GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for Reform Projects Pty Ltd
Name of Applicant
Address of site 14 Ocean Road, Palm Beach, NSW

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

I, Jarett Mones *on behalf of* JK Geotechnics Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 15 October 2021 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 $10million.

I:
Please mark appropriate box

☒ have 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

☒ 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. 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 I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

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

Geotechnical Report Details:

Report Title: **Geotechnical Investigation and Stability Assessment**
Report Date: **15 October 2021**
:
Author: **Jarett Mones**
Author's Company/Organisation: **JK Geotechnics Pty Ltd**

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

Architectural Drawings prepared by Mathieson (Ref: Drawing Nos. DA.00 to DA.15, Revision A, dated 22 September 2021)
Survey Drawings by CMS Surveyors Pty Ltd (Ref: Drawing No. 20223detail, Issue 1, dated 27 April 2021)

I am 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 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.

Signature … Jarett Mones ……………………………………….

Name …**Jarett Mones**……………………………………..

Chartered Professional Status…**Chartered**……………………………

Membership No. **3859881** ……………………………………………

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 Reform Projects Pty Ltd
Name of Applicant
Address of site 14 Ocean Road, Palm Beach, 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 and Stability Assessment**
Report Date: **15 October 2021**
Author: **Jarett Mones**
Author's Company/Organisation: JK Geotechnics Pty Ltd

Please mark appropriate box

☒ Comprehensive site mapping conducted **20 August 2021**
(date)
☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
☒ Subsurface investigation required
- ☐ No Justification ……………………………………………………...
- ☒ Yes Date conducted …**3 August 2021**…………………………

☒ 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.
☒ 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 aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring 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.

Signature ………… Jarett Mones ………………………………..
Name …………**Jarett Mones**……………………………………..
Chartered Professional Status……**Chartered**……………………………
Membership No. ……**3859881**………………………………….
Company……**JK Geotechnics Pty Ltd**……………………………

REPORT TO

REFORM PROJECTS PTY LTD

ON

GEOTECHNICAL INVESTIGATION AND STABILITY ASSESSEMENT
(In Accordance with Pittwater Council Risk Management Policy)

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

14 OCEAN ROAD, PALM BEACH, NSW

Date: 15 October 2021
Ref: 34272YJrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of our geotechnical investigation and stability assessment of the site at 14 Ocean Road, Palm Beach, NSW. The location of the site is shown in Figure 1. The geotechnical investigation and stability assessment was commissioned by Cassandra Gleeson of Reform Projects Pty Ltd, and was carried out in accordance with Option 1 of our fee proposal (Ref: P54422YJ, dated 1 July 2021).

Based on the architectural drawings prepared by Mathieson (Ref: Drawing Nos. DA.00 to DA.15, Revision A, dated 22 September 2021), we understand that following demolition of the existing residence, it is proposed to construct a four-storey house with a garage (with pool above) at the front of the property. The lower ground, ground and level 1 (including a terrace) proposed finished floor levels are RL5.05m, RL9.83m and RL13.23m, respectively. This will require cutting into the hillside as the levels step up the slope with excavation to maximum depths of about 8.5m, 7m and 4m for the lower ground, ground and level 1 floor levels, respectively. Localised deeper excavation will be required for a lift overrun, buried services and possibly an on-site detention (OSD) system. The approximate footprint of the proposed development is indicated on Figures 2 and 3.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 5 below. It is understood that the report will be submitted to Council as part of the DA documentation. Our report is preceded by the completed Council Forms 1 and 1a.

2 ASSESSMENT METHODOLOGY

2.1 Walkover Survey

This stability assessment which was carried out by our Associate, Mr Jarett Mones, is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 6 following our geotechnical assessment.

The attached Figure 3 presents a plan showing notable geotechnical site features and geotechnical hazards. Figure 3 is based on the survey plan prepared by CMS Surveyors Pty Ltd (CMS, Ref: Drawing No. 20223detail, Issue 1, dated 27 April 2021). Additional features on Figure 3 have been measured by hand held clinometer and tape measure techniques and hence are only approximate. The south-western portion of the hillside shown highlighted green in Figure 3 was generally inaccessible due to dense vegetation. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 6 defines the geotechnical mapping symbols adopted and used in Figure 3. Figure 4 presents a typical cross-section, Sections A-A', through the site based on the survey data augmented by our mapping observations. Figure 5 presents a more detailed cross-section, Section B-B', through the very large boulder at the north-western corner of the site. These sections include potential landslide hazards.

2.2 Subsurface Investigation

The fieldwork for the investigation was undertaken on 3 August 2021 and comprised the drilling of one borehole, BH1, to a total depth of 8.8m using portable drilling equipment (due to access constraints). The borehole was initially drilled using a hand auger to a depth of 2.1m and continued by portable Melvelle coring equipment using a TT56 core barrel with water flush. An additional borehole, BH2, was drilled at the front of the site to a depth of 3.0m using a hand auger. Dynamic Cone Penetration (DCP) tests were carried out adjacent to the boreholes (DCP1 to DCP2) and at an additional location (DCP3) to refusal depths of 2.35m (DCP1), 3.19m (DCP2) and 3.08m (DCP3). The purpose of the DCP tests was to assess the apparent compaction of the fill and the strength/relative density of the natural soils and to probe down to the surface of the underlying bedrock. We highlight that DCP refusal may also occur on harder bands within the soils or on 'floaters'/boulders. The apparent compaction of the clayey fill and strength of the natural clays also considered the results of the hand penetrometer testing in these materials.

The borehole locations, as shown on Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels and contour lines shown on the supplied survey plans by CMS survey plan referenced above. The datum of the levels is Australian Height datum (AHD).

Groundwater observations were made during and on completion of auger drilling. The use of water for core drilling of the bedrock limited further meaningful measurement of groundwater levels in BH1. No long-term monitoring of the groundwater levels has been carried out.

The strength of the bedrock within the cored portion of the boreholes was assessed by examination of the recovered rock core and subsequent correlation with Point Load Strength Index ($I_{S\,(50)}$) testing. The results of the Point Load Strength Index tests are presented in the attached JK Geotechnics Table C and on the cored borehole logs. The Unconfined Compressive Strength's (UCS's) were estimated from the Point Load Strength Index test results and are also summarised in Table C. Photographs of the recovered core are presented to the rear of this report with the borehole logs.

Selected samples were returned to Envirolab Services Pty Ltd (Envirolab), a NATA accredited laboratory, for pH, sulphate content, chloride content and resistivity testing. These results are presented in the attached STS Table A and Envirolab Certificate of Analysis No. 275759.

The investigation was carried out in the full-time presence of our Geotechnical Engineer, Mr Ben Sheppard, who set out the borehole locations, nominated the sampling and testing and prepared logs of the strata encountered. The borehole logs, are attached to the report together with our Report Explanation Notes, which further describe the investigation techniques adopted and their limitations, and define the logging terms and symbols used.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figure 1 (Site Location Plan), Figure 3 (Plan of Notable Geotechnical Features and Geotechnical Hazards) and Figures 4 and 5 (Sections A-A’ and B-B’). Plates 1 to 7, following our summary of observations below, present photographs of some of the relevant site features, including some of the main geotechnical hazards identified.

For ease of the site description, we have adopted site north using Ocean Road as east.

The site is located across from the beach along the lower portion (and at the base) of a south-east facing, steeply sloping hillside. The existing driveway, house and garage are located over the eastern portion of the site and have been cut into the hillside, predominantly the north-western corner of the house. Along the northern boundary adjacent to the house are irregular sandstone block masonry walls that have a maximum height of 2m and support the site to the north and large boulders that range in height to between 2m and 4m. One of these boulders extends into the garage. South of this boulder and within the garage are composite retaining walls (comprising irregular sandstone block masonry and brick masonry walls) that have a maximum height of 3m and support the rear or western side of the site. At the western side of the garage but below or to the east of the wall discussed above is a concrete masonry retaining wall with a maximum height of 2m. These walls appeared to be in good condition. Across the southern boundary, levels were generally the same with the exception of a sandstone block masonry retaining wall that had a maximum height of about 1.8m and supported the site along a length of about 5m. This wall similarly appeared to be in good condition.

To the rear of the southern portion of the house is a suspended timber deck/gazebo surrounded by an irregular rough-hewn sandstone block retaining wall with a maximum height of 1.5m and a timber retaining wall with a maximum height of 1.6m. A block had become loose in the sandstone block retaining wall and it appears that it may detach at any time without warning. The timber wall appears to be in good condition. The retaining walls have been classified as Hazard D. Hazard D has been sub-divided based on the types of the walls as follows, D1a for rough-hewn sandstone block retaining walls, D1b for sandstone block/brick masonry retaining walls, D2 for the concrete masonry retaining wall and D3 for the timber retaining walls.

From the rear of the property the site steps/slopes down towards the south-east and the existing house through a series of vertical sandstone bedrock rock ledges/cliff-line (Hazard A) and steep slopes, varying up to about 45° (Hazard C). The slopes comprise soil and extremely to highly weathered rock of soil strength to medium strength. A cliff-line on the southern side is estimated to have a maximum height of 3.5m to 4m high. The upper portion of the cliff-line and area upslope is densely vegetated and inaccessible. The cliff-line was assessed to be highly weathered and of low to medium strength. Some blocks of rock were isolated by joints/bedding partings in the cliff-line (Hazard A), and is discussed in more detail in Section 3.1. Some low height rock ledges were also observed and assessed to be of medium strength. Loose boulders ranging in diameter of 0.5m to 1.5m (Hazard B1), were observed on the surface of the slopes/bedrock outcrops. At the north-western boundary there is a very large sandstone boulder, with dimensions of ˜8m (north to south) x 5.5m (west to east) x maximum 5m (high, measured from the ground surface). The boulder is embedded into the ground which is sloping down at about 25° to 30°.

The site is bound by residential development to the north, west and south that have similar land forms to that of the site. These are two-storey residential developments that are within about 1m to 1.5m of the northern and southern boundaries and set-back about 10m to 15m from the western site boundary. All adjoining properties appeared in good structural condition when viewed from the site. To the east the site is bound by Pacific Road and the Palm Beach.

Plate 1 - Hazard A (3 Photographs)

Plate 2 - Hazard A, B1, B2 and C (2 Photographs) – Taken from downslope

Plate 3 - Hazard B1 (3 Photographs)

Plate 4 - Hazard B2 (2 Photographs)


Sewer

Plate 5 - Hazard D1a and D1b and D3 (2 Photographs)

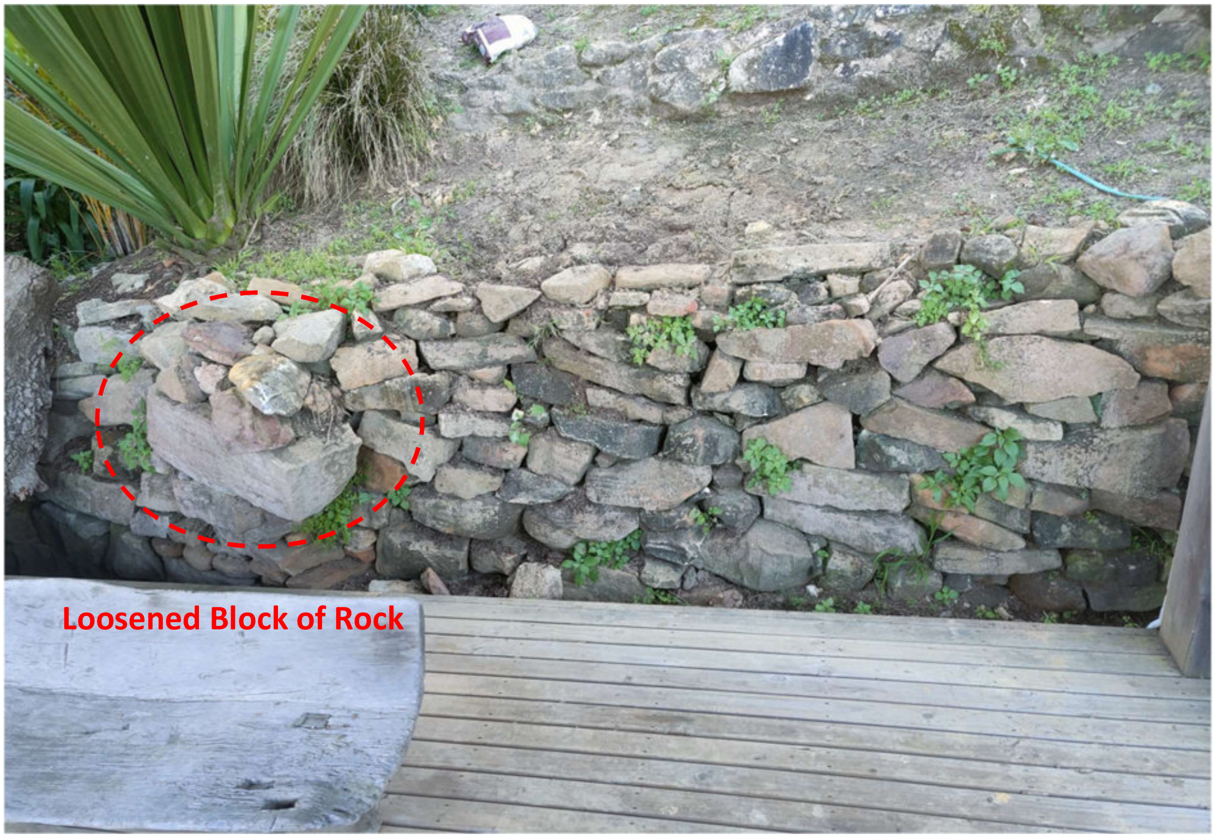
Loosened Block of Rock

Plate 6 - Hazard D1b and Large Boulders on Northern Boundary (1 Photograph)

Plate 7 - Hazard D1b and D3 and Large Boulders in garage (1 Photograph)

4 GEOLOGY AND SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by the Newport Formation, 'Narrabeen Group', which comprises interbedded laminite, shale and quartz, to lithic quartz sandstone. The western portion (or upslope) of the site is near the boundary with the Hawksbury Sandstone unit, and east of the site is the Quaternary aged sand unit.

