REPORT TO

SARAH P JOYCE

ON

GEOTECHNICAL ASSESMENT

FOR

PROPOSED NEW HOUSE

AT

16 ADDISON ROAD, MANLY, NSW

Date: 16 April 2021

Ref: 30147SYrpt

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1 INTRODUCTION

This report has been prepared to update our previous reports prepared on the site at 16 Addison Road, Manly to reflect the current proposed works. The updated report was commission by Mrs Sarah P Joyce and has been prepared in accordance with our proposal P51666A, dated 21 April 2020. The location of the site is shown in Figure 1.

Based on the architectural drawings prepared by Patterson Associates Ltd (Job No: 18010, Sheet No: 1.2 to 1.8, 2.1, 2.2 and 3.1 to 3.3, Revision A, dated 21 December 2020) and the landscape drawings prepared by Dangar, Barin, Smith (Drawing No's: LP01-D4819, LP02-D4819 and LP03-D4819, Revision E, Dated 28 January 2021) we understand that the proposed development comprises the following:

- Demolition of the existing house and construction of a new three-storey house, boathouse and carport.
- Both the house and boathouse will be cut into the hillside and will result in excavation to maximum depths of about 3m. The boathouse will not be located on the water but will be located roughly midway between the proposed house and foreshore at the rear of a relatively flat lawn area located at the crest of the cliff line that drops down to the Harbour. The boathouse will have a finished floor level of RL4.95m.
- To the rear of the house the rear garden will have a finished level of about RL10m and will extend to the face of the boathouse. To achieve these levels cuts to a maximum depth of about 1.5m will be required along the south-western side of the site while filling to a maximum depth of about 1.75m will be required along the north-eastern side of the site.
- Only minor excavation (less than 0.5m) is necessary over the front portion of the site for the carport.

The purpose of this updated report is to use the subsurface investigation obtained during our previous investigation to provide comments and recommendations on site stability, excavation, retention, footings and slabs on grade.

2 INVESTIGATION PROCEDURE

We previously completed a limited subsurface investigation of the site on 7 February 2017 and 15 August 2019. The fieldwork comprised the following:

- Three hand augered boreholes (BH3, BH4, BH5) were initially drilled to refusal depths of about 0.3m and 0.9m below existing surface levels.
- Initially Dynamic Cone Penetration (DCP) tests (DCP1 to DCP6) were carried out adjacent to the boreholes and at three additional locations across the site. The DCP tests were extended to refusal depths ranging between 0.3m and 0.6m below existing surface levels.
- Subsequently, one hand augered borehole (BH108) and 10 DCP's (DCP 101 to 110) were completed to depths ranging from 0.59m and 1.77m.

The test locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features as shown on the survey drawing prepared by Linker Surveying (Ref: 160320, Issue: 1, Dated 15 April 2016). The surface RLs shown on the attached borehole logs and DCP test results were interpolated between spot levels indicated on the provided survey plan. The survey plan forms the basis of Figure 2 and the survey datum is the Australian Height Datum (AHD). Figure 3 presents a section through the site.

The nature and composition of the subsoils were assessed by logging the materials recovered during drilling. The relative compaction and density of the subsoils were assessed by interpretation of the DCP test results. We note that refusal of the DCP equipment often indicates the depth to the underlying bedrock, however, due to the equipment's limitations, it may also refuse on floaters, obstructions within fill, tree roots, ironstone gravel bands, other 'hard' layers within the soil profile, and not necessarily on bedrock.

Groundwater observations were made in the boreholes during, and on completion of, hand auger drilling. No longer term groundwater monitoring has been carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

The fieldwork for the investigation was carried out under the direction of our geotechnical engineers who were present full-time on site and set out the test locations, logged the encountered subsurface profile and nominated in-situ testing and sampling. The borehole logs and DCP test results are presented with this report, together with a glossary of logging terms and symbols used.

Laboratory geotechnical testing was not carried out, as it was not deemed necessary. A contamination screen of site soils and groundwater was outside the agreed scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on the eastern flank of Smedley's Point peninsula, which extends southward into North Harbour, and drops down to Little Manly Cove. The site is a battle-axe property with the main body being roughly rectangular in plan measuring approximately 42m long (east-west) by 11m wide (north-south) and with a 31m long concrete driveway ('handle') which extends east off Addison Road.

At the time of the investigation this foreshore property was occupied by a single storey brick house over a subfloor space at its south-eastern end. The building was supported by steel and brick piers and brick walls. Sandstone bedrock was exposed at the base of brick footings within the subfloor space and was generally assessed to be distinctly weathered and at least very low strength. Beyond the house, the rear (south-eastern end) of the site was terraced and stepped and sloped down to the sandstone cliffline exposed at the foreshore over what appears to be a series of buried or partially buried clifflines.

Modification of site levels has been undertaken through a series of retaining walls to form level terraced areas over part of the site. Immediately behind the house running across the north-eastern side of the site is a timber retaining wall that has a height of about 1.7m and is in a state of failure, showing distress in the form of outward rotation. Roughly halfway down the rear yard two brick retaining walls have been constructed that are founded on sandstone bedrock and have a maximum height of about 3.0m, although as the walls are founded on shelves of sandstone bedrock the total change in elevation is up to about 3.6m. These walls appeared to be in good condition. While these walls are both located roughly midway down the rear yard, due to the fall across the site the terraced area on the south-western side is approximately 2m higher than that on the north-eastern side.

Below the two brick walls on the south-western side of the site the ground slopes down to the east at about 20o to the sandstone cliffline that runs along the south-eastern site boundary, while over the north-eastern side of the site a level grassed area is present that similarly extends to the sandstone cliffline. A low height (up to about 0.8m) sandstone block retaining wall runs along the crest of the cliff line and generally appears in good condition. A large sandstone floater is present in this portion of the site.

The sandstone cliffline along the south-eastern site boundary drops down to Little Manly Cove and has a total height of about 4.2m. The bedrock was assessed to be of at least low to medium strength and was undercut in places, with some sections of the cliffline underpinned. Subvertical jointing was noted in the cliffline which had a strike of 265o to 275o and dipped between 75o and 90o.

The properties to both the north-east and south-west had similar landforms to that of the site, being relatively over the front or north-western portion of the properties before dropping down steeply to Little Manly Cove.

To the south-west is a three-storey masonry building that appeared in good condition and while the house is setback about 1m from the boundary a masonry wall runs along the boundary adjacent to the house and provides a covered entrance for the house. To the rear of the house the property drops down to the foreshore through a series of clifflines. Jointing was noted in the upper of these clifflines that had a strike of about 190o and a dip of about 70o. Levels across the boundary were similar to those on site.

To the north-east is a one to two storey brick house that appeared structurally in good condition. Running across the rear yard of this property, just to the rear of the house, is a sandstone cliffline that has a height of about 4.4m and drops down to a level rear yard. Along the north-eastern site boundary levels are similar to that of the site over the northern portion of site. However, to the south of the cliffline levels drop down to the adjoining property by between about 1.1m and 3.3m. The greatest drop is closest to the cliffline and is retained by a blockwork retaining wall that appeared in good condition. Further to the south, to the rear of the lowest terraced area and running to the foreshore, the difference in height is 1.1m between the two properties and this is retained by a blockwork retaining wall. This wall appears in poor condition, showing distress in the form of outward rotation and cracking.

