2021	*Report Ref No:33510PDrpt Rev1*
Author: David Schwarzer	
Author's Company/Organisation: *JK Geotechnics Pty Ltd*	

Please mark appropriate box

N/A Comprehensive site mapping conducted Not Applicable – surface slope is less than 5°
(date)
N/A Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
☐ Subsurface investigation required

- ☒ No Justification, surface slope is less than 5°
- ☐ Yes Date conducted 9 September 2020

N/A Geotechnical model developed and reported as an inferred subsurface type-section
☒ Geotechnical hazards identified

- ☒ Above the site
- ☒ On the site
- ☒ Below the site
- ☒ Beside the site

☒ Geotechnical hazards described and reported
☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater – 2009

- ☒ Consequence analysis
- ☒ Frequency analysis

☒ Risk calculation
☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the ~~specified conditions are achieved~~ *recommendations presented in the Report are adopted.*
☒ Design Life Adopted:

- ☒ 100 years
- ☐ Other …………………………………………….
  specify

☒ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
☒ Additional action to remove risk where reasonable and practical have been identified and included in the report.
N/A Risk assessment within Bushfire Asset Protection Zone.

~~I am~~ *We are* aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ~~ensuring~~ *confirming* that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk *as discussed in the Report*.

Signature	P.Wright.
Name	Peter Wright
Chartered Professional Status	MIEAust CPEng
Membership No.	446230
Company	*JK Geotechnics Pty Ltd*