Based on our site observations and boreholes, the subsurface conditions generally comprised shallow or outcropping weathered sandstone bedrock on the hillside and at the toe of the hillside we encountered fill and then residual clay overlying weathered interbedded siltstone and sandstone bedrock. Fill overlying sands was encountered to the east that in turn overlay inferred bedrock. As discussed in our site description, numerous small to large sized sandstone boulders, 0.5m to 1.5m diameter, were observed on the hillside and at the north-western corner of the site there is a very large sandstone boulder. Very large sandstone boulders were also observed at the northern boundary along the existing house footprint.

Some of the more pertinent details of the strata encountered are described below. For further details of the conditions encountered at each borehole location, reference should be made to the attached borehole logs.

Pavements

There were concrete pavements around the house.

Fill

Fill was encountered to depths ranging from 1.0m (BH1) to 1.5m (BH2). At BH1, the fill comprised sandy clay and included traces of sandstone cobbles and slag fragments. At BH2, the fill comprised sandy clay and clayey sand. The fill appeared to be poorly compacted.

Residual Soils

Underlying the fill at BH1, residual silty clay was encountered to a depth of 2.1m. It is likely that the 540mm thick 'No Core' zone below this depth either represents the residual clay unit or poor quality bedrock that has been 'washed' out during the coring process. The residual clay was assessed to be of firm to very stiff strength.

Marine Soils

Underlying the fill at BH2, marine soils comprising sand that was assessed to be of loose to medium dense relative density was encountered that extended to a depth of 3.0m. Traces of shale gravel was noted within the sand.

Weathered Bedrock

Weathered interbedded siltstone and sandstone bedrock was encountered at a depth of 2.65m in BH1. The bedrock was typically assessed to be highly weathered and of low to medium strength, however a slightly weathered and medium to high strength band was encountered between depths of 7.82m and 8.25m. The bedrock included several 'No Core' zones, ranging in thickness between 0.34m and 0.65m. The 'No Core' zones below the start of the core (i.e. 2.65m) likely represent extremely weathered bands that have been washed away during the coring process, although it is possible that they may also indicate residual clays. Defects within the rock mass generally comprised extremely weathered seams (typically 30mm to 45mm thickness) and numerous sub-horizontal bedding partings and steeply inclined joints. Bedrock was inferred at the DCP refusal depths of 3.19m (DCP2) and 3.08m (DCP3). However, we highlight that premature DCP refusal may also occur on harder bands within soils or on 'floaters'/boulders. Therefore, investigations and/or inspections of piling will be required during construction to confirm that refusal has occurred on the bedrock unit.

Weathered sandstone bedrock was outcropping on the hillside, including a 2m to 4m high sandstone cliff-line, assessed to be highly weathered and of low to medium strength. Within the cliff-line three adverse joints, dipping down at about 60° and 80°, were observed. The joints intersect bedding partings and have isolated blocks of rock within the cliff-line.

Previous projects we have worked on in the vicinity of the site have exposed a heavily jointed rock mass. These joints typically dip in the direction of the hillside (i.e. to the east) with orthogonal jointing resulting in the isolation of discrete blocks.

Groundwater

Both boreholes were dry on completion of auger drilling to depths of 2.1m (RL9.5m) and 3.0m (RL2.3m). However, it should be noted that during the coring process water is introduced into the borehole and thus the water level measured on completion of coring is not considered to represent the level of the groundwater.

Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores from BH1 correlated well with our field assessment of bedrock strength. Point Load Strength Index ($I_{s\,(50)}$) tests ranged from 0.1MPa to 1MPa or an estimated unconfined compressive strength (UCS) of 2MPa to 20MPa.

The results of the pH, sulphate, chloride and resistivity tests are summarised in the table below and results are attached in the Envirolab Certificate of Analysis No. 275759. Refer to Section 6 for an interpretation of the results.

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides Cl (ppm)	Resistivity ohm.cm
BH1	0.1-0.2	Fill: Sandy Clay	6.1	20	<10	30,000
BH1	1.6-1.8	Silty Clay (Residual)	5.3	27	21	21,000
BH1	2.8-2.9	Interbedded siltstone and sandstone (bedrock)	6.0	20	<10	48,000

5 GEOTECHNICAL ASSESSMENT

5.1 Potential Landslide Hazards

We consider that the potential landslide hazards associated with the site to be the following:

A Blocks detaching from the cliff line;

B1 Existing small to large sized boulders, 0.5m to 1.5m diameter;

B2 Existing very large boulder upslope at the north-western corner;

C Stability of the hillside;

D1a Stability of the irregular rough-hewn sandstone block retaining walls;

D1b Stability of the sandstone block/brick masonry retaining walls;

D2 Stability of the concrete masonry retaining wall;

D3 Stability of the timber retaining walls; and,

E Stability of new engineered retaining walls

These potential hazards are indicated in schematic form on the attached Figure 3.

5.2 Risk Analysis

The attached Table A1 summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Use has been made of data in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A1 indicates that the assessed risk to property is Medium, which is considered unacceptable in accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy. However, provided the recommendations provided in Section 6.1.1 and 6.3.2 are followed, the assessed risk to property can be further reduced to at least low, which is considered an acceptable risk. Following the completion of the remedial works, we recommend that our risk assessment be reviewed to reflect the works completed and the current risk level.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B1, together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 3.3×10^{-5}. Therefore, the risk is considered unacceptable in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy. However, provided the recommendations provided in Sections 6.1.1 and 6.3.2 are followed, the assessed risk to the person most at risk can be further reduced to no greater than 1×10^{-6}, which is considered an acceptable risk. As, with the risk to property, following the completion of the remedial works, we recommend that our risk assessment be reviewed to reflect the works completed and the current risk level.

We recommend that following the Clients and structural engineers review of our assessment and preliminary recommendations, that a meeting be held between JK Geotechnics, the Client and structural engineer to discuss the results of the risk assessment and remedial measures required. Following this meeting we can provide more detailed design advice for the remedial measures of these hazards.

5.3 Risk Assessment

The Pittwater Risk Management Policy requires suitable measures 'to remove risk'. It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site and/or development cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners are made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, where the policy requires that 'reasonable and practical measures have been identified to remove risk', it means that there has been an active process of reducing risk, but it does not require the geotechnical engineer to warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

Similarly, the Pittwater Risk Management Policy requires that the design project life be taken as 100 years unless otherwise justified by the applicant. This requirement provides the context within which the geotechnical risk assessment should be made. The required 100 years baseline broadly reflects the expectations of the community for the anticipated life of a residential structure and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 100-year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, the Policy does not seek the geotechnical engineer to warrant the development for a 100-year period, rather to provide a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.

Our risk assessment has considered Hazards D2 and D3 to have been previously engineered and certified during construction. We understand that the retaining wall hazards, i.e. Hazards D1a, D1b, D2 and D3, will be demolished during construction and that Hazards A, B1, B2 and C can be managed during construction. In preparing our recommendations given below, we have adopted the above interpretations of the Risk Management Policy requirements. 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 services, including the sewer on the surface near the western boundary are, and will be regularly maintained to remain, in good condition.

Provided the assumptions above are correct and the recommendations set out in Sections 6.1.1 and 6.3.2 below are followed, we consider that our risk analysis has shown that the site and existing and proposed development can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy. However, we recommend that the stability assessment be reviewed following the completion of remedial works.

6 COMMENTS AND RECOMMENDATIONS

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 of the proposed development.

We recommend that following the Clients and structural engineers review of our stability assessment and preliminary recommendations below, that a meeting be held between JK Geotechnics, the Client and structural engineer to discuss the results of the risk assessment and remedial measures required. Following this meeting we can provide more detailed design advice for the remedial measures of these hazards.

6.1 Conditions Recommended to Establish the Design Parameters

6.1.1 Assuming that Hazard D1 will be removed during construction, the existing hazards that pose an unacceptable risk to the life and/or property, i.e. Hazards B1, B2 and C, must be either treated, removed or further investigated so that a more accurate understanding of the risk posed can be made. In addition we recommend Hazard A be remediated.

Hazard A (cliff-line instability) can be addressed by clearing and exposing the face so that it can be clearly viewed and then remediated by either removing or supporting any loose or unstable zones. Where zones require support, this may be achieved by the installation of rockbolts, mesh and shotcrete.

Hazard B1 may be managed by either removing or supporting the small to large sized boulders.

Hazard B2 consists of the very large boulder that is predominantly located on the adjoining upslope property. This boulder is embedded into the soils which provide support and prevent it from rolling/sliding downslope. Should these soils erode over time, the downslope support may be lost and the boulder may be free to roll downslope. Consequently, we recommend that further investigation be completed to better understand the risk posed by this hazard. Where this risk is found to be unacceptable, stabilisation of the soils on the downslope side of the boulder may be completed to maintain support to the boulder.

The magnitude of risk posed by Hazard C, the steep hillside present above the proposed development, is dependent on the depth to bedrock. Where bedrock is shallow, the risk posed is likely to be acceptable and may be managed by the installation of low height retaining walls founded on the underlying bedrock that support the shallow soil cover. Where the soil cover is deep the slopes will be over-steep and will require some form of stabilisation. Further investigation will be required to determine the depth to bedrock with potential stabilisation measures comprising retaining walls, soil nails with soft facings or similar.

Further advice on the provision of support for this section of the site should be sought from the geotechnical engineer during the design stage once there is a clearer understanding of the retention system proposed for this portion of the site.

6.1.2 Prior to the commencement of excavation, a retention system will need to be installed. We recommend a soldier or contiguous pile wall be adopted for support of the excavation. Contiguous piles will be needed on the southern and northern boundaries where the residual clays transition to sand at the eastern or front of the site while around the remainder of the excavation where sand is not present a soldier pile wall may be adopted. Following demolition, we recommend that further investigations be carried out to assess the ground conditions, particularly where the clay and sand units intersect. This is discussed further below in Section 6.1.4. Where cut depths are less than 3m the wall may be designed as cantilevered while where excavation is greater than 3m the wall must be designed as anchored. Alternatively, consideration could be given to the adoption of a soil nail/rock bolt wall that is progressively installed as the excavation deepens. Along the northern boundary where there are large boulders that extend into the proposed excavation and across the boundary, these must be carefully removed using non-vibratory excavation methods such as rock saws, rock splitting etc. Care must be taken that cutting and removal of that portion of the boulders that extends into the proposed basement does not damage the adjoining structures, which may, in part, be founded on these boulders. We recommend that a geotechnical engineer be present during removal to assess the temporary stability of the boulders. Design parameters for retaining wall options are provided below in Sections 6.1.4 and 6.1.5.