To the north-west is a two-storey weatherboard house set back greater than 10m from the site boundary. Surface levels across this boundary were similar to those of the subject site.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site is underlain by Hawkesbury Sandstone. The presence of sandstone was confirmed by the visible outcrops. The boreholes disclosed a generalised subsurface profile comprising fill over sandstone bedrock which was inferred at shallow depths. Reference should be made to the attached borehole logs and DCP test results for specific details at each location. A summary of the pertinent subsurface characteristics is presented below:

Pavement

A concrete pavement 80mm thick with no observed reinforcement was encountered at the surface of BH4.

Fill

Silty sand and silty clay fill was encountered beneath the pavement or from the surface in each borehole and extended to the borehole termination depths, which ranged between 0.3m (BH4 and BH5) and 0.4m (BH3). We note that BH4 refused on an obstruction within the fill profile whilst BH3, BH5 and BH108 refused on inferred bedrock. Based on the DCP test results the fill was assessed to be poorly compacted. Inclusions in the fill comprised sandstone gravel and brick fragments.

Sandstone Bedrock

The following DCP refusal depths have been interpreted to indicate the top surface of weathered sandstone bedrock:

- Between 0.5m (RL 10.7m) and 0.6m (RL 10.8m) at the south-eastern end of the existing building footprint.
- Between 0.3m (RL 11.3m) and 0.4m (RL 11.6m) over the central portion of the existing building footprint.
- Between 0.3m (RL 13.5m) and 0.4m (RL 13.4m) at the western end of the existing building footprint.
- Between RL4.03m and RL7.3m over the rear steeply sloping portion of the site.

Sandstone bedrock exposed at the base of brick footings within the sub floor space was assessed to be distinctly weathered and very low strength. Sandstone rock outcrops at the south-eastern end of the site were assessed to be distinctly to slightly weathered and of at least low to medium strength.

Groundwater

Groundwater was not encountered and the boreholes were 'dry' during and a short period following completion drilling the boreholes. No water seepage was observed over the sandstone outcrops. Long term groundwater monitoring was not carried out.

4 Geotechnical Assessment

4.1 Potential Landslide Hazards

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

A Stability of the natural hillside slope below residence

B Stability of timber crib retaining wall:
  (i) Above; and
  (ii) Below

C Stability of brick retaining walls:
  (i) above; and
  (ii) below.

D Stability of steep slope in rear yard:
  (i) on; and
  (ii) below.

E Low height sandstone block retaining wall:
  (i) above; and
  (ii) below.

F Stability of sandstone cliffline:
  (i) above; and
  (ii) below.

G Retaining wall along north-eastern site boundary – northern end:
  (i) above; and
  (ii) below.

H Retaining wall along north-eastern site boundary – southern end:
  (i) above; and
  (ii) below.

I Stability of new retaining walls:
  (i) above; and
  (ii) below.

These potential hazards are indicated in schematic form on the attached Figures 2 and 3.

4.2 Risk Analysis

The attached Table A 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 A indicates that the assessed risk to property is Very Low, which would be considered 'acceptable' in accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 10^{-7}. This would be considered to be 'acceptable' in relation to the criteria given in Reference 1.

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.

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

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

Until the site is developed we recommend that monitoring of the identified hazards should be carried out annually and after periods of heavy or prolonged rainfall, by the property owners. Should signs of instability be observed (eg. development of new cracks, opening up of existing cracks, bowing/leaning of retaining walls, cracks in the ground behind the crest of the retaining walls site, etc), we should be immediately contacted for further advice.

5 Comments and Recommendations

5.1 Excavation and Groundwater

Excavation recommendations provided below should be completed by reference to the WorkCover Australia Code of Practice 'Excavation Work', dated July 2015.

The proposed bulk excavation to depths of up to approximately 3.5m will encounter the shallow soil profile and extend into the underlying bedrock of variable strength. It is anticipated that excavation will be mainly through sandstone bedrock.

Where sandstone bedrock is of low strength or less, we anticipate that excavation should be able to be completed using medium sized excavators (say 15 to 20 tonnes) with buckets with "tiger teeth" attached. Where the sandstone bedrock is of greater than low 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 adjoining houses, retaining walls and boundary walls.

Due to the close proximity of the proposed excavation to the adjoining properties and the risk of damage due to transmitted vibrations, we recommend that all excavation be completed using non-percussive excavation techniques. This is likely to comprise a mix of saw cutting. ripping tynes and rotary grinders.

Although not generally recommended for the site, due to the set back of the proposed boatshed excavation from adjoining structures it is possible that percussive excavation techniques could be used in this part of the site provided appropriate mitigation measures are adopted. Consequently, where percussive rock excavation techniques are adopted, we recommend that considerable caution be taken as there will be direct transmission of ground vibrations to adjoining buildings and structures. Prior to rock excavation commencing, dilapidation reports should be completed on the houses to the north-east, south-west and north-west. A copy of these reports should be provided to the respective property owners and they should be asked to confirm, in writing, that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the works. In this manner the reports protect the builder from unfounded claims relating to damage existing prior to the commencement of work.

Excavation procedures and the dilapidation reports should be carefully reviewed prior to excavation commencing, so that appropriate equipment is used. To help manage the risks associated with percussive excavation we recommend that the following measures be taken:

- During percussive excavation continuous quantitative vibration monitoring must be completed. This will provide feedback to the excavation contractor on the suitability of the excavation equipment adopted. Vibration monitors should ideally be attached to the adjoining structures closest to the location of the percussive excavation. Where non-percussive excavation techniques are adopted no vibration monitoring is required,
- Percussive excavation should be completed so that the excavation is progressively enlarged by breaking small wedges out of the face,
- Rock hammers should only be operated in short bursts to prevent amplification of vibrations.
- Where transmitted vibrations exceed prescribed limits, excavation techniques must be altered to reduce transmitted vibrations to within acceptable limits. This may mean that the size of percussive equipment used may need to be reduced, or non-percussive techniques adopted. Whether reducing the size of the percussive equipment is effective in controlling transmitted vibrations must be confirmed by further quantitative vibration monitoring.

Alternatively, non-percussive excavation techniques may comprise the use of rock saws, ripping tynes, rotary grinders etc. Should ripping tynes be used it is anticipated that closely spaced saw cuts will be required to aid in the ripping process. Care must also be taken to ensure that the tyne is not hammered into the rock in an attempt to break or dislodge the bedrock as this action will result in the creation of possibly damaging transmitted vibrations. Where non-percussive excavation techniques are adopted we consider that vibration monitoring will not be necessary.

The prescribed vibration limits that should be adopted on this site where percussive excavation techniques are adopted are set out in the Vibration Emission Design Goals attached to the rear of this report. We recommend that subject to the results of the dilapidation reports, the PPV along the site boundaries do not exceed 5mm/sec. We note that this vibration limit will reduce the risk of vibration damage to the neighbouring building and structures. However, these vibrations may still be perceptible to occupants of the neighbouring buildings.

Although not encountered during the investigation, groundwater inflow may possibly occur within the sandy soil profile close to, or at, the contact with the underlying bedrock and through defects such as bedding partings, joints etc. within the sandstone bedrock, particularly during and after periods of heavy rain. We expect that groundwater inflows, where encountered, to be localised and typically of relatively small volume, and can be managed by conventional sump and pump or gravity drainage techniques.

