



REPORT TO
SQUILLACE ARCHITECTS PTY LTD

ON
**GEOTECHNICAL INVESTIGATION AND SLOPE
STABILITY RISK ASSESSMENT**

FOR
PROPOSED ALTERATIONS AND ADDITIONS

AT
88 BOWER STREET, MANLY, NSW

Date: 30 June 2021

Ref: 33662BMrpt

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ATTACHMENTS

Table A: Summary of Risk Assessment to Property

Table B: Summary of Risk Assessment to Life

Borehole Logs 2 and 5

Dynamic Cone Penetration Test Results Sheet (DCP1 to DCP5)

Figure 1: Site Location Plan

Figure 2: Borehole and DCP Test Location Plan

Figure 3: Geotechnical Mapping and Hazards Plan

Figure 4: Section A-A Showing Geotechnical Hazards

Figure 5: Geotechnical Mapping Symbols

Vibration Emission Design Goals

Report Explanation Notes

APPENDIX A: Landslide Risk Management Terminology

1 INTRODUCTION

This report presents the results of a geotechnical investigation and slope stability risk assessment for the proposed Alterations and additions to the existing residence at 88 Bower Street, Manly, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Jackson Perry of Squillace Architects Pty Ltd, on behalf of the property owners, and was carried out in accordance with our proposal dated 16 October 2020 (Ref. P52850B).

From review of Squillace Architecture drawings (Job No BOW-2020, DA30-33, 099 & 100-102, Revision A, dated 25 June 2021), we understand the proposed alterations and addition will include the following:

- Demolition of the existing driveway, roof, selected internal and external walls, various landscaping walls and some trees.
- Reconstruction of a 3 and 4 storey house (Lower Ground Floor, Ground Floor Level, Level 1 and Entry Level) with extensions and alterations to each level as described below.
- The lower ground floor level will be at RL3.8m and will extend up to about 8.5m further into the hillside than the existing lower ground floor level, for a proposed spiral staircase and lift in the south-eastern corner. The required excavation of about 2.9m depth below the existing ground floor level, will be set back from the eastern boundary by about 2m. The proposed lower ground floor will also require excavation for a proposed boat store and outdoor shower at RL4.060m requiring a similar maximum depth of excavation (2.5m from the top of a rubble retaining wall) but extending to the western boundary where the depth of excavation reduces and the finished floor level would be similar to the side path of the neighbouring property No 92.
- The proposed ground floor level will be at RL6.4m will also be extended into the hillside (towards Bower Street) for a laundry and Bedroom 2 and an adjacent patio to its west, for about 5m (horizontally), requiring excavation to maximum depths of about 2.9m but generally to about 1m.
- Where some of the existing walls are to remain portions of the walls below may be removed and this will require temporary support during construction and potentially additional permanent support.

In accordance with the requirements of the Manly DCP, the purpose of the geotechnical assessment is to assess the site for geotechnical stability hazards, and provide comments and recommendations on measures required to reduce the risk of landslip and subsidence of existing areas, where applicable. The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on excavation, seepage, retention, footings and further geotechnical input required.

2 ASSESSMENT AND INVESTIGATION PROCEDURE

2.1 Walkover Survey and Stability Assessment

The stability assessment is based upon a detailed inspection by our Associate Geotechnical Engineer, Matthew Pearce on 16 December 2020, of the topographic, surface drainage and geotechnical conditions of the site and its immediate environs. These features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the proposed development. The attached Appendix A defines the terminology adopted for the risk assessment together with a flow chart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

The principal geotechnical features, which were measured using taped measurements and hand held clinometer, are presented on Figure 3 using the existing survey by Bee & Lethbridge Pty Ltd (Ref. 15700, Rev 0, dated 23 August 2006) as a base plan. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 4 presents a typical cross-section through the site based on the survey data augmented by our mapping observations. Figure 5 defines the mapping symbols used.

2.2 Subsurface Investigation

To complement the observation of geotechnical features at the surface, Dynamic Cone Penetrometer (DCP) tests were carried out at five locations (DCP1 to DCP5) to refusal at depths ranging from 0.5m to 1.1m below the existing ground surface. At two of these locations hand augered boreholes (BH2 and BH5) were drilled to refusal at depths of 1.17m and 0.34m, respectively, to investigate the subsurface profile.

The DCP test and borehole locations are shown on Figure 2 and were recorded by taped measurements from site features shown on the survey plan. The surface reduced levels (RLs) were estimated by interpolation between spot heights and contours shown on the survey and are therefore approximate. The datum is the Australian Height Datum (AHD).

The refusal depth of hand augered boreholes and DCP tests can be inferred to represent the surface of the bedrock, but refusal may occur on other hard layers. To confirm the depth of the bedrock additional investigation methods involving diamond coring of the rock would be required, but were not considered warranted at this stage of the project.

Groundwater observations were made in the boreholes during drilling and shortly after completion. No longer-term monitoring of groundwater levels was carried out.

Our geotechnical engineer, Mr Sami Azzi, set out the borehole and DCP test locations, recorded the DCP test results and prepared logs of the subsurface conditions encountered. The borehole logs, which include groundwater observations, and the DCP test results are attached together with a set of explanatory notes

which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The following should be read with reference to Figures 1 and 3 in particular.

The site is located at the toe of a moderately steep hill that steps down to the north-west towards Fairy Bower Beach. The property has an irregular but almost triangular shape and also steps down in this direction, from Bower Street along its southern side, to its narrow frontage with Marine Parade on the northern side. The site is about 50m to 60m long (north-south) by about 25m wide at Bower Street and 4m wide at its northern end. Surface reduced levels (RLs) range from about RL13m in the south-eastern corner to RL3m at the northern end.

The site is occupied by a two and three storey brick house with a suspended driveway from Bower Street to an upper garage level which is integrated with the main building. There is a wide terraced southern garden and a relatively flat narrow northern garden. The site features a stone retaining wall of maximum 2.8m in height supporting Bower Street and an unsupported rock face of up to 3.6m in height towards the northern end of the eastern boundary.



Plate 1: looking southwards from Marine Parade

The house appeared to be in good condition with no cracks observed during our cursory viewing. The lower ground floor level is at a similar level to the northern garden and the promenade of Marine Parade. (Refer to

Plate 1.) There is a concrete path leading around the western side of the house to a lower ground floor store room and then some steps leading up to the southern garden at about the ground floor level. The rear (southern) wall of the storeroom was clearly damp inferring it is a retaining wall. Internal stairs provide access up from the ground floor level to the 1st Floor and garage. The eastern side of the house is generally set back about 1m from the common boundary with unsurfaced access via a northern side balcony at Level 1 on the northern side of the building. There is also a narrow crawl space along the eastern side of the garage from the south, off the driveway.

The driveway is set back about 1m from the boundary. Beneath the eastern side of the driveway is a masonry retaining wall of 0.5m to 1.5m in height, supporting a garden bed at similar levels to the driveway surface. The adjacent garden of No 86 is at similar levels to the planter bed/driveway.

The stone retaining wall along Bower Street is inclined about 75° back into the slope and appeared in good condition, although fig trees are growing out of the wall. Refer to Plates 2 and 3.