6.1.3 Should temporary batters be adopted for localised excavations within the excavation they may be formed at no steeper than 1 Vertical (V) in 1.5 Horizontal (H) within the fill and natural marine sands and 1V:1H within the natural residual clays and extremely weathered rock, subject to inspection by a geotechnical engineer. Temporary batters in soils below the groundwater table, such as for a lift overrun or an OSD tank should not be attempted. If steeper batters are required, further advice will be necessary from the geotechnical engineer and some form of temporary support will be necessary. All surcharge loads should be kept well clear of the crest of the temporary batters. As a guide, surcharge loads should be positioned no closer than 2H from the top of any batter, where H is the vertical height of the batter, assuming a horizontal backfill. Where this is not the case further advice should be sought from this office. Where ramps are required to provide access for machinery across the site, they will need to be formed from good quality angular material. Further advice should be sought from this office for advice on the formation of ramps and working platforms, if required. Based on the footprint of the proposed house, sufficient space does not exist for the adoption of temporary batters for the proposed excavation.

6.1.4 As sufficient space does not allow for the formation of temporary batters, a retention system will be required and must be installed prior to excavation commencing. We recommend the retention system comprise the following:

- Anchored (retained heights of greater than 3m) or cantilevered (retained heights of less than 3m) soldier pile walls with reinforced shotcrete infill panels over the western portion of the site where residual clays and weathered bedrock are present. The infill panels must be progressively installed as excavation proceeds (i.e. at maximum 1.8m depth intervals). Where anchored retaining walls are adopted, the anchors should be installed progressively as the excavation deepens. As discussed above in Section 6.1.2, contiguous piles are likely to be needed on the southern and northern boundaries where the residual clays transition to sand at the eastern or front of the site. Therefore, provision in the design should be made for this. We recommend that in the early stage of construction, test pits be excavated along the southern and northern boundaries to investigate the ground conditions to confirm where the transition between the soldier and contiguous pile walls should be.
- Due to the presence of groundwater and collapsible soils, we recommend that piles be drilled using Continuous Flight Auger (CFA) techniques.
- Particular attention will need to be given that during the drilling of piles excess spoil is not removed (termed decompression). Decompression can cause a settlement bowl in the ground around the piles, and in extreme cases this settlement bowl can extend some metres from the

pile location. The site superintendent should monitor ground settlement during piling to ensure that adverse settlements do not occur. Particular care that this does not occur will be required when piling adjacent to the southern and northern boundaries. Should this prove to be an appreciable risk during construction, it can be managed by using a double rotary drilling system which rotates in temporary casing in conjunction with the auger.

- Retaining wall design parameters are provided below.

6.1.5 The proposed new retaining walls should be designed using the following parameters:

- For cantilever walls a triangular lateral earth pressure distribution should be adopted. Where movement sensitive structures are not present within the zone of influence of the excavation (which is defined as everything above a line drawn upwards from bulk excavation level at 1 V:2 H) a coefficient of active pressure, K_a, of 0.35, for the retained soils and weathered bedrock may be adopted, assuming a horizontal surface behind the wall. Where movement sensitive structures are located within the zone of influence of excavation, a coefficient of lateral earth pressure, K, of at least 0.55 should be adopted for the retained soils and weathered bedrock, assuming a horizontal surface behind the wall. Where a sloping backfill is present, the additional pressure applied to the back of the wall should be allowed for by either applying an equivalent surcharge load or a higher K value. Advice on appropriate K values may be obtained from this office once the design backslopes are known. Where software such as Wallap is used to model the retention system and a sloping backfill is present (such as at the rear of the proposed excavation), we recommend that the sloping backslope, which slopes at between 35° and 45° be modelled as a surcharge behind the wall. This can be done by drawing a line up at 1H:1V from the toe of the pile and measuring the height of the surcharge material to calculate the surcharge applied at that location. For a 3m long pile wall with a backslope of 45°, a 0kPa surcharge will be modelled immediately behind the wall and at a horizontal distance of 3m back from the wall a surcharge of 60kPa (i.e. 3m x 20kPa) will be adopted.
- Propped or anchored walls supporting soils and weathered bedrock at the eastern portion of the site should be designed to resist a trapezoidal earth pressure distribution that is uniform over the middle 50% of the distribution but then linearly decreases to 0kPa at the top and bottom of the distribution. Where movement sensitive structures are not present within the zone of influence of the excavation, a lateral pressure of 6H kPa may be adopted. Where movement sensitive structures are present within the zone of influence of the excavation, a lateral earth pressure distribution of 8H kPa should be adopted. The 35° to 45° sloping backfill must be added to the above pressures as a surcharge load, as detailed above.
- All surcharge loads, such as from the sloping ground behind the shoring wall, construction equipment, stockpiles, structures, etc. and appropriate hydrostatic pressures must be added to the above pressures.
- A bulk unit weight of $20kN/m^3$ and $22kN/m^3$should be adopted for the soil and weathered bedrock profiles, respectively.
- The soldier pile retaining walls should be provided with complete and permanent drainage of the ground behind the walls that exits at the base of the shotcrete. The subsoil drains should

incorporate a non-woven geotextile fabric (eg. Bidim A34), to act as a filter against subsoil erosion.

- Toe resistance of the wall may be achieved by keying the footing into the weathered bedrock below bulk excavation level. As the level of the house step down to the east and the depth of excavation decreases, bedrock levels will need to be carefully checked in the design and on site during construction to confirm that the ground conditions assumed in the design match those on site. An allowable lateral restraint of 200kPa may be adopted for piles socketed into bedrock of at least very low strength. This assumes that full passive restraint can be mobilised in the rock and that features such as excavations in front of the walls do not reduce the available capacity. In this regard, we recommend that when calculating the required depth of embedment needed for lateral restraint the first 0.5m of the socket below bulk excavation and all localised excavations should be ignored. This allows for the potential for some over-excavation. All retaining wall designs should be reviewed by a geotechnical engineer prior to construction to confirm that appropriate design values have been adopted.
- Anchors may be designed based on an allowable bond strength of 60kPa in bedrock of at least very low strength. Temporary anchors used for lateral support should be bonded beyond a line drawn up at 45° from the bulk excavation level. All anchors should be proof stressed to at least 1.3 times their working load and then locked off at about 80% of the working load.
- Where temporary anchors extend below adjoining properties, permission and easements must be obtained from the respective property owners before installation.
- Long term support is understood to be provided by the built structure. Once constructed, temporary anchors will then be destressed.

We have found that detailed retaining wall designs using geotechnical software such as WALLAP and PLAXIS can produce more economical wall designs than by using the apparent earth pressure recommendations above. However, WALLAP, does not have input parameters to review scenarios where there are adverse joints, large wedges etc, and therefore is not well suited for the design of retaining walls supporting bedrock. PLAXIS can be used to model such scenarios. We consider that the following preliminary geotechnical design parameters could be adopted for shoring wall design using such software packages. However, as noted above, only designers with experience in modelling jointed rock masses should undertake this design. It is our expectation that the bedrock will be heavily jointed and that adversely orientated joints will be present.

Preliminary Shoring Wall Design Parameters Note 2					
Material Type	**Unit Weight (above/below GW) (kN/m^3)**	**Effective Friction Angle (degrees)**	**Effective cohesion (kPa)**	**Elastic Modulus (MPa)**	**Poisson's Ratio**
Fill: Sandy Clay and Clayey Sand	18/20	27	0	7	0.3
Residual Clay, Stiff to Very Stiff	17/19	28	3	15	0.3
Marine Sands, Loose to Medium Dense	18/20	32	0	17.5	0.3
Weathered Bedrock Note 1	22/23	30	30	100	0.3

Note on table above:

Note 1: The above assumes an intact rock mass free from adverse defects, such as adversely orientated joints forming large wedge failures etc. The designer must consider the support requirements necessary should adverse defects be present and ensure that the design can either accommodate these conditions or put measures in place to allow this risk to be managed during construction. Such measures may comprise a carefully staged and managed excavation sequence that allows the geotechnical engineer to inspect the rock mass for the presence of such defects prior to the removal of the support provided by the rock. Where adverse defects are identified, provision for the installation of additional temporary support in the form of temporary anchors, props etc. must be made. To allow the rock mass to be inspected, slots in the bedrock will need to be excavated perpendicular to the cut lines so that the rock mass can be inspected by a geotechnical engineer prior to the completion of bulk excavation. Further advice on this staging can be provided should this approach be adopted.

Note 2: The 35° to 45° sloping backfill present behind the western retaining wall must be added to the above pressures as a surcharge load, as detailed below.

6.1.6 Excavation will require the removal of clayey fill, sand, residual clay and weathered bedrock, which includes low to medium strength rock. High strength rock should also be expected. Excavation of the soil and weathered bedrock of up to low strength should be achievable using conventional excavation equipment, such as medium sized excavators (say 15 to 20 tonnes) with buckets with "tiger teeth" attached. Where the bedrock is of low or higher strength, "hard rock" excavation techniques will be required.

"Hard rock" excavation techniques may consist of percussive or non-percussive techniques. Percussive techniques comprise the use of rock hammers, while non-percussive techniques comprise rotary grinders, rock saws, ripping, rock splitting etc. Where percussive excavation techniques are adopted there is the risk that transmitted vibrations may damage nearby movement sensitive structures such as the residential buildings to the south and north. Where percussive excavation techniques are used continuous vibration monitoring will be necessary. The prescribed vibration limits that should be adopted are set out in the Vibration Emission Design Goals attached to the rear of this report. Particular care will be required to keep vibrations as low as possible to reduce the risk of damage to these structures. Consequently, it is our recommendation that percussive rock excavation techniques not be adopted. Where difficulties are experienced ripping the rock, closely spaced line sawing used in conjunction with ripping will be necessary.

6.1.7 It is our opinion that the water measured in BH1 on completion of coring is not the true groundwater level but reflects the water introduced during the coring process. Groundwater was not encountered on completion of augering in BH2 which was drilled to a depth of 3.0m (~RL2.3m). We anticipate groundwater will be below the base of the lower ground floor level and may be between about RL1m and RL2m. Further investigations will be required to confirm the above. We anticipate that the groundwater levels may fluctuate by at least 1m, especially following extended periods of wet weather and climatic changes. According to the Northern Beaches Council database, Ocean Road to the east is a 'low flood risk precinct' but close to a 'medium flood risk precinct' further south.

- A 'Low Flood Risk Precinct' is defined as covering 'flood prone land affected by the Probable Maximum Flood (PMF) but which is outside the Medium Flood Risk Precinct. The PMF is equivalent to the largest ever conceivable flood.'
- A 'Medium Flood Risk Precinct' is defined as 'equivalent to the Flood Planning Area, and covers all flood prone land which is affected by the 1% Annual Exceedance Probability (AEP) flood (equivalent to the 1 in 100 year flood) with a freeboard added'.

With regards to climate change related groundwater level rise, expert advice should be obtained. As a preliminary guide, global sea levels are anticipated to increase by between 45cm and 88cm by 2090 (Ref: CoastAdapt, Information Manual 2, National Climate Change, 2016), although it is likely that local variations will occur along the coastline.

Based on our investigation and the Council database, groundwater will be below bulk excavation level for the lower, ground and first floor finished floor levels of RL5.05m, RL9.83m and RL13.23m, respectively. To collect any seepage that makes its way into the basement excavation, we recommend that a dish drain be installed around the perimeter of the excavation and a drainage blanket be installed below the floor slabs. All water collected in the dish drain and drainage blanket should be discharged to Council's stormwater system in a controlled manner. Alternatively, an absorption system at the eastern side of the site could be considered but infiltration testing will be required to assess the feasibility of such a system.

Where deeper excavations, such as for the lift overrun, OSD system, subsurface drainage, etc., are required, it is anticipated that all water inflows will be able to be controlled by sump and pump during construction. It is anticipated that the lift overrun will be tanked.

6.1.8 All proposed footings must be founded on the underlying bedrock. Due to the varying depth to bedrock across the site, we anticipate that bedrock will only be exposed at bulk excavation level in the north-western portion of the site. Depth to bedrock will increase towards the front or eastern side of the site. While pad or strip footings may be adopted where bedrock is shallow, where bedrock is deeper than about 1m, pile footings are likely to be required. Where pile footings are adopted and are socketed a minimum of 0.3m into weathered bedrock of at least very low strength an allowable bearing pressure (ABP) of 700kPa may be adopted, subject to inspection of pile drilling by a geotechnical engineer. Where piles extend below a nominal 0.3m socket into rock, allowable shaft adhesions of 10% and 5% of the ABP may be adopted for compressive and tensile (uplift) loads, respectively. This assumes that the rock socket is suitably roughened. Due to the presence of groundwater, sandy soils and likelihood that the weathered bedrock will soften after drilling, CFA grout injected piles will be required.