Inspection and monitoring of groundwater seepage during bulk excavations is recommended, so that any unexpected conditions can be addressed in a timely manner. We further recommend that a toe drain be formed at the base of all cut rock faces to collect groundwater and direct it to a sump for disposal.

5.2 Retention

Based on the depth to bedrock encountered in the boreholes and the proposed set back of the excavation from the site boundaries, it appears that sufficient space exists for the formation of temporary batters around most of the site. Where space permits, temporary batters through granular soils may be formed no steeper than 1Vertical(V):1.75Horizontal(H). Where batters are adopted, all surcharge loads such as stockpiles, traffic loads etc must be kept well clear of the crest of the batters (ie below a line drawn upwards from the toe of the batter at no steeper than 1V:2H).

Sandstone bedrock of low strength (or greater) may be cut vertically and left unsupported provided it contains no adverse defects such as joints, clay seams bedding partings etc. Where adversely orientated defects are present, remedial measures such as rock bolts or shotcrete may be required to provide support to the excavated bedrock. To this end, we recommend that a geotechnical engineer inspect the excavation during and immediately upon its completion so that adverse defects present may be identified and remedial measures adopted whilst access is optimal. Most weathered sandstone will fret when exposed and even though the cut faces will be essentially stable, protection with shotcrete could be considered a minimum to avoid drains becoming blocked, necessitating regular maintenance. Alternatively, full height retaining walls may be used.

While we anticipate that adequate space exists for the formation of temporary batters it is possible that this may not be the case and in this instance some form of support will be required to be installed. It is best to determine whether such support is required in the very early stages of construction as at this stage it is generally relatively easy and cheap to install such support. Consequently, in the initial stages of excavation we recommend that a number of test pits be dug along the proposed cut lines to confirm the depth to bedrock and whether temporary batters will be able to be accommodated. These test pits would also be used provide some guidance on how well the soils stand vertically prior to the commencement of collapse. This would help determine the best approach to construction of a shoring system should it be required.

Where temporary batters are unable to be accommodated permission will either need to be sought from the adjoining neighbour to extend batters a short distance into their property or, alternatively, some form of retention will need to be installed prior to the commencement of construction. Where the second approach is adopted we anticipate that, due to the limited depth to sandstone bedrock, a gravity wall could be adopted. This wall could be constructed in a trench excavated down to the top of the sandstone bedrock that is then filled with concrete and any necessary reinforcement. Dowels may be required and would be installed into the underlying sandstone bedrock to resist sliding and overturning, although this may be unnecessary depending on the width of the wall. Depending on how well the soils stand up on excavation the wall may need to be incrementally constructed in short sections of say 1.5m to 3m. Where dowels are proposed for walls constructed at the crest of cuts/clifflines, further advice must be sought from this office on the design of these dowels.

For the design of cantilevered retaining walls a triangular earth pressure distribution and a coefficient of active earth pressure (k_a) of 0.35 should be adopted. Should movement sensitive structures be present within twice the proposed retained height or if walls are propped by the ground floor slab, a coefficient of earth pressure (k) of 0.55 rather than the above k_a value should be used; if this situation does occur anywhere further advice should be sought so that the structures are not put at risk during construction. All applicable surcharge loads such as stockpiles, traffic loadings etc should be added to the above pressures together with appropriate hydrostatic pressures. A unit weight of $20kN/m^3$ should be adopted for the retained material. Where the wall is proposed to be founded at the crest of a vertical cut through the sandstone bedrock and dowels are proposed to resist sliding and/or overturning further advice should be sought from this office.

Lateral toe restraint of retaining walls may be achieved by embedding the walls into the bedrock below adjacent excavation and bedrock surface levels. An allowable lateral resistance of 200kPa can be adopted for such design for sandstone bedrock of medium or greater strength. Care is required not to over-excavate in front of such walls, and all excavations in front of the walls, such as for footings etc. must be taken into account in the wall design. Alternatively, for gravity walls with no passive embedment, an allowable interface friction angle of 35° can be adopted for walls founded directly on sandstone bedrock, assuming the sandstone bedrock surface is roughened, and masonry blocks embedded into high strength mortar.

Where backfill has to be placed behind the retaining walls it is relatively difficult to complete compaction to a high standard in small areas with limited access. Consequently, the use of a single sized durable gravel, such as “blue metal” or crushed concrete gravel (free of fines), which do not require significant compactive effort could be considered if good performance is a priority. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in 200mm thick loose layers. If a single size gravel backfill material is adopted, then a geofabric separation layer must be provided between the general fill/natural soil profile and the single size gravel to protect against the migration of fines into the gravel profile. Further, a 200mm thick clayey layer must be provided at the top of the gravel profile to protect against surface water ingress.

At the rear of the site along the north-eastern site boundary are two retaining walls that appear to be located within the adjoining property but support the site. While one of these walls appears in good condition the most southern is in a state of failure and is cracked and rotating outwards. Consequently, we recommend that the structural engineer check the capacity of these walls to resist the loads applied. Where there is uncertainty regarding the suitability of these walls or they are unable to resist the applied loads we recommend that the new walls be designed and constructed on site to provide the required support.

5.3 Footings

Based on the investigation results, sandstone bedrock will be exposed at, or be present just below, bulk excavation level. Consequently, we recommend that the entire building be uniformly supported on sandstone bedrock. Pad and strip footings founded in sandstone bedrock may be designed for an allowable bearing pressure of 1,000kPa. Where footings are located above a line drawn upwards at 45^o from the toe of adjoining cuts/clifflines, the stability of the bedrock must also be taken into account, and additional support, or deepening of selected footings may be required. Additional advice would be provided at the time of the relevant inspection if such additional work is required.

It should be noted that part of the proposed development will be located at the crest of the sandstone cut/cliffline that runs along the north-eastern site boundary, particularly the boathouse. Consequently, care must be taken to confirm that adverse defects are not present in the cut/cliffline prior to the commencement of construction. As the cut/cliffline cannot be inspected due to the presence of the existing retaining wall the surface of the bedrock must be inspected so that if presence adverse defects may be identified and remedial measured initiated. Cored boreholes may also be required to confirm the orientation of defects where any uncertainty exists. Jointing with a strike of 275^o that dipped out of the cut/cliffline was observed in the cliffline along the rear site boundary. Consequently, there is the possibility that adversely orientated jointing may be present across the site.

With respect to earthquakes, the site classifies as 'Class Be–Rock', in accordance with AS1170.4. A Hazard Factor (Z) of 0.08 is applicable for Sydney.

Prior to pouring concrete all footings must be free from all loose and softened material. We recommend that all footing excavations be inspected by a geotechnical engineer prior to the pouring of concrete to confirm that the design allowable bearing pressures have been achieved.

5.4 On-Grade Floor Slabs

The proposed lower ground floor slab will directly overlie bedrock across the majority of its footprint, however, the eastern extent of the slab will overlie fill. Due to the difficulties associated with the completion of earthworks on a small scale, where slabs on grade are proposed and sandstone bedrock is not exposed we recommend that all slabs be designed as suspended and founded in the underlying sandstone bedrock.

Underfloor drainage should be provided over the exposed sandstone subgrade. The underfloor drainage should comprise a strong, durable, single sized washed aggregate (eg. 'blue metal' gravel) so as to lead groundwater seepage to a sump for pumped or gravity disposal to the stormwater system.