Plate 2

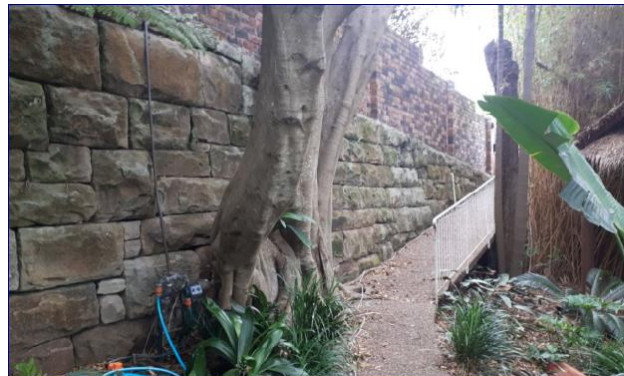


Plate 3

Along the top of the Bower Street stone wall is a 'single skin' brick fence, approximately 1.8m in height, for the full length of the wall. The brick fence has been lifted about 10mm off its base, and has other cracking in the vicinity of the fig tree(s). Within the property there is a concrete path adjacent to the base of the stone wall. The path leads up to a pedestrian gate at the western end where the path becomes suspended above gardens. The garden path links at its eastern end to the house at the 1st floor level. Close to the house the path appears to be supported by a 1.7m high timber retaining wall. The garden between the Bower Street frontage and the house steps down to the west where it is supported by masonry and rubble retaining walls of about 1.5m height, over sandstone bedrock outcrops, and a further masonry retaining wall along the apparent boundary with No 92 Bower Street. Refer to Plates 4 to 6.

Plate 4



Plate 5



Plate 6



Most of the retaining walls appeared to be in reasonable condition although the rubble wall is an ‘ad-hock’ structure and one section of the lower masonry wall is clearly being pushed over by the growth of a palm tree (Plate 4). The southern garden is also landscaped with a row of detached sandstone boulders. Separate to the row of boulders is an exposure of sandstone, at the base of steps leading to the lawn. We expect this is bedrock but it could be an embedded boulder/‘floater’.

Beyond the lower masonry wall along the common boundary with No 92, is a side path and then a 2 and 3 storey apartment building, which is cement rendered with a stone masonry frontage. The apartment building appeared to be in good condition. The southern end the masonry retaining walls within No. 92 (Refer Plate 6) appeared to support a partially suspended parking bay. At the northern end of No 92 is a garden with an inground pool. Surface levels were similar levels across the boundary, except at the very southern end as described above.

To the north of the house a rock face is present along the eastern boundary ranging from about 1m to 3.6m in height, with the greatest height at the corner of the house within No 88. The rock features two subvertical joints striking at an acute angle to the rock face, and a horizontal (discontinuous) shale seam about 1m to 1.5m above the garden level (with some fractured rock above), forming a large detached boulder and a potential flake of rock behind it. Refer to Plates 7 and 8.



Plate 7

Plate 8: view from balcony above



The rockface appears to extend southwards but with only a very narrow gap between it and the eastern wall of the house this could not be confirmed. The strength of the exposed rock was generally assessed to be at least medium strength. At the crest of the rock face is a stone wall supporting a cement rendered house within the adjoining property at No 86. About mid-length along the eastern boundary the neighbouring garage abuts the boundary and is supported by a 1.6m to 2m high masonry wall which is tilting over towards the subject site. Refer to Plate 9.

Plate 9: Looking southwards, at about Level 1



To the east of the site is No 86, containing a two-storey house with pool on its northern side, both built high up from the outcropping rock on the common boundary. There appears to be basement or subfloor level(s) below the house. On the Bower Street side of the house is a free-standing rendered garage also abutting the common boundary and this is supported by the above described retaining wall.

The structures adjacent to the site appeared in good condition with the exception of the garage retaining wall as described above.

Marine parade, to the north of the site, is an oceanside promenade with a sea wall on its northern side. Sandstone bedrock is visible in the seabed in close proximity to the site (directly to the north).

Bower Street to the south of the site comprises a concrete footpath, grass verge and asphaltic concrete road with kerb and gutter. It slopes at about 4° down to the west.

3.2 Geology and Subsurface Conditions

The Sydney 1:100,000 geological map indicates the site is underlain by Hawkesbury Sandstone.

As described in Section 3.1 above, sandstone bedrock was exposed at the northern end of the eastern boundary, centrally in the southern garden and at the southern end of the western boundary, as indicated on Figure 3. The boreholes refused at relatively shallow depths (1.15m and 0.35m for BH2 and BH5, respectively) with the DCP tests also refusing at shallow depths ranging from about 0.2m to 1.2m. These levels are consistent with the toe levels of nearby retaining walls and rock outcrops, hence we infer the DCP test and borehole refusal depths represent the surface of rock, although it is possible refusal may have been caused by floaters or other obstructions.

The field strength assessment of the exposed rock outcrops was at least medium strength. A discontinuous shale seam of lower strength, at about RL4.5m, was observed on the eastern boundary exposure. There was also some fracturing associated with the closely spaced jointing beneath the large detached boulder on that eastern rock face.

The boreholes encountered silty sandy fill with various minor inclusions of root fibres, sandstone gravel and tile fragments and slag. The fill was assessed as poorly compacted.

No groundwater seepage was encountered during or on completion of drilling.

For more details at specific test locations, reference should be made to the borehole logs and DCP test results. DCP1 has also been plotted on Section A-A on Figure 4 illustrating the typical stepped profile of the Hawkesbury Sandstone.

4 SLOPE STABILITY RISK ASSESSMENT

Based on our site inspection we consider that the potential landslip hazards at the site are as follows:

- Hazard A: Failure of exposed sandstone bedrock outcrops (Other than Hazard B) throughout the property.
- Hazard B: Failure of the potentially detached boulder located on the sandstone outcrop on the eastern boundary to the north of the existing house.
- Hazard C: Failure of the sandstone retaining wall supporting Bower Street.
- Hazard D: Failure of the various retaining walls in reasonable condition supporting garden areas within the southern portion of the site.
- Hazard E: Failure of tilting retaining wall supporting neighbouring garage along the eastern boundary.
- Hazard F: Failure of proposed rock faces and engineer designed retaining walls supporting the excavations for the extended lower ground floor and ground floor levels.

We note that while minor soil slopes exist within the southern garden, they are relatively limited and shallow and so the consequence of any failure would be insignificant and so have therefore not been formally assessed.

The attached Tables A and B summarise our qualitative and semi-quantitative assessments of each potential hazard for the risk to property and the risk to life, respectively, should instability occur. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. The qualitative assessments are based on judgements made in the field by the geotechnical engineer and in this regard are subjective and formed in part by the engineers' previous experiences.

The Management Concepts and Guidelines prepared by the Australian Geomechanics Society, Sub-Committee on Landslide Risk Management (Reference 1) recommend an acceptable risk for loss of life for the person most at risk of 1×10^{-5} for existing slopes/structures and 1×10^{-6} for new developments and this has been adopted for this risk assessment. For loss of property the acceptable risk should be determined by the owner, provided loss to property only affects the owners' property and does not impact on the property of others. In accordance with Reference 1 an acceptable risk of loss of property posed by existing slopes as 'Low' has been adopted for this risk assessment. Where risks posed by slope instability are considered unacceptable, remedial measures should be adopted to reduce the risk posed to an acceptable level.