Pad footings founded on weathered bedrock of at least very low strength may be designed for an allowable bearing pressure of 700kPa, subject to inspection by a geotechnical engineer prior to pouring concrete. Where footings are located close to the edge of excavations, such as where the excavation steps up between levels or for localised excavations such as for the lift overrun, OSD tank, buried services, etc. they must be wholly founded below a line drawn upwards from the base of the lower excavation at 1V:1H. Since there are some uncertainties regarding the depth of the rock profile east of BH1, we recommend that additional investigation be completed following demolition to confirm and better define the rock profile. This additional investigation may comprise the drilling of additional boreholes, some test piles or shallow test pits in the presence of a geotechnical engineer

6.1.9 The surface water discharging from the new roof and paved areas must be diverted to outlets for controlled discharge to Council's stormwater system. In addition, a drainage system should be installed to the west of the proposed residence to intercept surface water run-off from upslope and connect to the stormwater system. The drainage system installed behind retention systems should also connect to Council's stormwater system.

6.1.10 At bulk excavation we anticipate there will be weathered bedrock over the north-western portions of the floor levels grading to residual clayey soils, sandy soils and then uncontrolled fill as the excavation moves to the east. This means that where slabs on grade are adopted that they will span a number of different materials.

In addition to the lower ground floor slab, the western portion of both the ground floor slab and first floor slab extends out beyond the footprint of the level below and will be formed over the exposed subgrade. However, as the remainder of the ground and first floor slabs are fully suspended, we recommend that where these slabs span out over the subgrade that they also be designed as fully suspended slabs and not as slabs on grade.

As uncontrolled fill to depths of about 1m is likely to be exposed at bulk excavation over at least a portion of the lower ground floor level, for certainty of performance we recommend that the whole slab be designed as a fully suspended slab supported on the underlying bedrock.

Should it be proposed to adopt a slab on grade, it should be noted that it is difficult to successfully complete earthworks on a small scale such as this. However, where this approach is adopted and where soils are exposed at bulk excavation level, the subgrade should be first stripped of all topsoil and any root affected soils and the subgrade proof rolled in the presence of an experienced geotechnical engineer. All proof rolling should be carried out without vibration. The purpose of proof rolling is to increase the near surface density of the subgrade and to identify any loose or unstable zones. Where unstable zones are identified they should be excavated down to a sound base and replaced with engineered fill. Where such zones be identified, further advice will be provided on site and insitu density testing will be required to confirm that any fill placed has been placed as engineered fill to the required specification. Care should be taken not to use rollers that are excessively large. We suggest a roller of only about 5 tonnes be used. Based on the size of the site and localised excavations that will likely be required (i.e. lift overrun, buried services, etc.), the above subgrade preparation will not be convenient and may lead to the preparation of a substandard subgrade. Consequently, as discussed above, our preferred approach is that all slabs be designed as fully suspended and supported on the underlying bedrock.

Where bedrock is exposed no proof rolling is required, although a coarse granular de-bonding layer should be placed over the surface of the rock. A construction joint should be detailed at the interface between the soil and bedrock subgrade. This same coarse granular layer may be placed over the remainder of the site and rolled in to form a sound working layer, particularly where the exposed subgrade is not particularly trafficable.

6.1.11 According to AS1170.4-2007 'Structural Design Actions – Part 4: Earthquake Actions in Australia', including Amendment No. 1 (August 2015) and Amendment No. 2 (February 2018) the site classifies as Class D_e. This is based on soil with an equivalent SPT N-value of less than 6, but with a soil layer less than 10m thick.

The Hazard Factor, Z, for Sydney is 0.08.

6.1.12 The laboratory test results indicate that for buried concrete structures the soils have a 'Mild' exposure classification in accordance with Table 6.4.2 (C) of AS2159-2009 "Piling Design and Installation''. For buried steel structures the soils have a 'Non-Aggressive' exposure classification in accordance with Table 6.5.2 (C) AS2159-2009.

6.1.13 The guidelines for Hillside Construction given in Appendix B should also be adopted.

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 Dilapidation surveys must be carried out on the neighbouring buildings and structures to the north and south. A copy of the dilapidation report must be provided to the relevant property owners who should be asked to review and sign the reports to confirm that they represent an accurate record of existing conditions. These reports may also be required to be submitted to Council or the Principle Certifying Authority.

6.2.5 An excavation/retention methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to the proposed excavation techniques, excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types and contingency plans in case of exceedances.

6.2.6 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 In the early stages of construction, further investigation of the subsurface conditions above the cliff line and a detailed inspection of the cliff line will be required following de-vegetation. This will need to be complete by, or in conjunction with, the geotechnical engineer. Further advice on the suitability of the proposed removal method of the boulders will also be required at this point to manage the risk posed by their removal.

6.3.3 Where percussive excavation techniques are used, continuous vibration monitoring will be necessary. The prescribed vibration limits set out in the Vibration Emission Design Goals attached to the rear of this report should be adopted. We recommend that a peak particle velocity (PPV) of 5mm/s be adopted as the trigger level.

6.3.4 As discussed above, we recommend that in the early stage of construction, test pits be excavated along the southern and northern boundaries to investigate the ground conditions to confirm where the transition between the soldier and contiguous pile walls will be required.

The geotechnical engineer is to witness the installation of the retaining wall piles to confirm that the design criteria is satisfied.

6.3.5 All rock anchors must be proof-tested to 1.3 times the working load. In addition, the anchors must be subjected to lift-off testing no sooner than 24 hours after locking off at the working load. The proof-testing and lift-off tests must be witnessed by the geotechnical engineer. The anchor contractor must provide the geotechnical engineer with all field records including anchor installation and testing records.

6.3.6 Where the retaining walls have not been designed to support adversely orientated defects, excavation must be completed in a carefully controlled and staged manner. This will allow the geotechnical engineer to inspect the rock mass for the presence of such defects prior to the removal of the rock that provides support to the excavation. Where adverse defects are identified, provision for the installation of additional temporary support in the form of temporary anchors, props etc. must be made. To allow the rock mass to be inspected, slots in the bedrock will need to be excavated perpendicular to the cut lines so that the rock mass can be inspected by a geotechnical engineer prior to the completion of bulk excavation. Further advice on this staging can be provided should this approach be adopted.

6.3.7 The geotechnical engineer must inspect all piles and footing excavations prior to placing reinforcement or pouring the concrete to confirm that the design ABP's have been achieved.

6.3.8 The material proposed to be used for backfilling behind retaining walls must be approved by the geotechnical engineer prior to placement.

6.3.9 Compaction density of any fill placed must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in AS3798. These results should be submitted to the geotechnical engineer for review.

6.3.10 If they are to be retained, the existing stormwater system, sewer and water mains must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer or architect, and repaired if found to be leaking.

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

6.3.12 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).

6.3.13 The geotechnical engineer must confirm that the proposed works have 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.

6.4 Conditions Recommended for Ongoing Management of the Site/Structure(s)

The following recommendations have been included so that the current and future owners of the subject property are aware of their responsibilities:

6.4.1 All existing and proposed surface (including roof) and subsurface drains must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than ten yearly intervals; including provision of a written report confirming scope of work completed (with reference to the 'as-built' drawing) and identifying any required remedial measures.

6.4.2 No cut or fill in excess of 0.5m (eg. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Council.

6.4.3 Where the structural engineer has indicated a design life of less than 100 years then the structure and/or structural elements must be inspected by a structural engineer at the end of their design life; including a written report confirming scope of work completed and identifying the required remedial measures to extend the design life over the remaining 100 year period.

6.4.4 The risk assessment must be updated following the removal/support of the stability hazards.

7 OVERVIEW

It is possible that the subsurface soil, rock or groundwater conditions encountered during construction may be found to be different (or may be interpreted to be different) between investigation locations or from those inferred in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

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.

Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

Reference 2: MacGregor, P, Walker, B, Fell, R, and Leventhal, A (2007) *'Assessment of Landslide Likelihood in the Pittwater Local Government Area'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp183-196.

TABLE A

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	**A** **Stability of the cliff-line – blocks of rock detaching**	**B1** **Existing small to large sized boulders, 0.5m to 1.5m diameter**	**B2** **Existing very large boulder upslope at the north-west corner**	**C** **Stability of the hillside**	**D1a** **Stability of the rough-hewn dry packed sandstone block retaining walls**	**D1b** **Stability of the sandstone block /brick masonry retaining walls**	**D2** **Stability of the concrete masonry retaining wall**	**D3** **Stability of the timber retaining walls**	**E** **Stability of new engineered walls**
Assessed Likelihood	Likely	Possible	Rare	Possible	Almost Certain	Possible	Rare	Possible	Barely Credible
Assessed Consequence	Insignificant	Minor	Catastrophic	Minor	Insignificant	Insignificant	Insignificant	Insignificant	Minor
Risk	Low	Medium	Medium	Medium	Low	Very Low	Very Low	Very Low	Very Low

* The above consequences are based on an assumed property value of $7M (Source: **www.onthehouse.com.au**, 9 September 2021)

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A Stability of the cliff-line – blocks of rock detaching	B1 Existing small to large sized boulders, 0.5m to 1.5m diameter	B2 Existing very large boulder upslope at the north-west corner	C Stability of the hillside	D1a Stability of the rough-hewn dry packed sandstone block retaining walls	D1b Stability of the sandstone block /brick masonry retaining walls	D2 Stability of the concrete masonry retaining wall	D3 Stability of the timber retaining walls	E Stability of new engineered walls
Assessed Likelihood	Likely	Possible	Rare	Possible	Almost Certain	Possible	Rare	Possible	Barely Credible
Indicative Annual Probability	10^{-2}	10^{-3}	10^{-5}	10^{-3}	10^{-1}	10^{-3}	10^{-5}	10^{-3}	10^{-6}
Duration of Use of area Affected (Temporal Probability)	(i) Above cliff line 1 minute/month 2.3×10^{-5} (ii) Below cliff line 1 minute/month 2.3×10^{-5}	Below 8 hours/day 3.3×10^{-1}	Below 8 hours/day 3.3×10^{-1}	(i) On slope 1 minute/month 2.3×10^{-5} (ii) Below slope 1 minute/day 6.9×10^{-4} (Assume shallow rock and does not penetrate house walls, requires further investigation to confirm)	(i) Above wall 1 minute/month 2.3×10^{-5} (ii) Below wall 1 minute/day 6.9×10^{-4}	(i) Above wall 1 minute/month 2.3×10^{-5} (ii) Below wall 1 minute/day 6.9×10^{-4}	(i) Above wall 0.5 minute/day 3.5×10^{-4} (ii) Below wall 1 minute/month 2.3×10^{-5}	(i) Above wall 1 minute/month 2.3×10^{-5} (ii) Below wall 20 seconds/day 2.3×10^{-4}	(i) Above wall 8 hours/day 3.3×10^{-1} (includes time on upper level within new house) (ii) Below wall 8 hours/day 3.3×10^{-1}
Probability of not Evacuating Area Affected	(i) 1.0 (ii) 0.9	0.9	0.9	((i) 0.9 (ii) 0.9	(i) 0.9 (ii) 1.0	(i) 0.9 (ii) 1.0	(i) 0.9 (ii) 1.0	(i) 0.9 (ii) 1.0	(i) 1.0 (ii) 1.0
Spatial Probability	1m length of failure over ˜12m full length, 1/12 = 0.083	1.5m over 19m width, 1/19 = 0.053	75% width, 0.75	4m length, 4/19 = 0.21 (Assume shallow rock as above)	4m length of failure over total ˜15m, 4/15 = 0.27	4m length of failure over total ˜33m, 4/33 = 0.12	4m length of failure over total ˜10m, 4/10 = 0.4	7m length, 50% fails, 0.5	6m length of failure over ˜60m combined lengths, 6/60 = 0.1
Vulnerability to Life if Failure Occurs Whilst Person Present	(i) 0.01 (ride down) (ii) 0.1 (buried/hit)	1.0	1.0	(i) 0.5 (ride down) (ii) 1.0	(i) 0.1 (fall from above) (ii) 0.8 (low height)	(i) 0.1 (fall from above) (ii) 0.8 (low height)	(i) 0.1 (fall from above) (ii) 1.0	(i) 0.1 (fall from above) (ii) 0.8 (low height)	(i) 0.1 (fall from above) (ii) 1.0 (likely to be buried)
Risk for Person most at Risk	(i) 1.91×10^{-10} (ii) 1.72×10^{-9}	1.57×10^{-5}	2.22×10^{-6}	(i) 2.17×10^{-9} (ii) 1.30×10^{-7} (Note 1)	(i) 5.59×10^{-8} (ii) 1.50×10^{-5}	(i) 2.48×10^{-10} (ii) 6.62×10^{-8}	(i) 1.26×10^{-10} (ii) 9.20×10^{-11}	(i) 1.04×10^{-9} (ii) 9.20×10^{-8}	(i) 3.30×10^{-9} (ii) 3.30×10^{-8}
Combined Risk for Person Most at Risk	**3.3×10^{-5} (To achieve an acceptable risk to life for the site the risk level must be reduced to no greater than 1×10^{-6})** **Reference report recommendations for further details to lower risk level** **Note 1: The magnitude of risk posed by Hazard C, the steep hillside present above the proposed development, is dependent on the depth to bedrock. Where bedrock is shallow, the risk posed is likely to be acceptable and may be managed by the installation of low height retaining walls founded on the underlying bedrock that support the shallow soil cover. Where the soil cover is deep the slopes will be over-steep and will require some form of stabilisation. Further investigation will be required to determine the depth to bedrock with potential stabilisation measures comprising retaining walls, soil nails with soft facings or similar.** **Risk assessment to be carried out following stabilisation works**								