5.5 Further geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Prepare dilapidation reports on the adjoining houses to north-east, south-west and north-west prior to the commencement of construction.
- Confirmation of the suitability of the bedrock to support the design footing loads where located within above a line drawn upwards from the toe of any cuts/clifflines.
- Confirmation by the structural engineer of the suitability of the existing retaining walls located along the southern end of the north-eastern boundary to support the applied loads. This will include an assessment of the rock mass for the presence of adverse defects which may also require the drilled of cored boreholes.
- Continuous or periodic vibration monitoring during percussive excavation, depending on the size of equipment and level of assurance required.
- Inspection of all vertical cuts through sandstone bedrock to allow any adverse defects to be identified and, where required remedial measures initiated.
- Inspection by a geotechnical engineer of all footing excavations prior to pouring concrete to confirm that the design ABP's have been achieved.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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

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

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

TABLE A

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**
	Stability of the natural hillside slope below residence	Stability of timber crib retaining wall	Stability of brick retaining walls	Stability of steep slope in rear yard	Low height sandstone block retaining wall	Stability of sandstone cliffline	Retaining wall along north-eastern site boundary – northern end	Retaining wall along north-eastern site boundary – southern end	Stability of new retaining walls
Assessed Likelihood	Barely Credible	Almost Certain	Unlikely	Possible	Possible	Possible	Unlikely	Likely	Barely Credible
Assessed Consequence	Major	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant	Insignificant
Risk	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low
Comments	Significant damage to house								

Assumed value of property is $5.5M

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**
	Stability of the natural hillside slope below residence	Stability of timber crib retaining wall	Stability of brick retaining walls	Stability of steep slope in rear yard	Low height sandstone block retaining wall	Stability of sandstone cliffline	Retaining wall along north-eastern site boundary – northern end	Retaining wall along north-eastern site boundary – southern end	Stability of new retaining walls
Assessed Likelihood	Barely Credible	Almost Certain	Unlikely	Possible	Possible	Possible	Unlikely	Likely	Barely Credible
Indicative Annual Probability	1×10^{-6}	1×10^{-1}	1×10^{-4}	1×10^{-3}	1×10^{-3}	1×10^{-3}	1×10^{-4}	1×10^{-2}	1×10^{-6}
Duration of Use of area Affected (Temporal Probability)	10 hours/day 4.17×10^{-6}	(i) 1 minute/week 6.94×10^{-4} (ii) 5 minutes/day 3.472×10^{-3}	(i) 1 hour/week 5.95×10^{-3} (ii) 5 minutes/week 4.96×10^{-4}	(i) 1 minute/week 6.94×10^{-4} (ii) 5 minutes/week 3.472×10^{-3}	(i) 1 minute/week 6.94×10^{-4} (ii) 5 minutes/week 3.472×10^{-3}	5 minutes/week 3.472×10^{-3}	(i) 5 minutes/week 4.96×10^{-4} (ii) 5 minutes/day 3.47×10^{-3}	(i) 5 minutes/week 4.96×10^{-4} (ii) 5 minutes/day 3.47×10^{-3}	(i) 5 minutes/day 3.47×10^{-3} (ii) 8 hours/day 3.31×10^{-1}
Probability of not Evacuating Area Affected	0.9 Little warning likely	(i) 0.9 Little warning (ii) 0.1 Warning likely	(i) 0.9 Little warning (ii) 0.1 Warning likely	(i) 0.9 Little warning (ii) 0.1 Warning likely	(i) and (ii) 0.9 Little warning	0.9 Little Warning	(i) 0.9 Little warning (ii) 0.1 Warning likely	(i) 0.9 Little warning (ii) 0.1 Warning likely	(i) and (ii) 0.9 Little warning. Walls likely to have gyprock covering and early signs of distress not likely to b observed.
Spatial Probability	1	2m/7m 0.29	3m/11m	(i) $9m^2/24m^2$ 0.375 (ii) 3m/6m 0.5	1m/12m 0.083	1m/12m 0.083	3m/4m 0.75	1m/8m 0.125	3m/25m 0.12
Vulnerability to Life if Failure Occurs Whilst Person Present	1 Likely to be buried	(i) 0.01 Likely to ride failure down (ii) 0.1 Unlikely to be buried	(i) 0.1 Likely to ride failure down (ii) 0.9 Likely to be buried	(i) 0.01 Likely to ride failure down (ii) 0.1 Unlikely to be buried	(i) 0.1 Likely to ride failure down (ii) 0.5 Unlikely to be buried	0.5 May be buried	(i) 0.1 Little to ride failure down (ii) 0.9 Likely be buried	(i) 0.01 Little to ride failure down (ii) 0.1 Unlikely to be buried	(i) 0.1 Little to ride failure down (ii) 0.9 Likely to be buried
Risk for Person most at Risk	3.8×10^{-7}	(i) 9.8×10^{-11} (ii) 2.0×10^{-7}	(i) 1.5×10^{-9} (ii) 1.2×10^{-9}	(i) 3.4×10^{-10} (ii) 2.5×10^{-9}	(i) 7.4×10^{-10} (ii) 1.9×10^{-8}	1.96×10^{-8}	(i) 2.3×10^{-8} (ii) 3.4×10^{-9}	(i) 3.9×10^{-8} (ii) 6.2×10^{-9}	(i) 3.2×10^{-8} (ii) 3.8×10^{-11}
Total Risk for Person Most at Risk	7.4×10^{-7}								

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
3
1/1

Client: MR CHRISTOPHER HO

Project: PROPOSED RESIDENCE

Location: 16 ADDISON ROAD, MANLY, NSW

Job No. 30147Z **Method:** HAND AUGER **R.L. Surface:** ≈ 13.8m

Date: 7/2/17 **Datum:** AHD

Logged/Checked by: L.M./A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown, with organic material.	D			GARDEN BED
									FILL: Silty sand, fine to medium grained, brown, trace of fine to medium grained sandstone gravel.				APPEARS POORLY COMPACTED
						0.5			END OF BOREHOLE AT 0.4m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 4

1/1

Client: MR CHRISTOPHER HO
Project: PROPOSED RESIDENCE
Location: 16 ADDISON ROAD, MANLY, NSW

Job No. 30147Z **Method:** HAND AUGER **R.L. Surface:** ≈ 13.8m
Date: 7/2/17 **Datum:** AHD
Logged/Checked by: L.M./A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS	0			CONCRETE: 80mm.t				NO OBSERVED REINFORCEMENT
								-	FILL: Silty sand, fine to medium grained, light brown, trace of fine to medium grained sandstone gravel and ash.	D			APPEARS POORLY COMPACTED
									END OF BOREHOLE AT 0.3m				HAND AUGER REFUSAL ON OBSTRUCTION IN FILL
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **5**
1/1

Client: MR CHRISTOPHER HO
Project: PROPOSED RESIDENCE
Location: 16 ADDISON ROAD, MANLY, NSW

Job No. 30147Z **Method:** HAND AUGER **R.L. Surface:** ≈ 13.8m
Date: 7/2/17 **Datum:** AHD
Logged/Checked by: L.M./A.Z.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, light brown, with fine to medium grained sandstone gravel, trace of brick fragments and ash.	D			SUB FLOOR SPACE APPEARS POORLY COMPACTED
									END OF BOREHOLE AT 0.3m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No. 108