As shown in Table A we assess the risks to property to be "Very Low" or "Low", which would be considered 'acceptable' in accordance with the criteria given in Reference 1. For Hazard B we have assumed that the boulder will be supported by an engineered design retaining walls as discussed in Section 5.1 below. For Hazard C we have assumed that the fig tree growing in the wall will be monitored by the owner and removed

if it starts to destabilise the wall, otherwise, if this was not carried out the risk would be “Moderate”, which is considered ‘unacceptable’. As shown in Table B, our assessed risk to life for the person most at risk is about 10^{-6} , which would also be considered ‘acceptable’ in accordance with Reference 1.

Provided our comments and recommendations given below are followed, the risk to both property and life for the person most a risk following the construction of the proposed alterations and additions will be acceptable provided our recommendations in Section 5 are adopted.

5 COMMENTS AND RECOMMENDATIONS

5.1 Primary Geotechnical Considerations

The risk assessment has identified the following three hazards requiring attention:

- The large potentially detached sandstone boulder on the eastern boundary is potentially unstable (Hazard B). It was discussed during the site meeting with the structural engineer, Ms Marie O’Looney, of SDA Structures, that it may be difficult to arrange permission to stabilise the boulder using permanent rock bolts as the bolts would extend across the boundary. Instead it was agreed to permanently support the boulder and fragmented rock around it with a new retaining wall constructed as part of the lower ground floor patio works.
- The fig tree growing in the stone retaining wall supporting Bower Street (Hazard C) must be monitored in the future for potential destabilising actions on the stone retaining wall, although it should be noted it has already compromised the integrity of the brick boundary fence above it.
- The tilting retaining wall supporting the garage of No 86 (Hazard E) will require stabilisation prior to any excavation or demolition works that may be required below it.

Our risk assessment has assumed that these works will be carried out as part of the proposed alterations and additions.

In addition to the above, the principal geotechnical issues for the proposed alterations and additions will be maintaining stability to the excavation faces and nearby structures during excavation into the hillside for the extended lower ground floor and ground floor levels. Excavation is expected to be mostly through medium to high strength sandstone bedrock, but the rock faces must be progressively inspected by a geotechnical engineer so that any unstable defects can be identified and stabilisation carried out whilst the identified defects are within reach, to protect persons working below and any structures within the zone of influence above. In addition, vibration emissions from excavation activities must be controlled to prevent potential damage to structures both within the subject site and neighbouring properties.

Based on the investigation results sandstone is expected to be at shallow depth beneath the entire building footprint presenting a good stratum for footings.

These and other issues are discussed further below but the proposed development is feasible from a geotechnical perspective provided the following comments and recommendations are followed.

It is worth noting that while groundwater was not encountered within the depth of the boreholes (base of BH5 was RL 3.15m AHD), coastal storm surge related water may become an issue during construction and possibly in the long term. A coastal engineer should be consulted for further advice.

5.2 Dilapidation Surveys

Prior to demolition and excavation, detailed dilapidation surveys should be carried out on the adjoining properties to the west and east. The dilapidation surveys should comprise detailed inspections of the structures, both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of the existing conditions.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works and in this way can guard against opportunistic claims for damage that was present prior to the start of the works.

5.3 Demolition

The proposed partial demolition of existing walls may reveal previously cut rock faces and may remove of support from some of the walls above that are to remain or existing retaining structures, such as the tilting wall on the eastern boundary below the neighbouring garage. It is possible that some of the soil to the east of the house within No. 88 is providing some passive restraint to the retaining walls supporting No. 86. There is also an existing garden located opposite the adjoining garage, at Level 1 (estimated to be at approximately RL9m) which, in plan, steps into the existing building footprint. It appears that this garden is suspended above the ground floor level. Therefore, care must be taken during demolition.

Demolition must be carefully planned and carried out to reduce the risk of instability during demolition. We recommend that prior to the start of any demolition the structural engineer prepares a detailed demolition methodology/plan nominating the sequence of the demolition works and any propping required to provide support during demolition. The methodology must include appropriate structural engineer's inspections and where required provision for excavation of small test pits to investigate the footing conditions of any retaining structures where excavations are required in front of those walls. The geotechnical engineer should also inspect any test pits that are excavated during construction to provide additional advice to the structural engineer. The methodology will need to be revised and updated during construction as conditions are exposed.

5.4 Excavation

Excavation for the proposed lower ground floor and ground floor level extensions southwards into the hillside is expected to be required to depths of about 1m to 3m.

Excavation to such depths will likely encounter a limited depth of sandy fill and then sandstone bedrock. Based on our limited field strength assessment of the outcrops in the north-eastern and south-western portions of the site, the sandstone is expected to be of medium to high strength but further investigation comprising cored boreholes prior to demolition is advised to confirm the rock strength in the pertinent area to reduce potential variations of excavation tenders and to provide a further indication of the likely stabilisation treatment that may be required.

The excavation equipment that can be used will be dictated by access considerations. We expected that only a small excavator of say 5 tonnes in size may be able to be used. If access for larger equipment is possible additional advice on the precautions during excavation should be obtained.

Excavation of fill, any residual soil and extremely weathered sandstone bedrock that may be present is expected to be achievable using conventional techniques, such as buckets and ripping tynes fitted to a small hydraulic excavator. Excavation of the sandstone bedrock will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws.

Since the excavations will be carried out close to the portions of the existing house that will remain and the potentially vibration sensitive (cement rendered) neighbouring buildings, and retaining walls (including a potentially unstable retaining wall (Hazard E)), we recommend that low vibration emitting equipment be used, such as rock saws, ripping hooks, rotary grinders, and hand held jack hammers.

If hydraulic rock hammers are to be used, they should be limited in size and the excavation commenced away from likely critical areas (i.e. as far as possible from existing structures) to allow monitoring of transmitted vibrations prior to excavation close to the adjoining structures. The vibrations transmitted to the structures within the subject site and the adjoining sites must be quantitatively monitored at all times during rock hammer use. Vibration monitors should be solidly fixed to the existing walls, with the monitors attached to flashing warning lights, or other suitable warning systems, so that the operator is aware when acceptable limits have been reached at which point such excavation techniques should cease. It is possible that excavation using a rock hammer may not be possible while maintaining the transmitted vibrations within acceptable limits.

Vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec. However, if any particularly sensitive structures or equipment are present in adjacent properties then a lower target limit may be appropriate. The appropriate limit should be assessed following review of the dilapidation reports.

If higher vibrations are recorded than the target limits, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be feasible depending on the associated vibration frequency. However, any on site warning devices can only be set against the PPV and not the associated vibration frequency so will need to be set for the lower PPV values. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative lower percussion techniques as discussed above.

We recommend use of excavation contractors experienced in such work and with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this report and have all appropriate statutory and public liability insurances.

5.5 Groundwater

Given the ground profile is predominantly sandstone bedrock at shallow depths, on a hillside, and no groundwater was encountered in the boreholes, groundwater is not expected to be a significant issue for the proposed development. Some small volumes of water may be perched on undulations on the rock surface and possibly in joints and seams in the rock. Such seepage from rock usually reduces following initial excavation, but should be expected to increase during and following rain. Such seepage is expected to be readily managed by gravity drainage and sump and pump techniques.

In the long term, drainage should be provided behind all retaining walls, and possibly below the lowest slab, to control and direct any seepage that does occur. The completed excavation should be inspected by the geotechnical and hydraulic engineers to confirm if the designed drainage system is adequate for the actual seepage flows.