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Reform Projects Pty Ltd
Ref No: 34272YJ

Project: Proposed Residential Development
Report: A

Location: 14 Ocean Road, Palm Beach, NSW
Report Date: 4/08/21

Page 1 of 1

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	2.85 - 2.88	0.2	4	A
	3.36 - 3.40	0.1	2	A
	3.89 - 3.93	0.1	2	A
	4.38 - 4.42	0.3	6	A
	5.33 - 5.37	0.2	4	A
	5.60 - 5.64	0.4	8	A
	6.41 - 6.44	0.2	4	A
	6.84 - 6.87	0.1	2	A
	7.10 - 7.13	0.6	12	A
	7.79 - 7.82	1	20	A
	8.08 - 8.11	0.9	18	A
	8.65 - 8.69	0.2	4	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the IS(50) has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 IS(50).

Envirolab Services Pty Ltd
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CERTIFICATE OF ANALYSIS 275759

Client Details

Client	JK Geotechnics
Attention	Ben Sheppard
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34272YJ, Palm Beach**
Number of Samples	3 Soil
Date samples received	12/08/2021
Date completed instructions received	12/08/2021

Analysis Details

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

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

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

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	19/08/2021
Date of Issue	19/08/2021

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Client Reference: 34272YJ, Palm Beach

Misc Inorg - Soil				
Our Reference		275759-1	275759-2	275759-3
Your Reference	UNITS	BH1	BH1	BH1
Depth		0.1-0.2	1.6-1.8	2.8-2.9
Date Sampled		23/07/2021	23/07/2021	23/07/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	17/08/2021	17/08/2021	17/08/2021
Date analysed	-	17/08/2021	17/08/2021	17/08/2021
pH 1:5 soil:water	pH Units	6.1	5.3	6.0
Sulphate, SO4 1:5 soil:water	mg/kg	20	27	20
Chloride, Cl 1:5 soil:water	mg/kg	<10	21	<10
Resistivity in soil*	ohm m	300	210	480

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	275759-2
Date prepared	-			17/08/2021	1	17/08/2021	17/08/2021		17/08/2021	17/08/2021
Date analysed	-			17/08/2021	1	17/08/2021	17/08/2021		17/08/2021	17/08/2021
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	6.1	6.1	0	100	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	20	20	0	98	99
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	88	91
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	300	300	0	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH Samples were out of the recommended holding time for this analysis.

BOREHOLE LOG

Borehole No.
1
1 / 2

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Job No.: 34272YJ **Method:** HAND AUGER **R.L. Surface:** ~10.5 m
Date: 8/3/21 **Datum:** AHD
Plant Type: **Logged/Checked By:** B.S./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				■	REFER TO DCP TEST RESULTS	10		▩		FILL: Sandy clay, low to medium plasticity, grey and brown, fine grained sand, trace of fine to medium grained igneous and sandstone gravel, sandstone cobbles, slag fragments, silt and roots.	w>PL			APPEARS POORLY COMPACTED
				■			1	▨	CI	Silty CLAY: medium plasticity, orange brown and brown, trace of fine to medium grained sand, fine to medium grained sandstone gravel.	w~PL	VSt	210 210 210	RESIDUAL HP READINGS ON LUMPS RECOVERED FROM AUGER
						9		▨				St	110 100	
				■				▨	CI-CH	Silty CLAY: medium to high plasticity, light grey mottled orange brown.		VSt	210 230 220	
				■			2	▨		as above, but light grey mottled orange brown.		St - VSt	150 170 200	HAND AUGER REFUSAL
						8				REFER TO CORED BOREHOLE LOG				
							3							
						7								
							4							
						6								
							5							
						5								
							6							
						4								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34272YJ PALMBEACH.GPJ <<DrawingFile>> 23/09/2021 17:09 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CORED BOREHOLE LOG

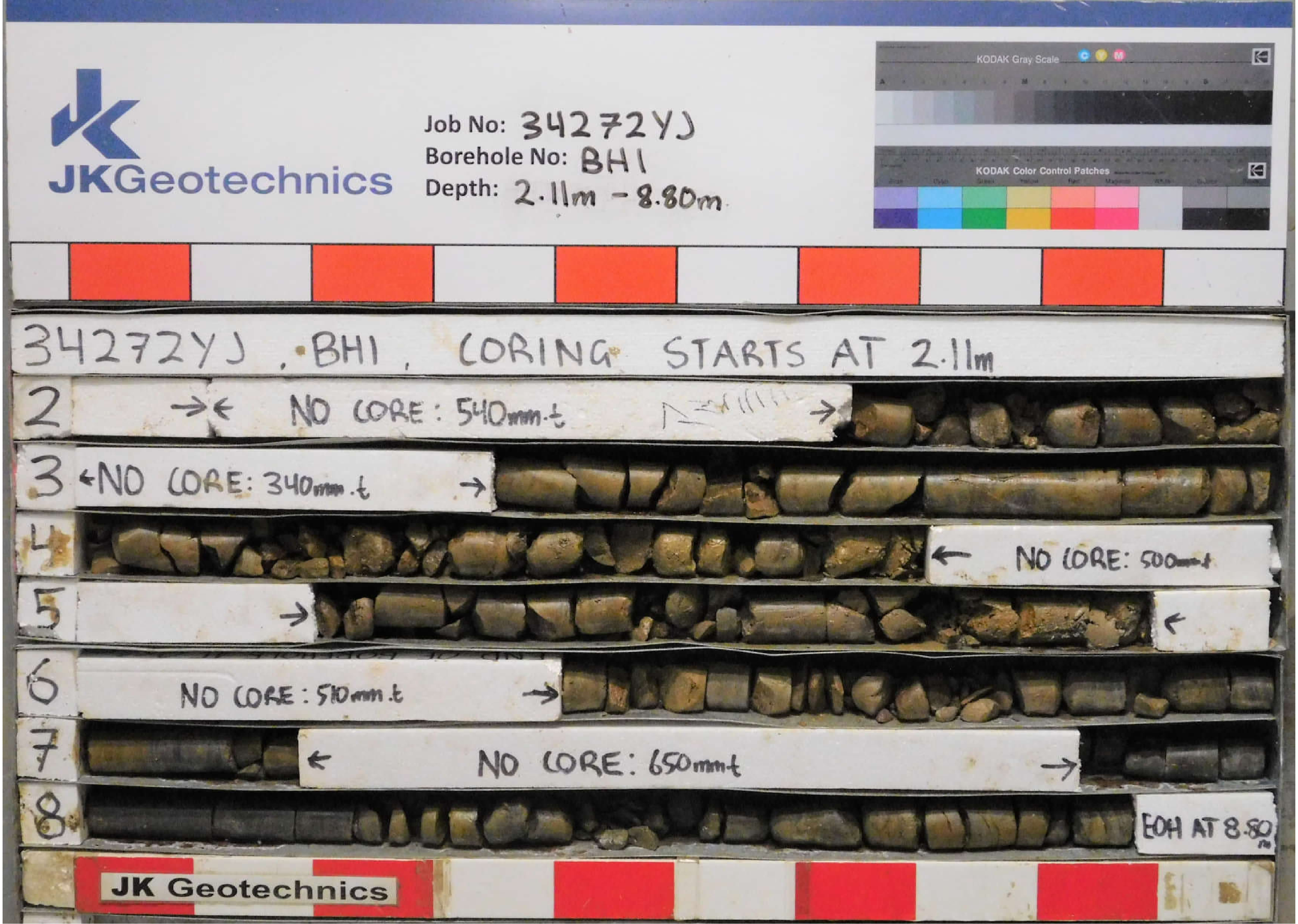
Borehole No.
1

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Job No.: 34272YJ
Core Size: TT56
R.L. Surface: ~10.5 m
Date: 8/3/21
Inclination: VERTICAL
Datum: AHD
Plant Type: MELVELLE
Bearing: N/A
Logged/Checked By: B.S./J.M.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components START CORING AT 2.11m	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS: SPACING (mm) 600 200 60 20	DEFECT DETAILS: DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
25% RETURN		8			NO CORE 0.54m						
			3		Interbedded SILTSTONE(70%) and SANDSTONE(30%): dark grey and brown, and fine grained, grey, distinctly bedded at 0-10° (possible boulder).	HW	L	0.20		(2.65-3.00m) Rock is fractured, numerous beddings, joints and crushed seams as Be, 0°, P, R, Fe, Sn, J,40-90°, P, R, Fe, Sn & Cr, 0°, <10mm.t	
ON COMPLETION OF CORING					NO CORE 0.34m						
		7 6	4		Interbedded SILTSTONE(80%) and SANDSTONE(20%): dark grey and brown, and fine grained, grey, distinctly bedded at 0-5°.	HW	L L - M	0.10 0.10 0.30		(3.41m) J, 45°, P, R, Fe Sn (3.34-3.55m) Numerous J, 10 - 90°, Fe (3.46m) J, 20°, P, R, Fe Sn (3.51m) J, 80°, Un, R, Fe Sn (3.62m) J x 2, 70 - 90°, Un, S, Fe Sn (3.69m) J, 30°, P, R, Fe Sn (3.91m) J, 30°, P, R, Fe Sn (3.95m) J, 30°, P, R, Fe Sn (4.00-4.40m) Rock is fractured, numerous Be, 0-5°, P/Un, R, Fe, Sn, & J, 10-90°, P/Un, S, Fe, Sn (4.39m) CS, 0°, 4 mm.t (4.48m) J, 70°, P, R, Fe Sn (4.49m) Be, 0°, P, R, Fe Sn (4.53m) Be, 0°, P, R, Fe Sn (4.56m) Cr, 0°, 7 mm.t	
			5		NO CORE 0.50m					(4.62m) J x 2, 10 - 90°, Un, R, Clay Cn (4.68m) XWS, 0°, 30 mm.t	
0% RETURN		5			Interbedded SILTSTONE(60%) and SANDSTONE(40%): dark grey and brown, and fine grained, grey and red brown, distinctly bedded at 0-10°.	HW	L - M	0.20 0.40		(5.21-5.26m) Rock is fractured, numerous Be, 0-5°, P/Un, R, Fe, Sn, & J, 20-90°, P, R, Fe, Sn (5.32m) J, 30°, P, R, Fe Sn (5.38m) J, 25°, Un, R, Fe Sn (5.42m) Be, 5°, Un, R, Fe Sn (5.47-5.57m) Rock is fractured, numerous Be, 0°, P, R, Fe, Sn, & J, 70-90°, P, R, Fe, Sn (5.70m) J, 70 - 90°, Ir, R, Fe Sn (5.72m) XWS, 0°, 45 mm.t (5.78m) Be, 5°, Un, R, Fe Sn	Narrabeen Group
			6		NO CORE 0.51m					(5.85m) XWS, 0°, 40 mm.t	
		4	7		Interbedded SILTSTONE(50%) and SANDSTONE(50%): dark grey, and fine grained, dark grey, distinctly bedded at 0-10°, with laminite and iron indurated bands.	HW	L M	0.20 0.10 0.60		(6.40-6.84m) Rock is fractured, numerous Be, 0-10°, P/Un, R, Clay, Sn, & J, 20-90°, P/Un, R, Fe, Sn (6.90m) J, 45°, P, R, Fe Sn (6.98m) J, 90°, P, R, Fe Sn (7.16m) J, 40°, P, R, Fe Sn	
		3			NO CORE 0.65m			1.0			
			8		SILTSTONE: dark grey, distinctly bedded at 0-10°, with iron indurated bands.	SW	M - H	0.90		(7.89m) J, 85°, P, R, Cn (8.25m) Be, 0°, P, R, Clay Vn	
		2			Interbedded SILTSTONE(70%) and SANDSTONE(30%): dark grey, and fine grained, grey, distinctly bedded at 0-10°, with iron indurated bands.	MW	L	0.20		(8.30-8.60m) Rock is fractured, numerous Be, 0-10°, P/Un, R, Fe, Sn/Cn, & J, 30-90°, P, R, Fe, Sn (8.68m) Be, 0°, Un, R, Fe Sn (8.74m) J, 20°, P, R, Fe Sn (8.71-8.85m) Numerous Ji, 25 - 45°, Fe Sn (8.82m) Be x 2, 0 - 5°, P, R, Fe Sn	
					END OF BOREHOLE AT 8.88 m				6 2 6 2		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