1/1

Client: MR CHRISTOPHER HO

Project: PROPOSED RESIDENCE

Location: 16 ADDISON ROAD, MANLY, NSW

Job No.: 30147PN2 **Method:** HAND AUGER **R.L. Surface:** ≈ 5.8m

Date: 12/6/19 **Datum:** AHD

Plant Type: N/A **Logged/Checked by:** S.M./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0		-	FILL: Silty sand, medium to coarse grained, brown, trace of root fibres and sub angular fine to medium grained sandstone gravel.	M			GRASS COVER APPEARS POORLY TO MODERATELY COMPACTED
									FILL: Silty clay, high plasticity, light grey.	w>PL			
						0.5			FILL: Silty sand, medium to coarse grained, light yellow brown, trace of sub angular fine to medium grained sandstone gravel.	M			
									FILL: Silty clay, high plasticity, light grey.	w>PL			
									FILL: Silty sand, medium to coarse grained, yellow brown, with clay fines and angular fine to medium grained sandstone gravel.	M			
						1			END OF BOREHOLE AT 0.9m.				HAND AUGER REFUSAL
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MR CHRISTOPHER HO		
Project:	PROPOSED RESIDENCE		
Location:	16 ADDISON ROAD, MANLY, NSW		
Job No.	30147Z	Hammer Weight & Drop: 9kg/510mm	
Date:	7-2-17	Rod Diameter: 16mm	
Tested By:	L.M.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration

Test Location	RL≈11.2m	RL≈11.4m	RL≈13.8m	RL≈13.8m	RL≈11.6m	RL≈12.0m	
Depth (mm)	**1**	**2**	**3**	**4**	**5**	**6**	
0 - 100	3	1	1	EXCAVATED	1	1	
100 - 200	1	2	1	3	4	3	
200 - 300	1	1	2	3	5	3	
300 - 400	4	3	3	REFUSAL	REFUSAL	4	
400 - 500	5	4	REFUSAL			REFUSAL	
500 - 600	REFUSAL	4					
600 - 700		REFUSAL					
700 - 800							
800 - 900							
900 - 1000							
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MR CHRISTOPHER HO
Project:	PROPOSED RESIDENCE
Location:	16 ADDISON ROAD, MANLY, NSW

Job No.	30147PN2	Hammer Weight & Drop: 9kg/510mm
Date:	12-6-19	Rod Diameter: 16mm
Tested By:	S.M.	Point Diameter: 20mm

Test Location	**101a**	**101b**	**102**	**103**	**104**	**105**	**106**
Surface RL	≈8.3m	≈8.3m	≈6.6m	≈6.7m	≈6.7m	≈8.1m	≈5.8m
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK	5	1	SUNK	2	SUNK	SUNK
100 - 200	5	12	2	1		2	2
200 - 300	4	2	1	1	2	3	4
300 - 400	4	1	2	1			2
400 - 500	8	5	3	1		1	2
500 - 600	5	3	6	1		1	3
600 - 700	2	5	4	3	3	2	19/60mm
700 - 800	1	17	3	3	2	4	REFUSAL
800 - 900	3	REFUSAL	3	2	2	2	
900 - 1000	12		3	2	2	14	
1000 - 1100	REFUSAL		6	2	3	7	
1100 - 1200			11	3	4	9	
1200 - 1300			8/20mm	4	5	7	
1300 - 1400			REFUSAL	4	10/70mm	20	
1400 - 1500				14	REFUSAL	14	
1500 - 1600				REFUSAL		11	
1600 - 1700						20	
1700 - 1800						30/70mm	
1800 - 1900						REFUSAL	
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

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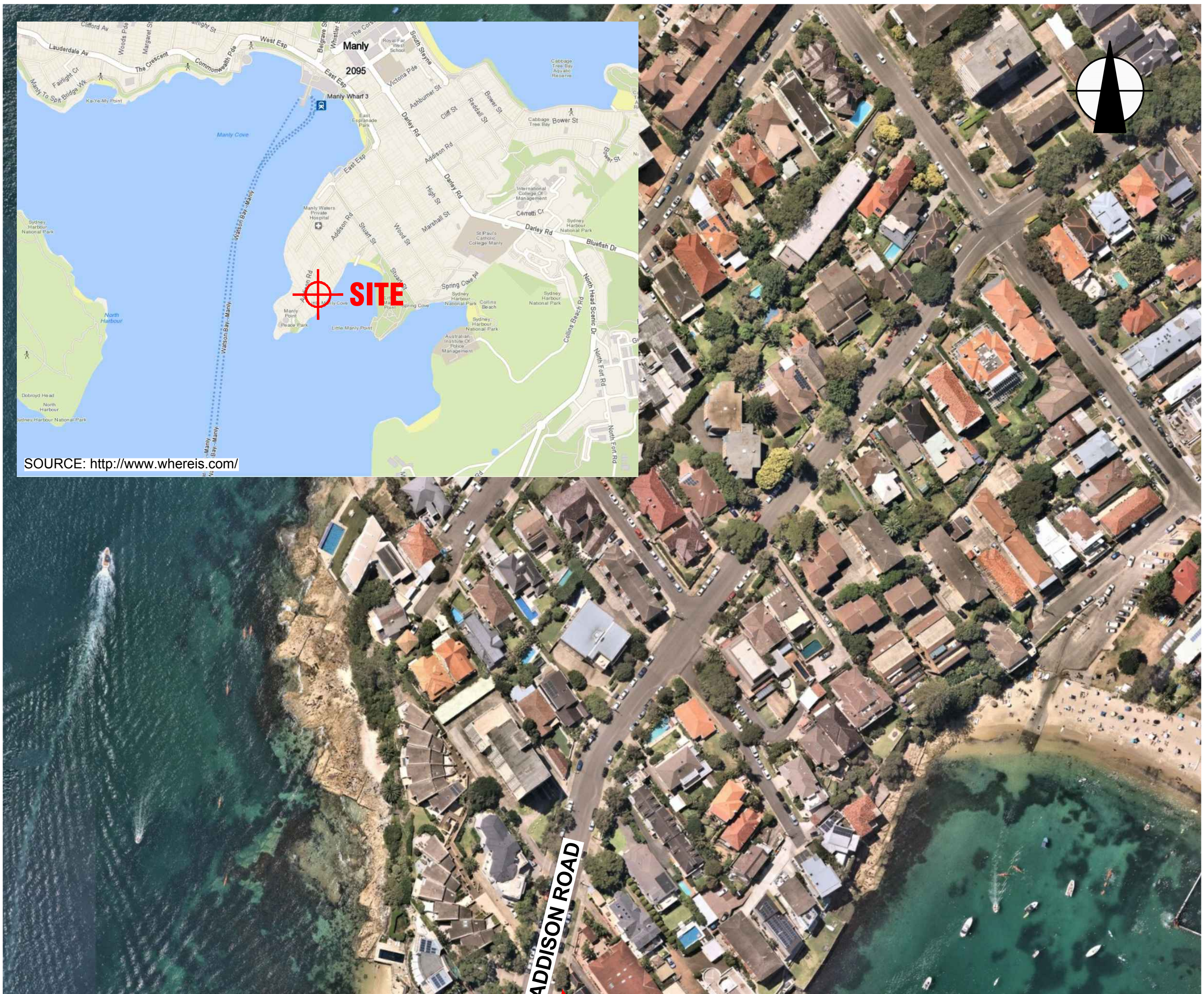
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MR CHRISTOPHER HO		
Project:	PROPOSED RESIDENCE		
Location:	16 ADDISON ROAD, MANLY, NSW		
Job No.	30147PN2	Hammer Weight & Drop: 9kg/510mm	
Date:	12-6-19	Rod Diameter: 16mm	
Tested By:	S.M.	Point Diameter: 20mm	