5.6 Batters and Rock Cuts

Where there is room to accommodate them, temporary batters in sandy soil should be no steeper than 1 Vertical in 1.5 Horizontal (1V:1.5H) to maximum heights of 3m. Sandbagging along the toe of the batter will be required if they are located near the top of a rock cut. No surcharges should be placed within a horizontal distance from the crest equal to the height of the batter. Permanent batters are not expected but should not be steeper than 1V:2H but preferably flatter for ease of maintenance and planted with deep rooting plants and runner grass like vegetation to reduce erosion. If temporary or permanent batters of higher than 3m are proposed additional geotechnical advice should be obtained.

Rock of up to very low strength should be temporarily battered at 1V:1H but this will need to be supported in the long term by a suitably designed retaining wall or reinforced shotcrete with appropriate lateral support. Sandstone of low strength or better can be cut vertically, subject to inspection of the cut faces as recommended below.

Vertically cut rock must be progressively inspected by a geotechnical engineer at no more than 1.5m depth intervals to check for any adversely inclined joints or weak seams that require additional support. Any defects requiring stabilisation measures, such as rock bolts, shotcrete and mesh or dental treatment of thin seams, must be stabilised prior to further excavation. Given the strike (orientation in plan) of the vertical joints in the sandstone already identified on site (Hazard B and the flake behind) an allowance should be made for temporary rock bolting (assuming there is room to achieve this without encroaching on the boundary). Since the use of permanent rock bolts extending into adjoining properties is not preferred, the eastern walls of the house should be designed as retaining walls to support the potential rock blocks. If the orientation of jointing elsewhere is not adverse to stability then the walls may be free standing.

Where demolition of existing lower ground floor walls exposes previous sandstone rock faces, these should also be inspected by a geotechnical engineer to assess if any potentially unstable areas are present.

Toe drainage at the base of all rock cuts should be provided to channel away any seepage that may occur and should be linked to the stormwater system.

Unless fully supported by a retaining structure, any weak seams should be grubbed out and dry packed with no-shrink grout, or shotcreted to prevent spalling in the long term and potential clogging up of the toe drainage. Drainage such as weepholes should be installed in the seams to prevent build-up of hydrostatic pressures. Exposed sandstone faces will also deteriorate and fret in the long term and allowance must be made for maintenance in the long term to clear any debris from the drains. Alternatively, all sandstone cut faces could be covered with shotcrete to reduce such maintenance.

5.7 Retaining Walls

New retaining walls should be provisionally designed using the following parameters, but further advice and clarification should be sought once design options are selected:

- For cantilever walls, adopt a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a , of 0.3, for the retained height, assuming a horizontal backfill surface.
- Where walls are limited from movement, such as those propped by other structures in front of the wall, or where movement are to be reduced, an 'at rest' earth pressure coefficient, K_0 , of 0.6 should be used, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m³ should be adopted for the soil profile.
- Any surcharge affecting the walls (e.g. traffic loading, live loading, compaction stresses, etc) should be allowed in the design.
- The retaining walls should be provided with complete and permanent drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion.
- Retaining wall should be founded on sandstone bedrock.
- The design of retaining walls to support potentially unstable sandstone floaters must take into account the specific dimensions and orientation of the boulders so that the appropriate support can be provided.

5.8 Footings

All proposed footings must be founded in sandstone bedrock. The footings should be designed for an allowable bearing pressure of 600kPa, subject to inspection by a geotechnical engineer prior to pouring of concrete. Based on the expected shallow depth to inferred bedrock, pad and strip footings will be appropriate for most locations.

At least the initial footings at a representative spread of locations across the site should be inspected by a geotechnical engineer. Footings must be dry and free of loose material prior to pouring concrete.

In addition, any footings above a line of 1V:1H drawn up from the toe of an excavation/cut rock face must be specifically inspected by a geotechnical engineer for the presence of adverse defects within its zone of influence. Ideally, footings will be set back at least 0.1m from the crest of a cut.

5.9 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:

- Further geotechnical investigation comprising cored boreholes, if desired for tendering and estimating of support measures.
- Inspection of test pits to confirm founding conditions of existing structures affected by proposed demolition/excavation in accordance with the demolition methodology/plan prepared by the structural engineer.
- Progressive inspection of existing and proposed rock faces.
- Inspection of seepage/drainage.
- Inspection of footing excavations.

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.

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.

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

**TABLE A: SUMMARY OF RISK ASSESSMENT TO PROPERTY**

POTENTIAL LANDSLIDE HAZARD	A	B	C	D	E	F
	Failure or Rock Outcrops Throughout the Property	Failure of the Potentially Detached Boulder Located on the Sandstone Outcrop on the Eastern Boundary	Failure of the Stone Retaining Wall Supporting Bower Street	Failure of the Various Retaining Walls in Garden Areas within the Southern Portion of the Site	Failure of Tilting Retaining Wall Supporting Neighbouring Garage Along the Eastern Boundary	Failure of Rock Faces and Engineer Designed Retaining Walls supporting the Excavations for the Extended Lower Ground Floor and Ground Floor Levels
Assessed Likelihood	Unlikely	Rare	Possible	Likely	Likely	Rare
Assessed Consequence	Minor	Insignificant	Insignificant	Insignificant	Insignificant	Minor
Risk	Low	Very Low	Low	Low	Low	Very Low
Comments	Assumes inspected by geotechnical engineer during construction and any defects identified are stabilised	Assumes that engineer designed retaining wall is construction in front of boulder to provide support	Assumes fig tree is monitored and not allowed to compromise integrity of wall, otherwise risk would be Moderate	Failure would only affect limited surrounds and generally be readily repaired	Assumes failure would not cause too much damage to house wall of No 88, otherwise consequence could be minor and risk Moderate . If excavation is proposed near this wall additional engineer designed stabilisation works must be carried out.	Assumes rock faces progressively inspected by geotechnical engineer during excavation and any stabilisation treatment installed. Assumes all new walls and the wall supporting the house of No 86 are engineer designed and well constructed.

Assumed property price: \$ 4,000,000 (estimate from comparison with neighbouring developed properties as per onthefhouse.com.au 22 December 2020)

TABLE B: SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A	B	C	D	E	F
Assessed Likelihood	Unlikely	Rare	Possible	Likely	Likely	Rare
Indicative Annual Probability	10 ⁻⁴	10 ⁻⁵	10 ⁻³	10 ⁻²	10 ⁻²	10 ⁻⁵
Persons at risk	Person gardening	Person on patio garden near boulder	Person walking above or below wall	Person gardening	Person parking car in garage	Person in southern end of proposed lower ground floor or ground floor levels
Duration of Use of Area Affected (Temporal Annual Probability)	10mins/week 9.9 x 10 ⁻⁴	2hrs/week 1.2 x 10 ⁻²	20 secs/day 2.3 x 10 ⁻⁴	30mins/week 3.0 x 10 ⁻³	1min/day 6.9 x 10 ⁻⁴	8hrs /day 0.33
Probability of Not Evacuating Area Affected	0.5: Space to step back	0.8: Space to step back but little to no advanced warning signs	0.1: Early signs of instability likely to be noticed, and space to step back	0.5: Space to step back	0.9: Driver unlikely to notice any early signs of instability	0.5: Could be early signs of movement
Spatial Probability	0.1	0.5	0.2	0.1	1	0.5
Vulnerability to Life if Failure Occurs Whilst Person Present	0.5: Maybe crushed but unlikely to be killed due to limited height	0.8: Likely to be crushed and killed	Person above: 0.2: Unlikely to be killed	Person below: 0.8: Likely to be killed	0.2: Unlikely to be killed	0.1: Unlikely to be killed in car
Risk for Person Most at Risk	2.5 x 10 ⁻⁹	3.8 x 10 ⁻⁸	9.2 x10 ⁻¹⁰	3.7 x 10 ⁻⁹	3.0 x 10 ⁻⁷	6.2 x 10 ⁻⁷
Total Risk for Person Most at Risk	1.6 x 10⁻⁶					