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Job No: 34272YJ

Borehole No: BH1

Depth: 2.11m - 8.80m

BOREHOLE LOG

Borehole No.
2
1 / 1

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Job No.: 34272YJ
Method: HAND AUGER
R.L. Surface: ~5.3 m
Date: 8/3/21
Datum: AHD
Plant Type: -
Logged/Checked By: B.S./J.M.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	5		▨		FILL: Sandy clay, low plasticity, dark brown, with organic material, trace of fine grained ironstone gravel and root fibres.	w<PL	VD		MULCH COVER APPEARS POORLY COMPACTED
						4	1	▨		FILL: Clayey sand, fine to medium grained, grey and brown, trace of fine to medium grained igneous gravel and silt.	M			
				■ ■		3	2	░	SP	SAND: medium to coarse grained, orange brown, trace of shale gravel.	M	MD L		MARINE
						2 1 0 -1	3 4 5 6			END OF BOREHOLE AT 3.00 m				

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JKGeotechnics

DYNAMIC CONE PENETRATION TEST RESULTS

Client: REFORM PROJECTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Job No. 34272YJ
Date: 3-8-21
Tested By: B.S.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	**1**	**2**	**3**	Test Location		**2**	**3**
Surface RL	≈10.6m	≈5.3m	≈5.0m	Surface RL		≈5.3m	≈5.0m
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	SUNK	EXCAVATED	3000-3100		5	7/80mm
100 - 200	↓	↓	↓	3100-3200		12/90mm	REFUSAL
200 - 300	3	↓	↓	3200-3300		REFUSAL	
300 - 400	1	1	2	3300-3400			
400 - 500	1	3	7	3400-3500			
500 - 600	1	6	3	3500-3600			
600 - 700	↓	3	3	3600-3700			
700 - 800	2	1	3	3700-3800			
800 - 900	2	1	3	3800-3900			
900 - 1000	2	2	3	3900-4000			
1000 - 1100	4	33	4	4000-4100			
1100 - 1200	4	8	4	4100-4200			
1200 - 1300	2	8	4	4200-4300			
1300 - 1400	3	8	4	4300-4400			
1400 - 1500	3	5	4	4400-4500			
1500 - 1600	3	4	4	4500-4600			
1600 - 1700	3	6	4	4600-4700			
1700 - 1800	3	6	3	4700-4800			
1800 - 1900	4	6	3	4800-4900			
1900 - 2000	4	5	5	4900-5000			
2000 - 2100	7	5	4	5000-5100			
2100 - 2200	6	7	4	5100-5200			
2200 - 2300	9	6	3	5200-5300			
2300 - 2400	17/50mm	7	4	5300-5400			
2400 - 2500	REFUSAL	7	3	5400-5500			
2500 - 2600		7	4	5500-5600			
2600 - 2700		6	4	5600-5700			
2700 - 2800		5	3	5700-5800			
2800 - 2900		3	3	5800-5900			
2900 - 3000		3	2	5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

Ref: JK Geotechnics DCP 0-6m Rev5 Feb19

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:
SITE LOCATION PLAN

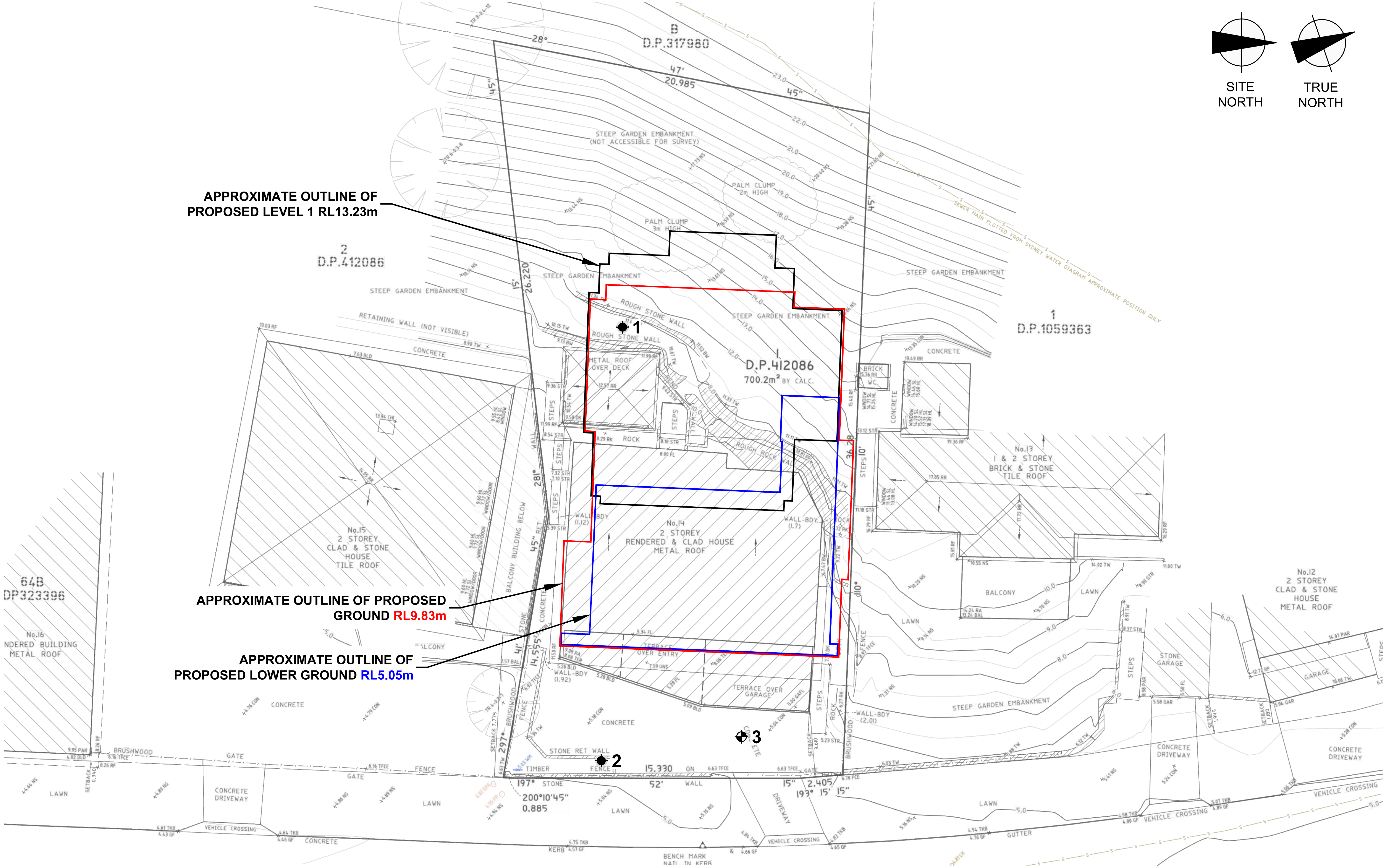
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ

Figure No: 1

JKGeotechnics

PLOT DATE: 11/10/2021 10:30:18 AM DWG FILE: Y:\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG



LEGEND

- ● BOREHOLE AND DCP TEST
- ⊕ DCP TEST

DRAWING REF: ARCHITECTURAL DRAWINGS BY MATHIESON ARCHITECTS PTY LTD, DRAWING NOS. DA.02, REVISION D AND DA.03, REVISION E, DATED 22 SEPTEMBER 2021.

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

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

Title:
INVESTIGATION LOCATION PLAN

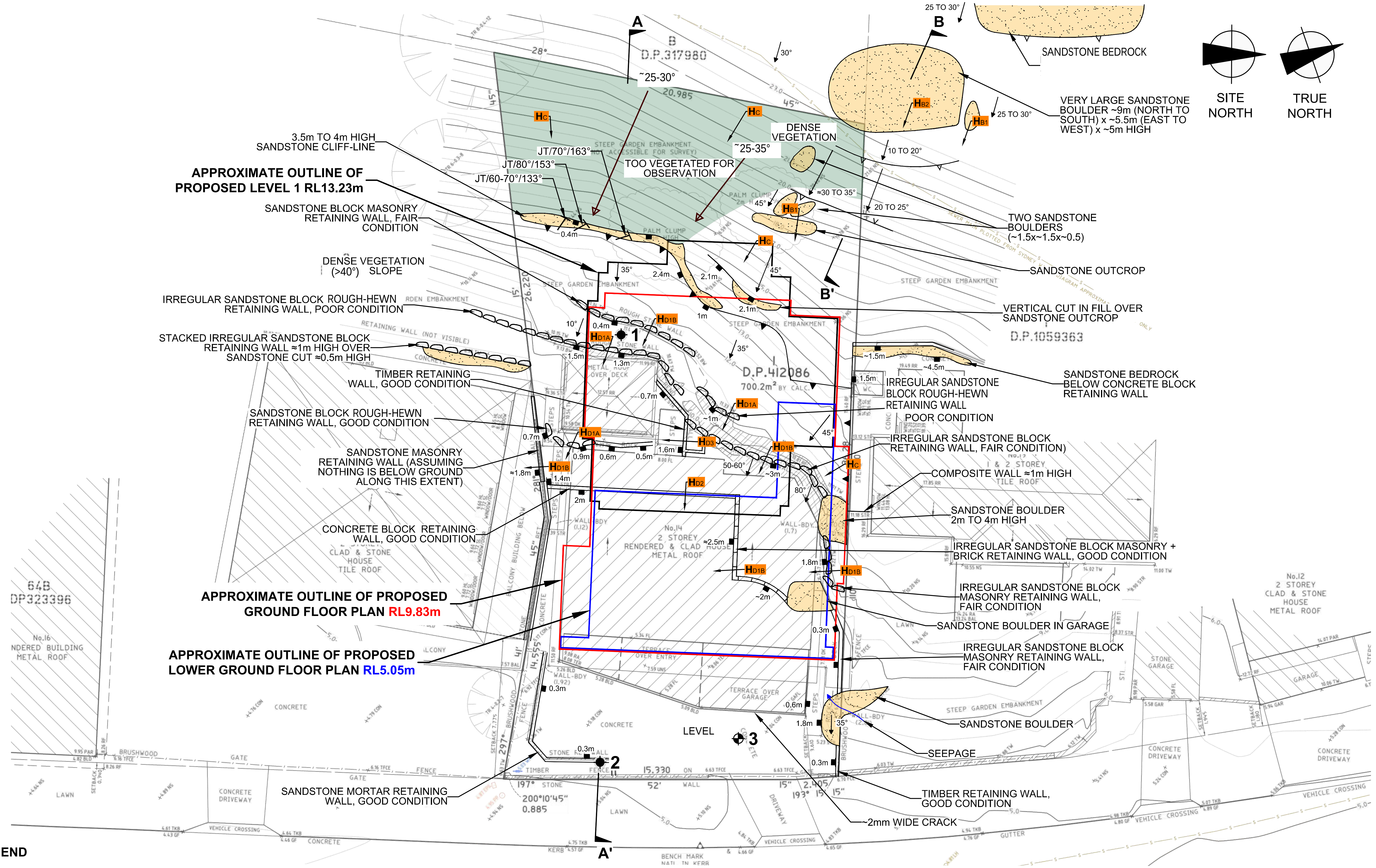
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ

Figure No: 2

JKGeotechnics

PLOT DATE: 11/10/2021 9:55:01 AM DWG FILE: Y:\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG



LEGEND

●	BOREHOLE AND DCP TEST
⊕	DCP TEST
HA →	GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS
A ⌞ A'	CROSS SECTION
⟋ JT	DIP/DIP DIRECTION (TRUE NORTH)

NOTES:

1. REFER TO FIGURE 4 FOR SECTION A-A'.
2. REFER TO FIGURE 5 FOR SECTION B-B'.
3. REFER TO FIGURE 6 FOR GETECHNICAL MAPPING SYMBOLS.