Test Location	**107**	**108**	**109**	**110**			
Surface RL	≈6.0m	≈5.8m	≈4.9m	≈4.8m			
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	SUNK	1	1	1			
100 - 200	1	1	1	2			
200 - 300	2	4	6	1			
300 - 400	2	3	7	1			
400 - 500	2	4	4	3			
500 - 600	20/90mm	2	3	20			
600 - 700	REFUSAL	2	8	10/20mm			
700 - 800		4	23	REFUSAL			
800 - 900		REFUSAL	20/70mm				
900 - 1000			REFUSAL				
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	16 ADDISON ROAD, MANLY, NSW
Report No: 30147SY	Figure No: 1

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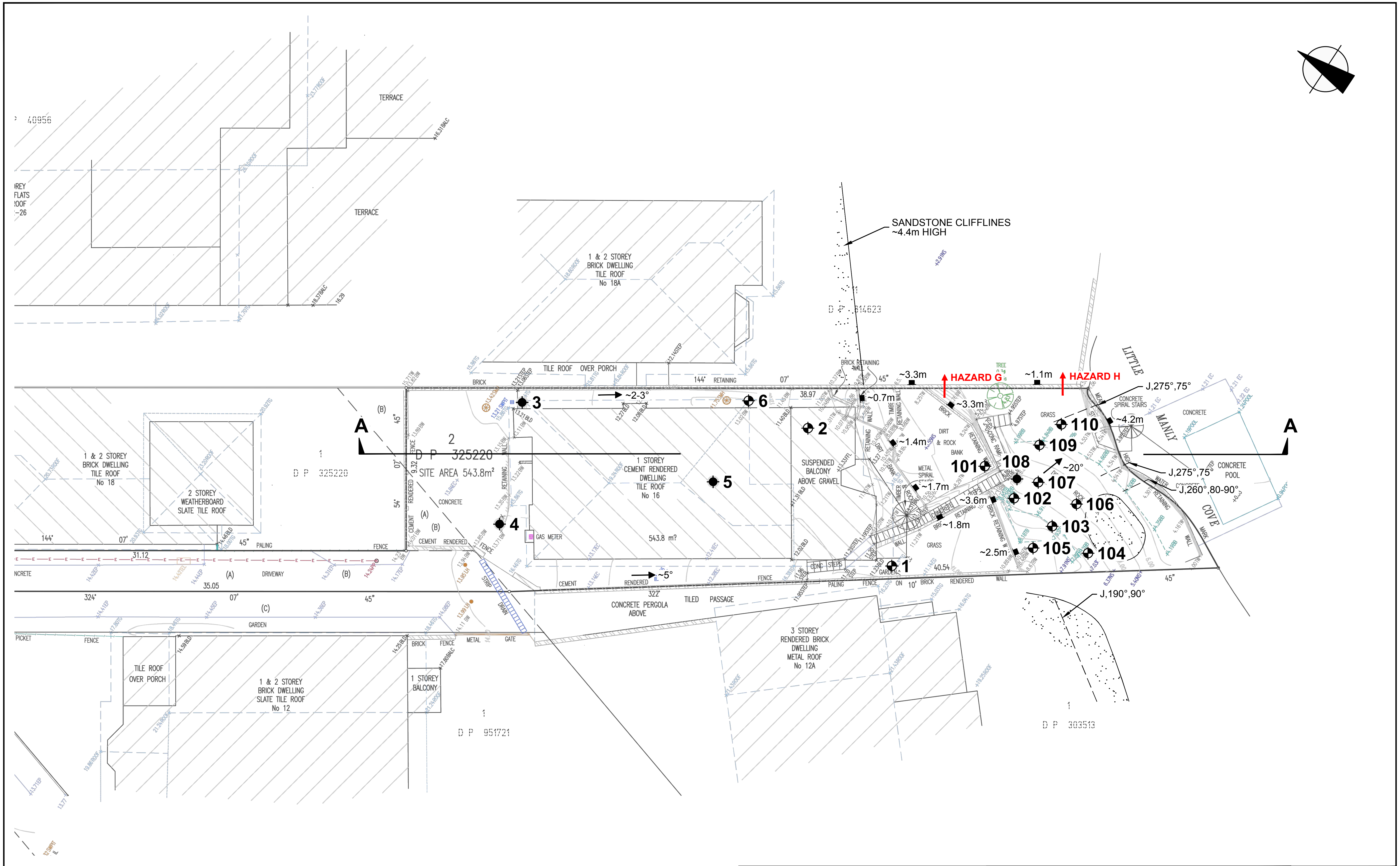
PLOT DATE: 9/04/2021 10:06:07 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30147Z MANLY\CAD\2021\30147SY.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST

NOTES:
1. REFER TO FIGURE 3 FOR SECTION A-A.
2. REFER TO FIGURE 4 FOR GEOTECHNICAL MAPPING SYMBOLS.

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

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

Title:	GEOTECHNICAL MAPPING PLAN
Location:	16 ADDISON ROAD, MANLY, NSW
Report No:	30147SY
Figure No:	2

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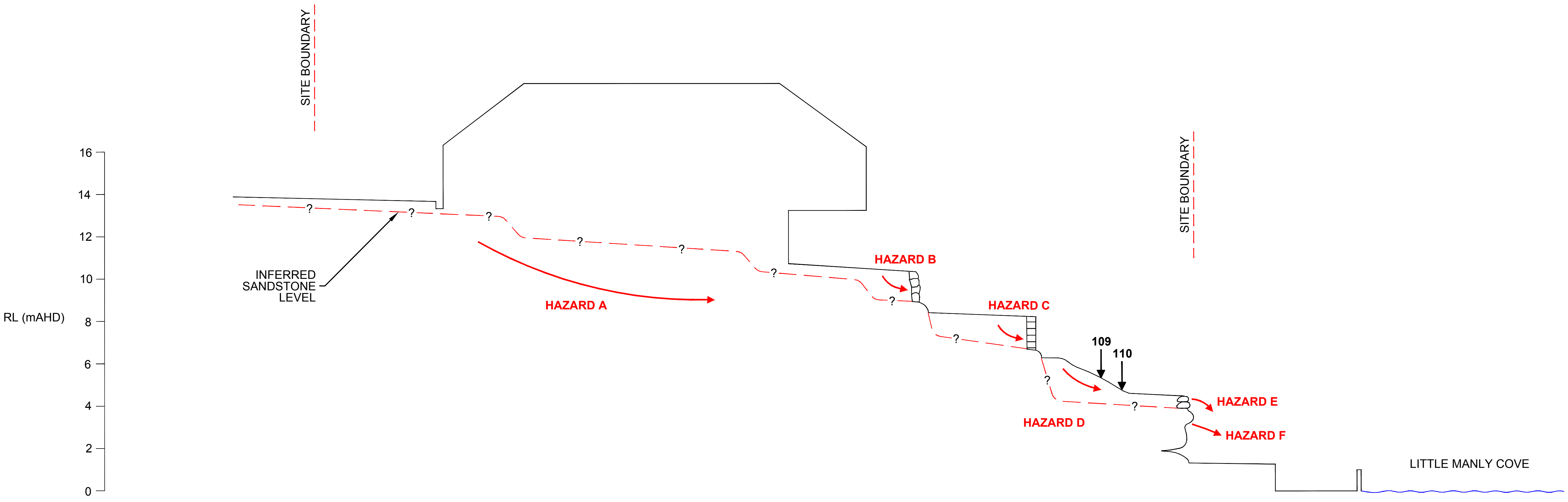