BOREHOLE LOG



Borehole No.
2
1/1

Client: SQUILLACE ARCHITECTS Project: PROPOSED ALTERATIONS AND ADDITIONS Location: 88 BOWER STREET, MANLY, NSW													
Job No.: 33662BM Date: 3/12/2020 Plant Type:			Method: HAND AUGER Logged/Checked by: S.A./MP				R.L. Surface: ≈ 6.8m Datum: AHD						
Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB	DS									
DRY ON COMPLETION					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, trace of tree root fibres, sandstone gravel and tile fragments.	M			MULCH COVER APPEARS POORLY COMPACTED
						0.5							
						1							
						1.5			END OF BOREHOLE AT 1.15m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						2							
						2.5							
						3							
						3.5							

BOREHOLE LOG



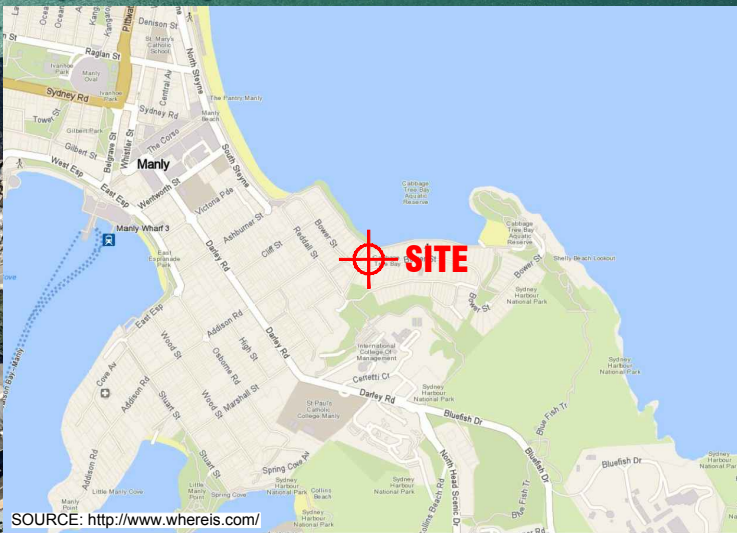
Borehole No.
5
1/1

Client: SQUILLACE ARCHITECTS Project: PROPOSED ALTERATIONS AND ADDITIONS Location: 88 BOWER STREET, MANLY, NSW												
Job No.: 33662BM Date: 3/12/2020 Plant Type:			Method: HAND AUGER Logged/Checked by: S.A./MP				R.L. Surface: ≈ 3.5m Datum: AHD					
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	US	DB									
DRY ON COMPLETION				REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, trace of terracotta fragments, root fibres, sandstone gravel and slag.	D			GRASS COVER APPEARS POORLY COMPACTED
					0.5			END OF BOREHOLE AT 0.35m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
					1							
					1.5							
					2							
					2.5							
					3							
					3.5							



DYNAMIC CONE PENETRATION TEST RESULTS

Client:	SQUILLACE ARCHITECTS PTY LTD						
Project:	PROPOSED ALTERATIONS AND ADDITIONS						
Location:	88 BOWER STREET, MANLY, NSW						
Job No.	33662BM			Hammer Weight & Drop: 9kg/510mm			
Date:	3-12-20			Rod Diameter: 16mm			
Tested By:	SA			Point Diameter: 20mm			
Test Location	1	2	3	4	5		
Surface RL	≈ 7.5m	≈ 6.8m	≈ 6.8m	≈ 7.1m	≈ 3.5m		
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	2	SUNK	SUNK	SUNK	2		
100 - 200	1	1	↓	3	5		
200 - 300	2	3	3	3/5mm	7		
300 - 400	2	3	4	REFUSAL	17		
400 - 500	2	3	5		REFUSAL		
500 - 600	1	3	4				
600 - 700	10/95mm	2	4				
700 - 800	REFUSAL	2	5				
800 - 900		2	4				
900 - 1000		2	8				
1000 - 1100		3	4/50mm				
1100 - 1200		6/70mm	REFUSAL				
1200 - 1300		REFUSAL					
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

Title:

SITE LOCATION PLAN

Location:

88 BOWER STREET, MANLY, NSW

Report No:

33662BM

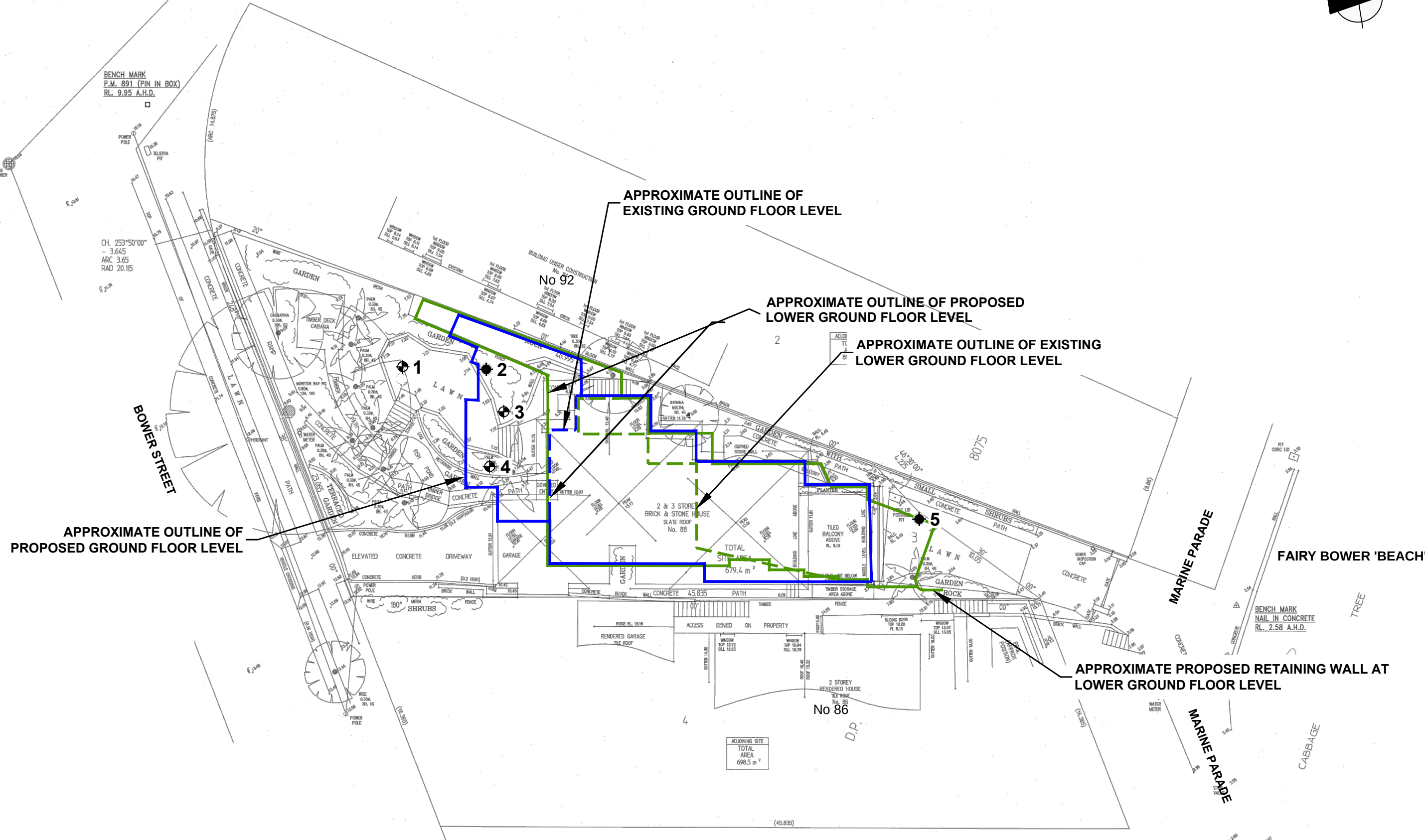
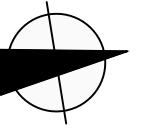
Figure No:

1

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

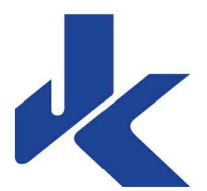
JKGeotechnics

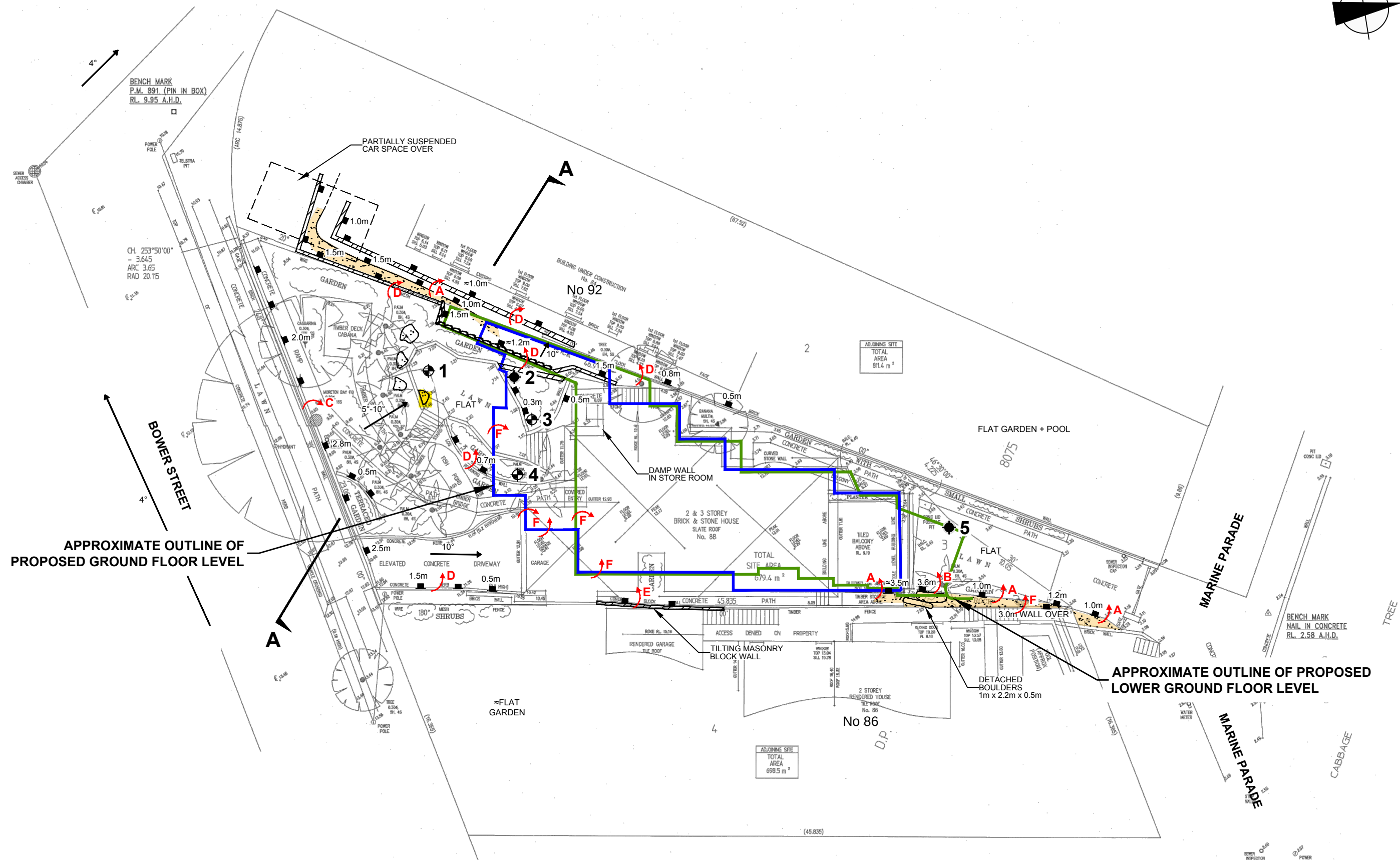
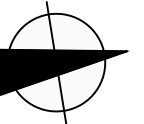




- LEGEND**
- BOREHOLE AND DCP TEST
 - ⊙ DCP TEST

0 2.5 5 7.5 10 12.5 SCALE 1:250 @A3 METRES	
This plan should be read in conjunction with the JK Geotechnics report.	
Title: BOREHOLE AND DCP TEST LOCATION PLAN	
Location: 88 BOWER STREET, MANLY, NSW	
Report No: 33662BM	Figure No: 2
JKGeotechnics	