DRAWING REF: ARCHITECTURAL DRAWINGS BY MATHIESON ARCHITECTS PTY LTD, DRAWING NOS. DA.02, REVISION D AND DA.03, REVISION E, DATED 22 SEPTEMBER 2021.

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

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

Title: **PLAN OF NOTABLE GEOTECHNICAL FEATURES + HAZARDS**

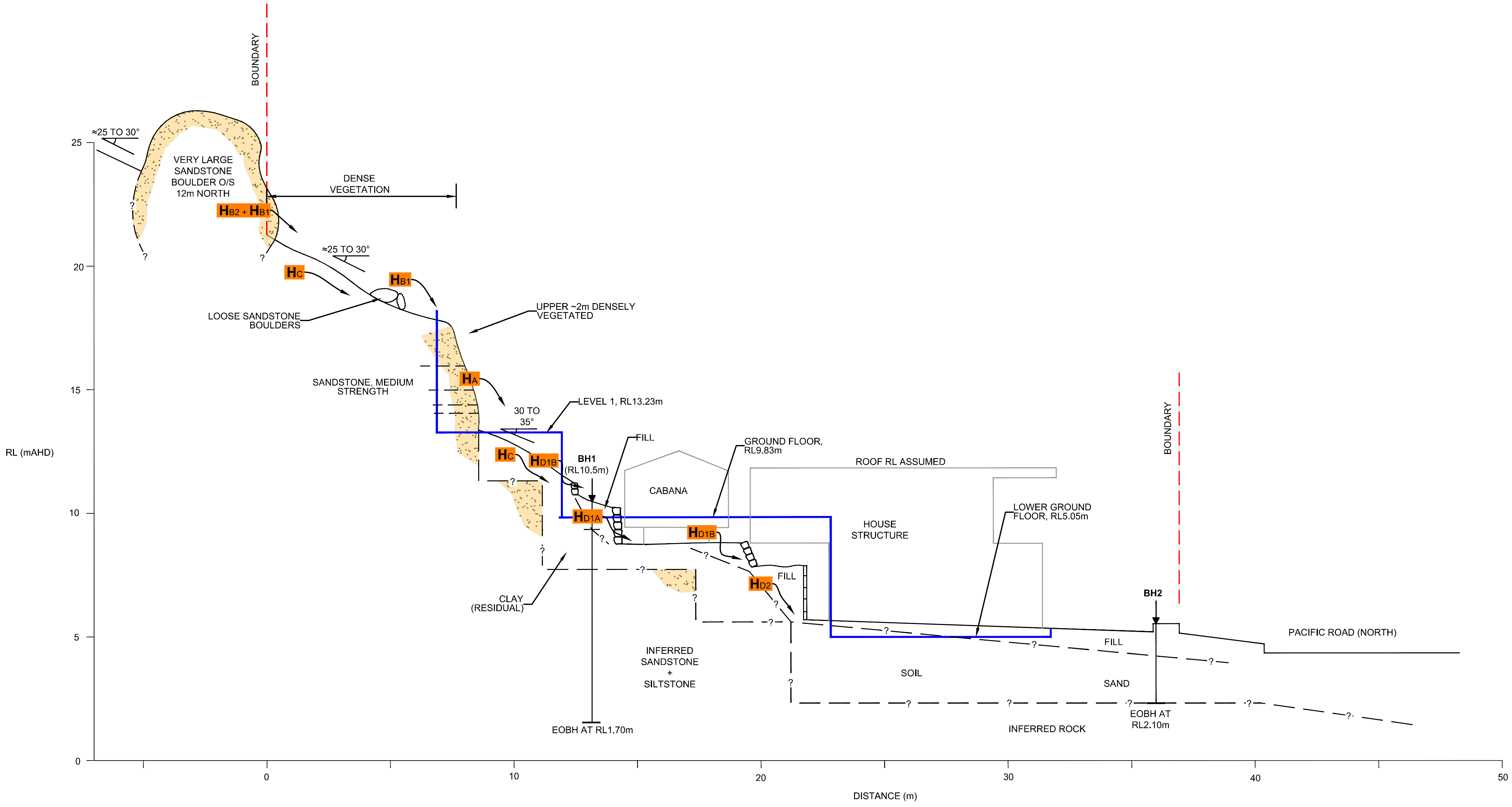
Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ

Figure No: 3

JKGeotechnics

PLOT DATE: 11/10/2021 9:55:13 AM DWG FILE: Y:\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG



LEGEND

H_A → GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS

—?— — INFERRED GEOTECHNICAL UNIT

DRAWING REF: ARCHITECTURAL DRAWINGS BY MATHIESON ARCHITECTS PTY LTD, DRAWING NOS. DA.02, REVISION D AND DA.03, REVISION E, DATED 22 SEPTEMBER 2021.

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

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

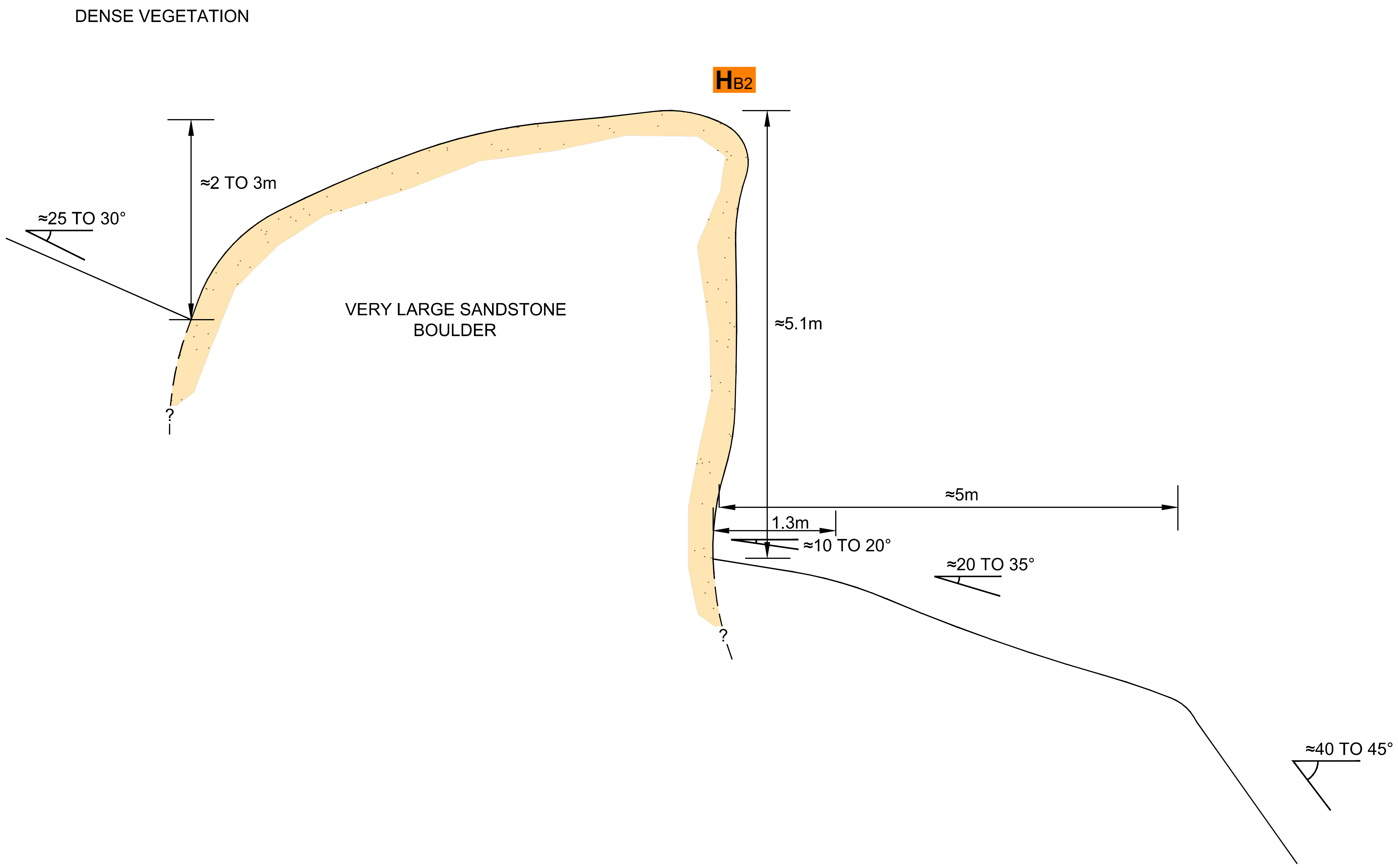
Title:
SECTION A-A'

Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ
Figure No: 4

JKGeotechnics

PLOT DATE: 11/10/2021 9:55:30 AM DWG FILE: Y:\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG



LEGEND

HA→ GEOTECHNICAL HAZARD + DIRECTION OF MOVEMENT IF FAILURE OCCURS

—?— — INFERRED GEOTECHNICAL UNIT

DRAWING REF: ARCHITECTURAL DRAWINGS BY MATHIESON ARCHITECTS PTY LTD, DRAWING NOS. DA.02, REVISION D AND DA.03, REVISION E, DATED 22 SEPTEMBER 2021.

0 0.5 1 1.5 2 2.5

SCALE 1:50 @A3 METRES

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

Title:
SECTION B-B'

Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ

Figure No: 5

JKGeotechnics

PLOT DATE: 27/09/2021 1:55:03 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG

TOPOGRAPHY

Symbol *Ground Profile*

convex
concave
} well defined or angular break of slope

convex
concave
} poorly defined or smooth change of slope

breaks of slope
changes of slope
} convex and concave too close together to allow the use of separate symbols

sharp
rounded
} ridge crest

3 Cliff or escarpment or sharp break 40° or more (estimated height in metres)

15 Uniform Slope
10 Concave Slope
8 Convex Slope
} Slope direction and angle (Degrees)

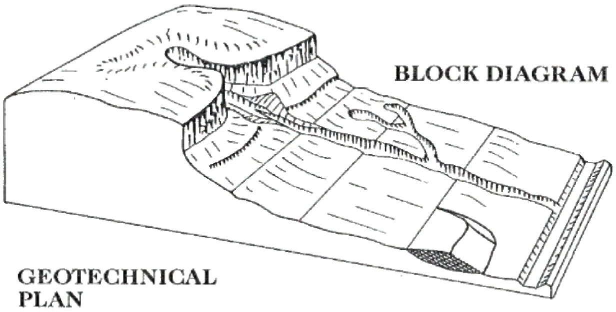
Top
Bottom
} Cut or fill slope, arrows pointing down slope

Hummocky or irregular ground

OTHER FEATURES

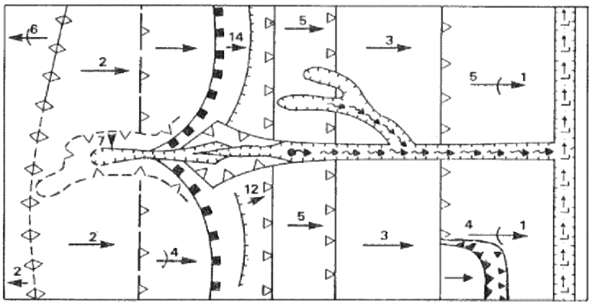
- Boulder
- Seepage/spring
- Swallow hole for runoff
- Natural water course
- Open drain, unlined
- Open drain, lined
- Fenceline
- Property boundary
- Dry Stone Wall
- J 200 J Major joint in rock face (opening in millimetres)
- – T – T – 10 Tension crack (opening in millimetres)
- Masonry or concrete wall
- Ponding water
- Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



BLOCK DIAGRAM

GEOTECHNICAL PLAN



(After Gardiner, V & Dackombe, R. V. (1983), Geomorphological Field Manual; George Allen & Unwin).