SCALE 1:200 @A3 METRES (0 2 4 6 8 10)	Title: **SECTION A-A**
	Location: 16 ADDISON ROAD, MANLY, NSW
	Report No: 30147SY — Figure No: 3
This plan should be read in conjunction with the JK Geotechnics report.	JKGeotechnics

PLOT DATE: 9/04/2021 10:06:47 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30147Z MANLY\CAD\2021\30147SY.DWG

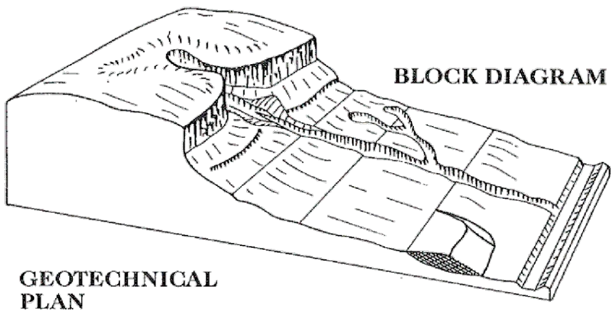
TOPOGRAPHY

Symbol	Ground Profile		
		convex	well defined or angular break of slope
		concave	
		convex	poorly defined or smooth change of slope
		concave	
	breaks of slope		convex and concave too close together to allow the use of separate symbols
	changes of slope		
	sharp		ridge crest
	rounded		
3	Cliff or escarpment or sharp break 40° or more (estimated height in metres)		
15	Uniform Slope		Slope direction and angle (Degrees)
10	Concave Slope		
8	Convex Slope		
	Top		Cut or fill slope, arrows pointing down slope
	Bottom		
	Hummocky or irregular ground		

OTHER FEATURES

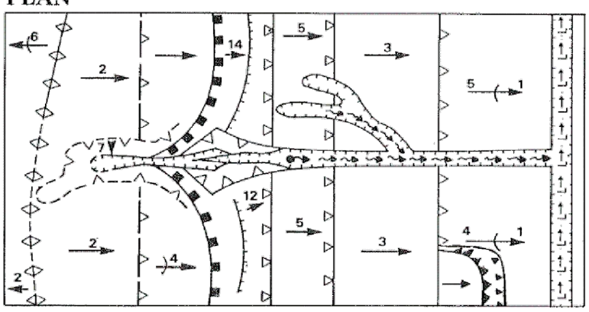
- Boulder
- Seepage/spring
- Swallow hole for runoff
- Natural water course
- Open drain, unlined
- Open drain, lined
- Fenceline
- Property boundary
- Dry Stone Wall
- J —— J 200 — Major joint in rock face (opening in millimetres)
- – T – T – 10 — Tension crack (opening in millimetres)
- Masonry or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

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

Title:	GEOTECHNICAL MAPPING SYMBOLS
Location:	16 ADDISON ROAD, MANLY, NSW
Report No: 30147SY	Figure No: 4

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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

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

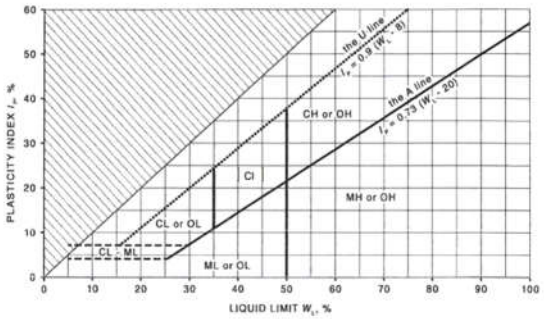
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



JKGeotechnics

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

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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

APPENDIX A

LANDSLIDE RISK

MANAGEMENT

TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

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

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

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

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

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

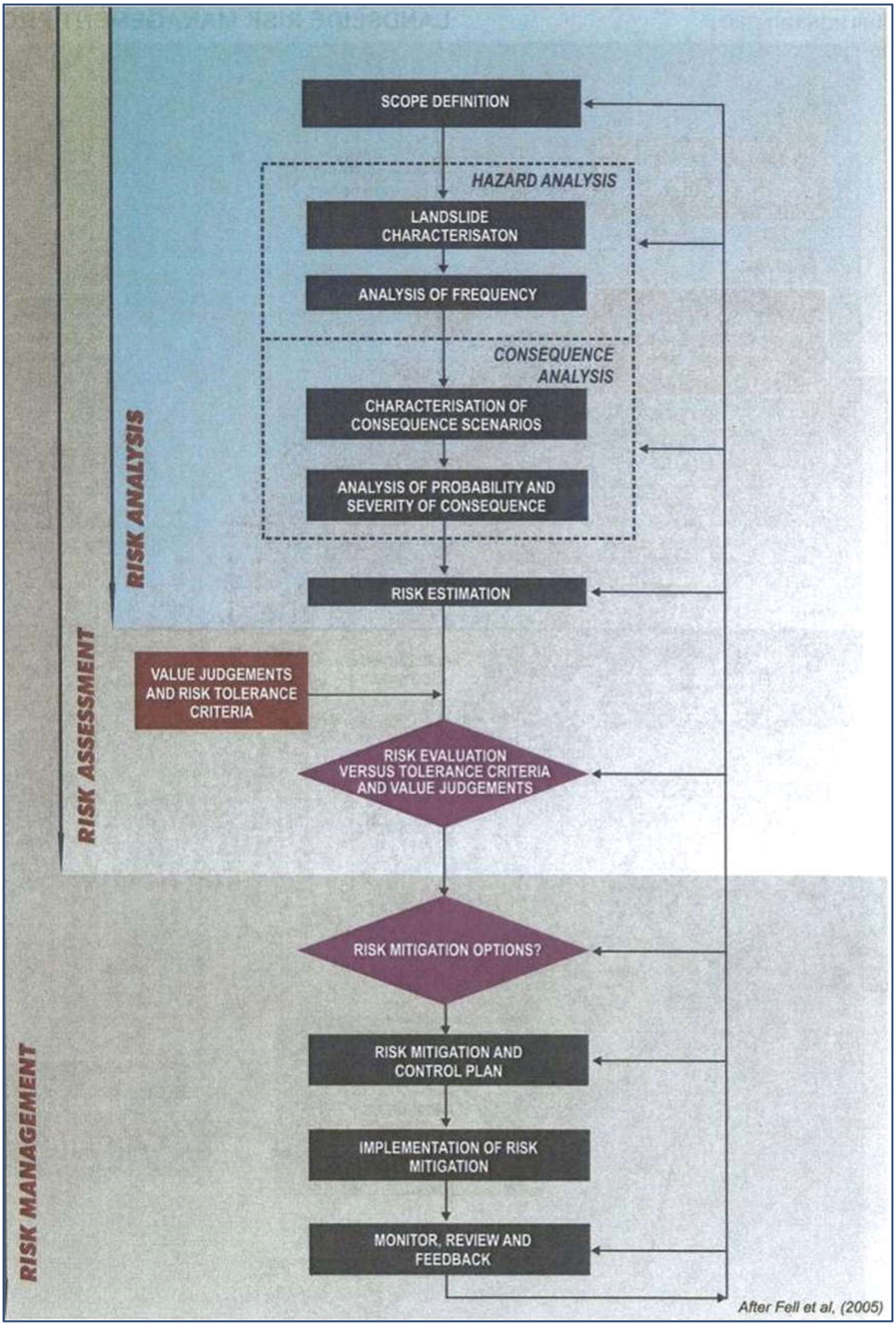


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

RISK LEVEL IMPLICATIONS

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

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

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

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

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

What Causes a Landslide?