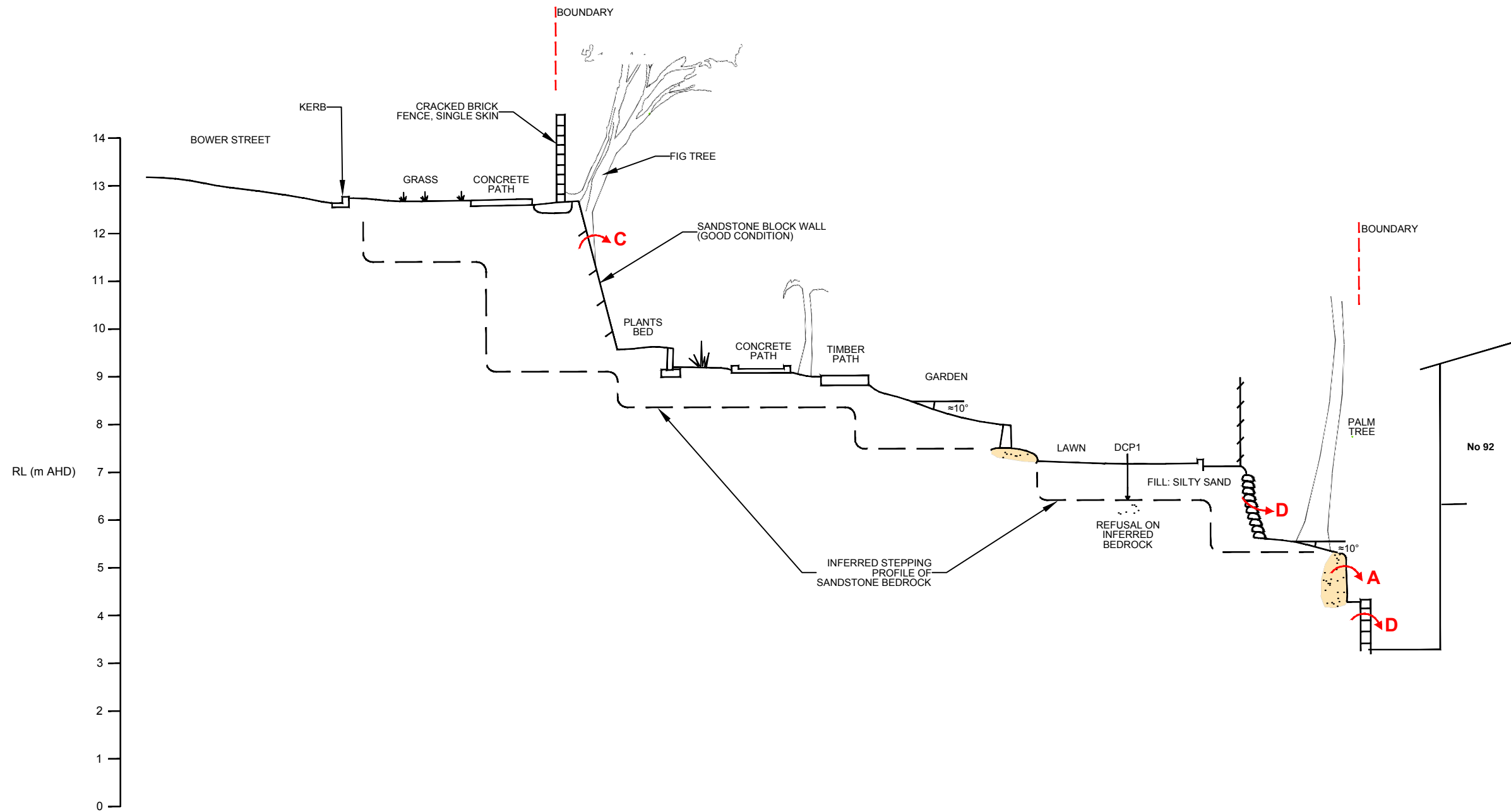
- LEGEND**
- BOREHOLE AND DCP TEST
 - DCP TEST
 - ➔ GEOTECHNICAL HAZARD
 - ROCK OUTCROP + HEIGHT (m)

GEOTECHNICAL MAPPING AND HAZARDS	
Location: 88 BOWER STREET, MANLY, NSW	
Report No: 33662BM	Figure No: 3
JK Geotechnics	

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

PLOT DATE: 22/12/2020 3:38:43 PM DWG FILE: Y:\33000\33662BM MANLY\CAD\33662BM.DWG

PLOT DATE: 14/12/2020 11:55:54 AM DWG FILE: Y:\33000\33662BM MANLY\CAD\33662BM.DWG



LEGEND

- GEOTECHNICAL HAZARD
- ROCK OUTCROP

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

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

Title: SECTION A-A SHOWING GEOTECHNICAL HAZARDS	
Location: 88 BOWER STREET, MANLY, NSW	
Report No: 33662BM	Figure No: 4
JKGeotechnics	



TOPOGRAPHY

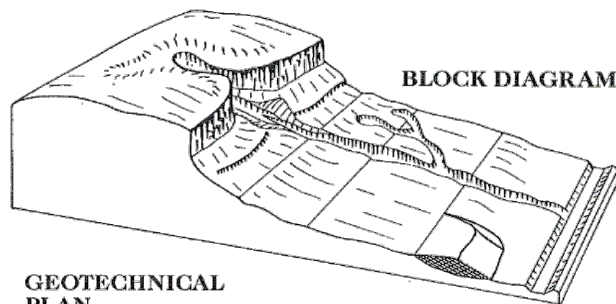
Symbol Ground Profile

		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		
		Cliff or escarpment or sharp break 40° or more (estimated height in metres)		
		Uniform Slope	}	Slope direction and angle (Degrees)
		Concave Slope		
		Convex Slope		
		Top	}	Cut or fill slope, arrows pointing down slope
		Bottom		
		Hummocky or irregular ground		

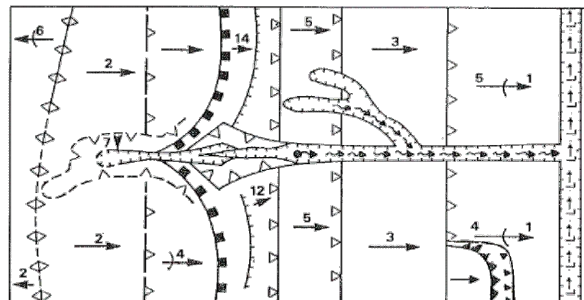
OTHER FEATURES

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

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

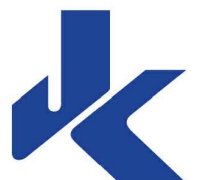


GEOTECHNICAL PLAN



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

Title:	GEOTECHNICAL MAPPING SYMBOLS	
Location:	88 BOWER STREET, MANLY, NSW	
Report No:	33662BM	Figure No: 5
JKGeotechnics		



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

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

N = 13
4, 6, 7

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

N > 30
15, 30/40mm

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

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

Cone Penetrometer Testing (CPT) and Interpretation:

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_0).

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would

be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

- i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
- ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or
- iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL



FILL



TOPSOIL



CLAY (CL, CI, CH)



SILT (ML, MH)



SAND (SP, SW)



GRAVEL (GP, GW)



SANDY CLAY (CL, CI, CH)



SILTY CLAY (CL, CI, CH)



CLAYEY SAND (SC)



SILTY SAND (SM)



GRAVELLY CLAY (CL, CI, CH)



CLAYEY GRAVEL (GC)



SANDY SILT (ML, MH)



PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK



CONGLOMERATE



SANDSTONE



SHALE/MUDSTONE



SILTSTONE



CLAYSTONE



COAL



LAMINITE



LIMESTONE



PHYLLITE, SCHIST



TUFF



GRANITE, GABBRO



DOLERITE, DIORITE



BASALT, ANDESITE



QUARTZITE

OTHER MATERIALS



BRICKS OR PAVERS



CONCRETE



ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 60% 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	

Laboratory Classification Criteria

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

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

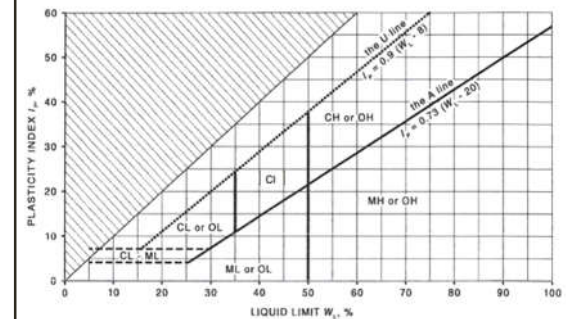
Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