PLOT DATE: 27/09/2021 1:55:55 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34272YJ PALM BEACH\CAD\34272YJ.DWG

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

Title:
GEOTECHNICAL MAPPING SYMBOLS

Location: 14 OCEAN ROAD, PALM BEACH, NSW

Report No: 34272YJ

Figure No: 6

JKGeotechnics

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_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

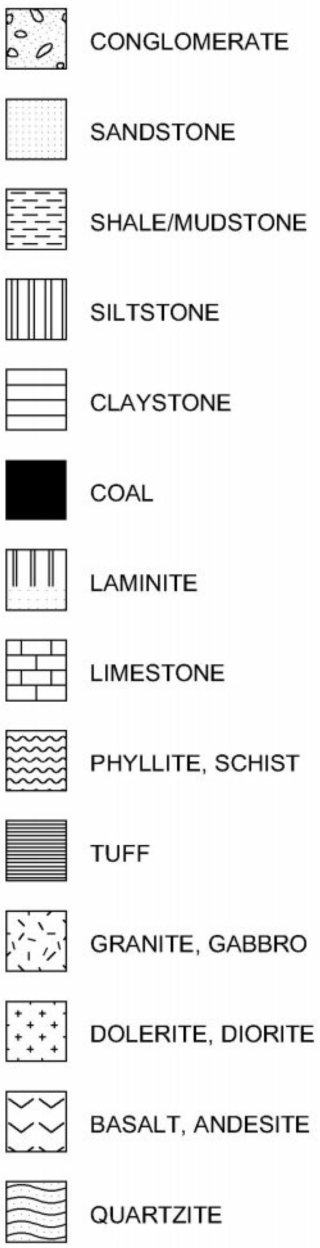
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

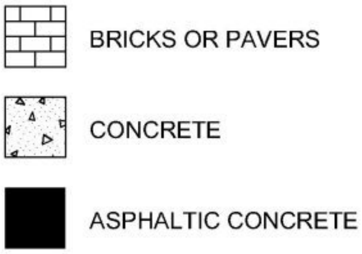
SYMBOL LEGENDS

SOIL

ROCK



OTHER MATERIALS



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:

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

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

NOTES:

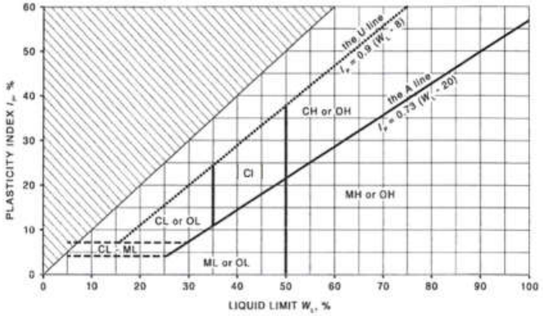
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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

APPENDIX A

LANDSLIDE RISK MANAGEMENT TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

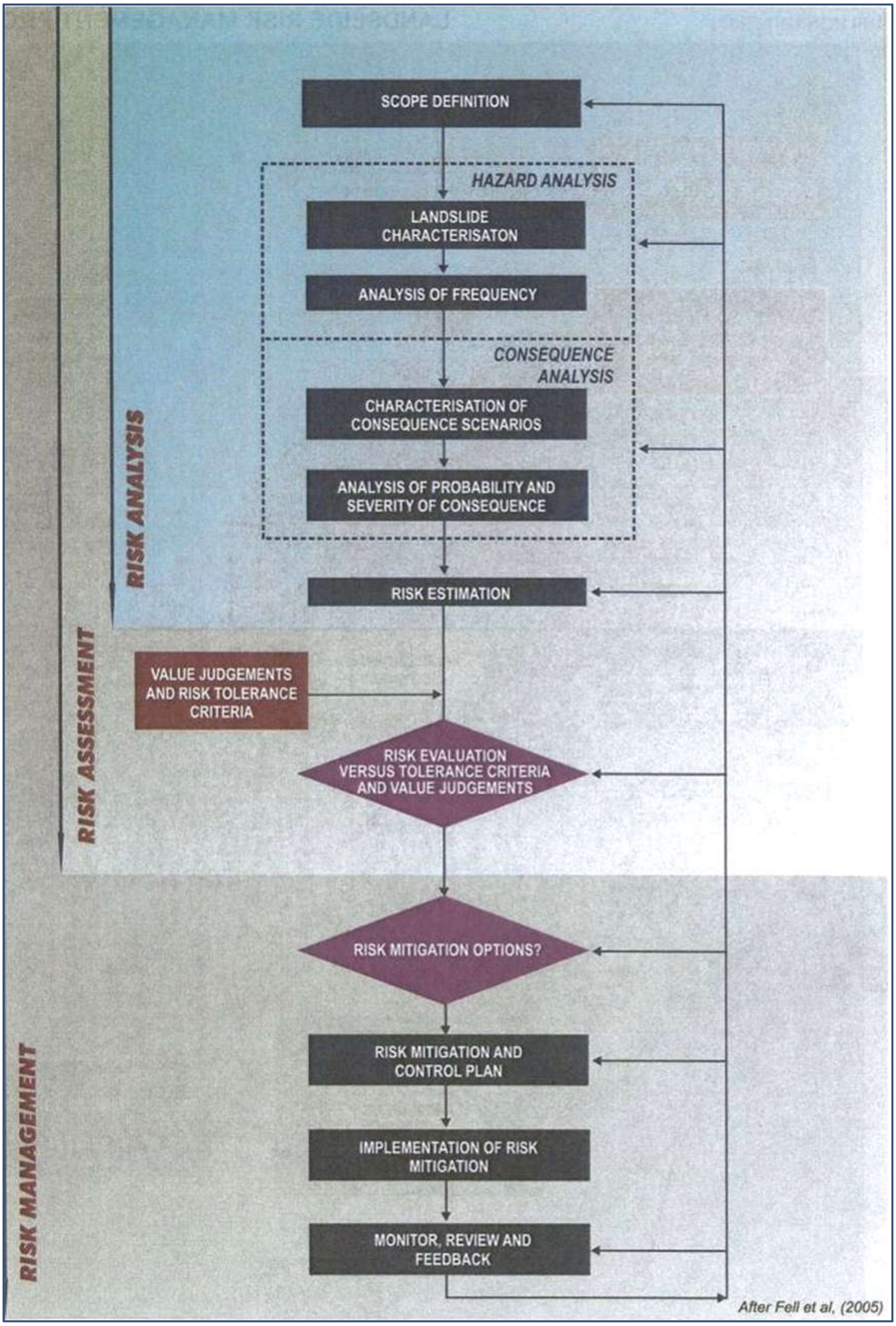
Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated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

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

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

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

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



After Fell et al, (2005)

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.

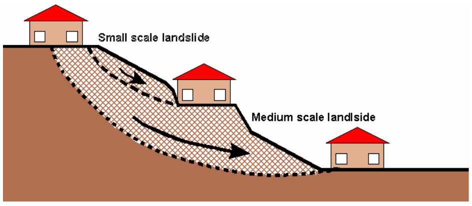


Figure 1

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.

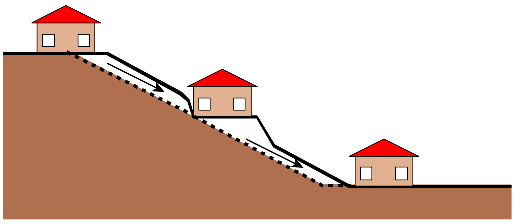


Figure 2

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.

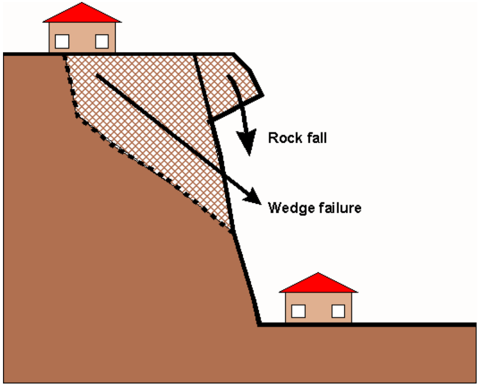


Figure 3

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.

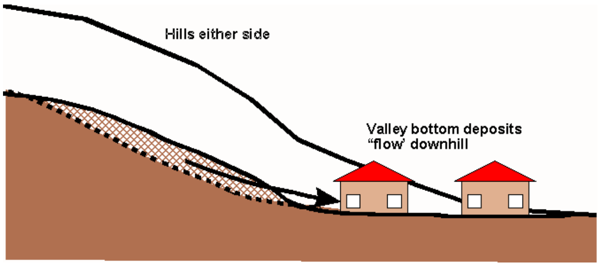


Figure 4

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

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- GeoGuide LR4 - Rock Slopes
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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

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APPENDIX B

SOME GUIDELINES
FOR
HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

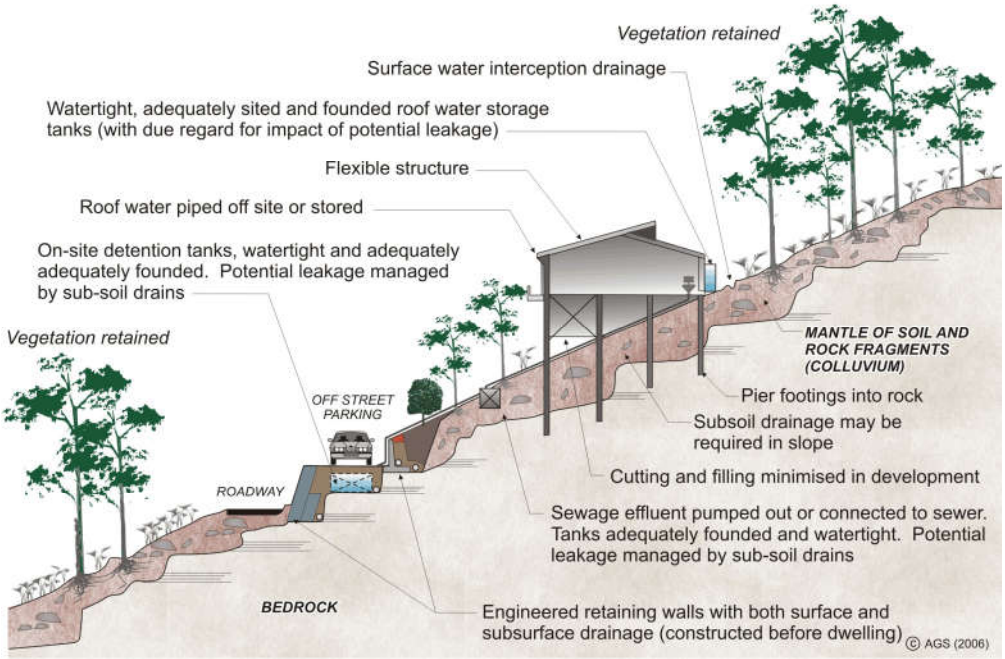
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminant bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements.
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance (including onto properties below). Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc. in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on bedrock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within bedrock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide generous falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge of roof run-off into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use of absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by a geotechnical consultant.	
SITE VISITS	Site visits by consultant may be appropriate during construction.	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is extracted 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 LR8 (CONSTRUCTION PRACTICE)

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES FOR GOOD HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that due to level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfill the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

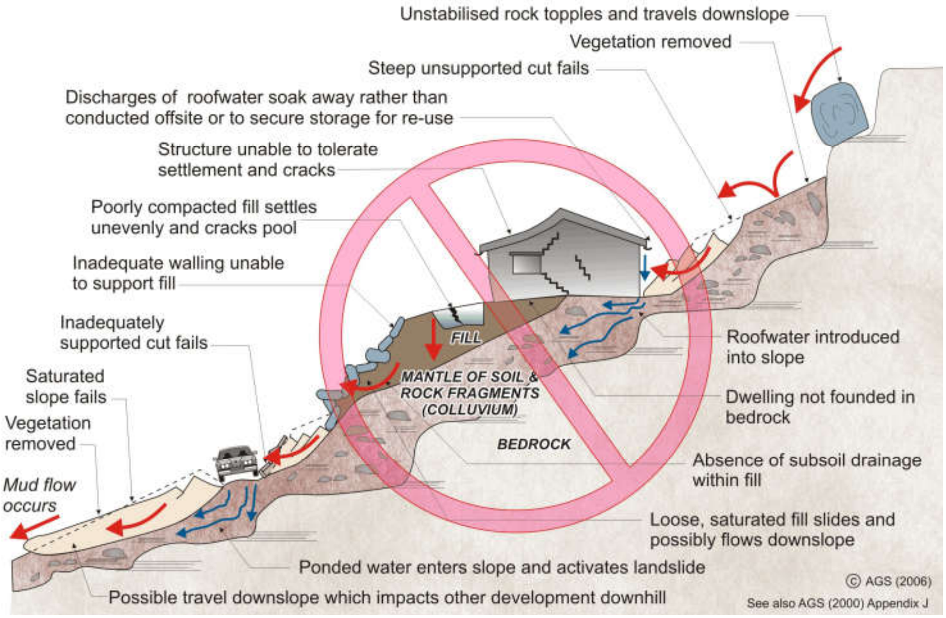
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR POOR HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soaks into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herringbone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

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