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

Does a Landslide Affect You?

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

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

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

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

TABLE 1 – Slope Descriptions

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

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

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

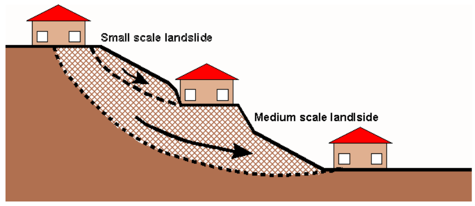


Figure 1

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

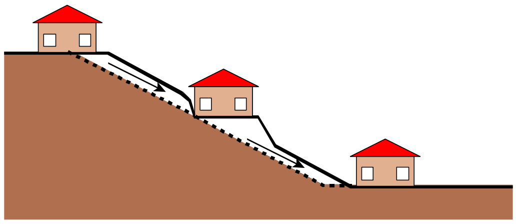


Figure 2

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

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

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

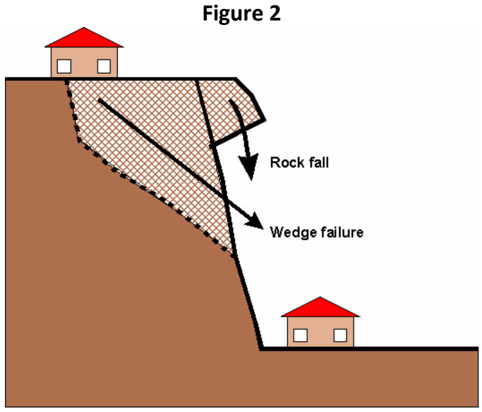


Figure 3

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

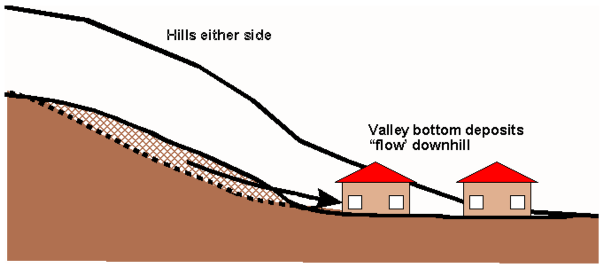


Figure 4

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

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

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

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

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

Landslide Risk Assessment

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

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

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

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

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

TABLE 2 – LIKELIHOOD

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

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

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

TABLE 1 – RISK TO PROPERTY

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

Risk to Life

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

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

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

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

TABLE 3 – RISK TO LIFE

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

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

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APPENDIX B

SOME GUIDELINES

FOR

HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

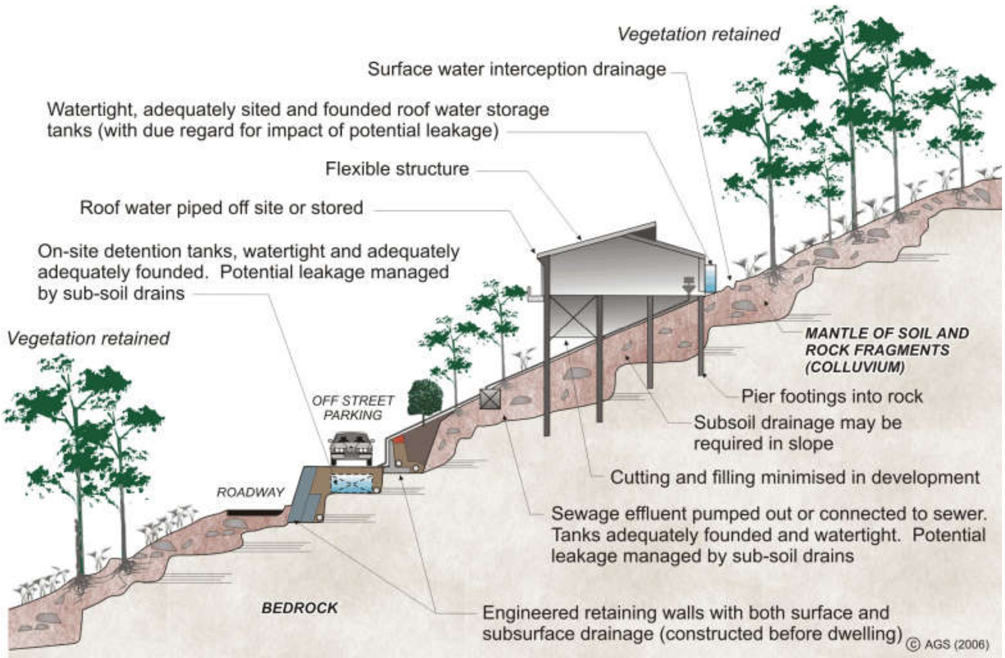
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having 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.

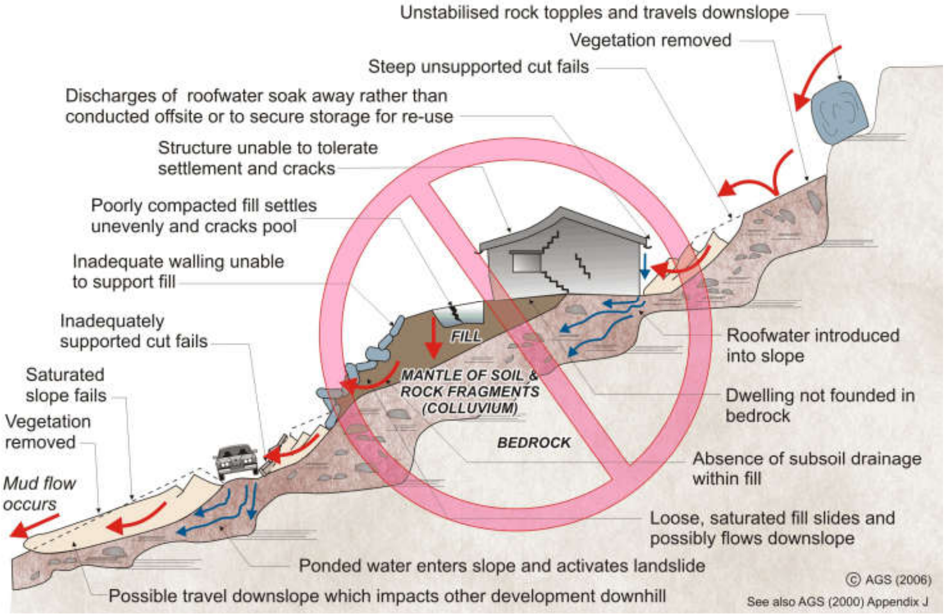
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

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

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

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

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