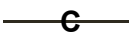
- For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
- Where the grading is determined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

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

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



LOG SYMBOLS

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



Log Column	Symbol	Definition
Remarks	'V' bit 'TC' bit T_{60} Soil Origin	Hardened steel 'V' shaped bit. Twin pronged tungsten carbide bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers. 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
	– Roughness	Ir	Irregular
		Vr	Very rough
		R	Rough
		S	Smooth
	– Infill Material	Po	Polished
		Sl	Slickensided
		Ca	Calcite
		Cb	Carbonaceous
	– Coatings	Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
		Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
	– Thickness	Filled	Coating > 1mm thick
		mm.t	Defect thickness measured in millimetres



APPENDIX A

**LANDSLIDE RISK
MANAGEMENT
TERMINOLOGY**

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

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

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

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

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

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

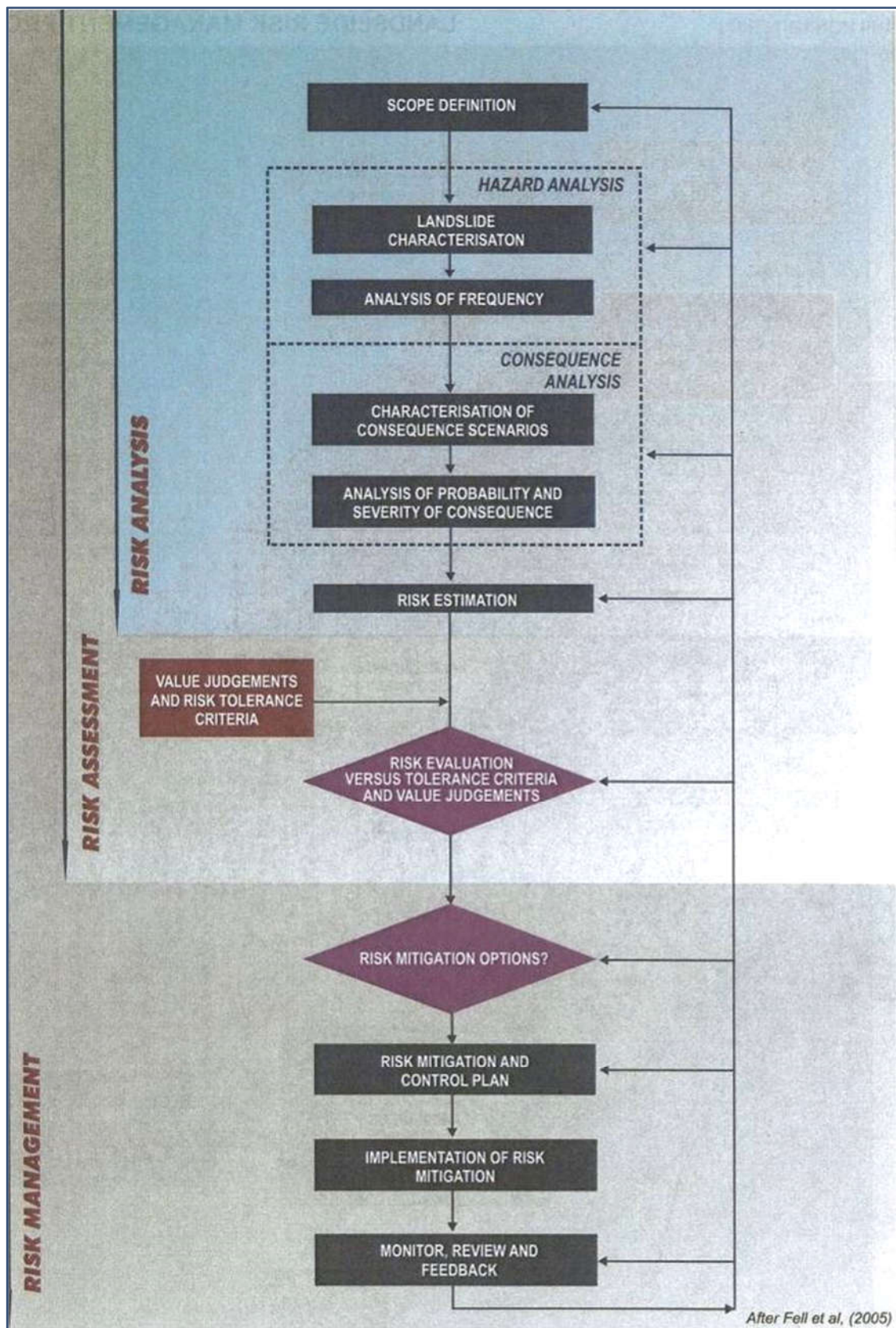


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

RISK LEVEL IMPLICATIONS

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

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

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

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

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

What Causes a Landslide?

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

Does a Landslide Affect You?

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

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

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

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

TABLE 1 – Slope Descriptions

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

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

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

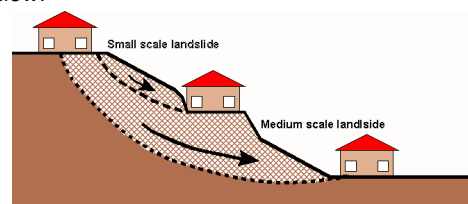


Figure 1

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

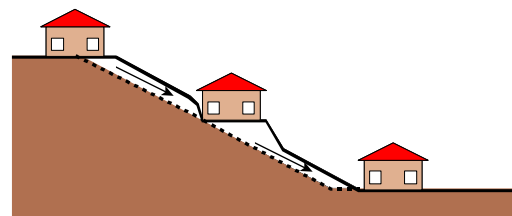


Figure 2

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

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

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

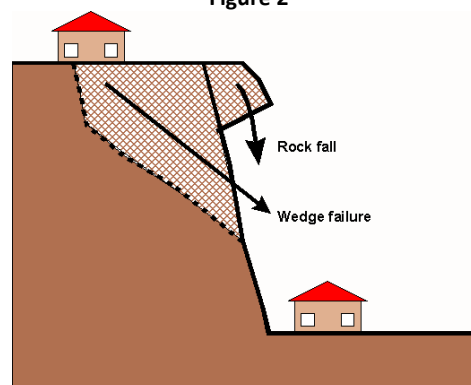


Figure 3

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

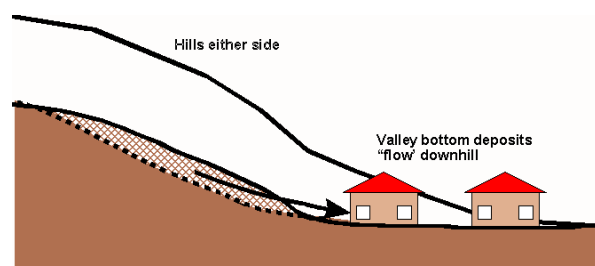


Figure 4

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

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

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

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

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

Landslide Risk Assessment

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

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

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

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

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

Risk to Property

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

TABLE 2 – LIKELIHOOD

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

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

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

TABLE 1 – RISK TO PROPERTY

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

Risk to Life

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

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

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

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

TABLE 3 – RISK TO LIFE

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

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

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- GeoGuide LR4 - Rock Slopes
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