REPORT TO

MR & MRS S ROONEY

ON

GEOTECHNICAL ASSESSMENT
(In Accordance with Pittwater Council Risk Management Policy)

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

205 RIVERVIEW ROAD, AVALON BEACH, NSW

Date: 11 September 2019
Ref: 27796Rrpt2

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of our geotechnical assessment of the site at 205 Riverview Road, Avalon Beach, NSW. The location of the site is shown in Figure 1. We note that we prepared a previous report (Ref. 27796ZRrpt Rev2) dated 21 June 2016. However, the proposed development has been amended and so an updated report was requested. This revised assessment report was commissioned on behalf of Mr and Mrs S Rooney by Grant Mills (Mark Hurcum Design Practice Pty Ltd (MHDP) in a letter dated 30 May 2019. The commission was on the basis of our proposal (Ref. P49500R rev1) dated 16 May 2019. The site was inspected by our engineering geologist (Paul Roberts) on 7 October 2014, in order to assess the existing stability of the site and the effect on stability of the proposed development.

Details of the proposed development are presented in Section 5 below. In summary, however, it is proposed to it is proposed to demolish portions of the existing buildings and structures and reconfigure the existing house and include an additional level that connects to the existing car port. To achieve design subgrade levels, excavations to maximum depths between about 0.9m and 1.4m back into the hillside will locally be required over the proposed pool level, which will include an 'outdoor room', bathroom and storage room and a new pool and spa.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 6 below. It is understood that the report will be submitted to Council as part of the DA documentation. Our report is preceded by the completed Council Forms 1 and 1a.

2 ASSESSMENT METHODOLOGY

2.1 Walkover Survey

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

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 7 following our geotechnical assessment.

The attached Figure 2 presents a geotechnical sketch plan showing the principal geotechnical features present at the site with the proposed pool level and lower ground floor level superimposed. Figure 2 is based on the survey plan prepared by prepared by True North Surveys (Drawing Number 7204DU dated 23 July 2013). Additional features on Figure 2 have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 3 presents a typical cross-section through the site based on the survey data, the provided architectural section A-A augmented by our mapping observations. Figure 4 presents an explanation of the geotechnical mapping symbols used on Figure 2.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figure 2. Based on information provided by MHDP, since our previous site visit in 2014, we understand the following:

- There has been no re-development work at the site.
- The has been no Development Application (DA) submitted for No. 207 Riverview Road since 2003.
- For the neighbouring No. 203 Riverview Road, the DA for alterations and additions included reconfiguration of the upper two levels. Our review of the provided DA consent architectural plans indicated that over the central portion of the rear of the lower level new retaining walls were indicated, which appeared to require localised excavations to maximum 2.0m depth back into the hillside within the neighbouring site.

On the basis of the above, it was considered that there have been no substantive changes to the site or the neighbouring properties such that a new site inspection was required and the site observations which follow are essentially based on our site inspection in 2014.

- The site is located on a hillside that slopes and steps down to the west towards the eastern Pittwater foreshore.
- The site is approximately rectangular in plan and has an eastern frontage onto Riverview Road. The site is a maximum of about 52m long (east-west) and a maximum of about 16m wide (north-south).
- Riverview Road was paved with asphaltic concrete (AC) and the eastern side of the road was lined with a concrete gutter. The concrete gutter was in good condition and the AC surface was generally in reasonable condition.
- The southern half of the street frontage was lined by concrete block wall (2m high) and a brick car port with a suspended concrete floor slab accessed by a gently sloping concrete paved driveway. The northern half of the street frontage was lined by a planter bed and a sandstone masonry retaining wall (about 2m high). The western side of the planter bed was supported by a concrete block wall (maximum 2.6m high). Sandstone paved steps were supported by a brick wall (maximum 2m high) which extended down from the street frontage to the sandstone paved front yard area. The street frontage structures were in reasonably good condition with occasional hairline to maximum 5mm wide cracking evident. However, the sandstone masonry wall supporting the northern end of the street frontage contained hairline to 20mm wide cracking.
- The eastern side of the paved front yard was lined by stepped planter beds supported by brick and sandstone masonry retaining walls (maximum height about 1.3m). The western side of the front yard was lined by the eastern section of the existing two and three storey house. Timber walkways and sandstone paved steps lined the northern and southern sides of the house. The house appeared to be

in good condition and the front yard structures appeared to be in reasonably good condition with occasional hairline to 5mm wide cracking evident.

- From the south-western corner of the house an inclinator extended down the steep rear yard to the foreshore. The inclinator was supported by steel and concrete columns. The upper section of the slope surface below the inclinator had a stepped profile and the steps were formed by timber retaining walls about 1m high.
- The western end of the storage level of the existing house was supported by a stacked sandstone retaining wall (about 2m high) which extended south into the neighbouring site. The sandstone masonry wall appeared to be in reasonably good condition but observations were restricted by vegetation and a small toilet shed.
- The stepped rear yard area lining the western side of the house was stepped and contained planter beds and sandstone paved areas. The steps were supported by sandstone masonry and concrete block retaining walls ranging between about 0.5m and 1.5m height and which appeared to be in reasonably good condition.
- A sandstone paved and timber stepped walkway 'zig-zagged' down the steep western half of the rear yard. A sandstone outcrop face maximum height about 2.5m and orientated approximately north-south, formed the upper portion of the steeper western half of the rear yard. The first landing of the stepped walkway was located at the northern end of the outcrop and was supported by a sandstone masonry retaining wall (maximum height about 1.5m). The outcrop contained an overhang feature about 1.8m high and which extended back a maximum 'depth' of about 1.8m.
- The remaining vegetated steep rear yard below the outcrop sloped down to the west at between about 45^{o} and 70^{o}. Several trees were scattered across the uneven slope surface, some of which were leaning over from vertical. Numerous traces of dilapidated low height timber landscape walls were also scatted across the slope surface. The slope surface appeared to comprise colluvial clayey soils with detached sandstone blocks and occasional possible sandstone bedrock outcrop faces (maximum height about 1m) evident. Traces of poor condition sandstone masonry walls (maximum height about 0.5m) were also scattered across the slope.
- The base of the steeply sloping rear yard was supported by sandstone masonry and concrete panel retaining walls (ranging between about 0.5m and 2m height) which appeared to be in good condition. A concrete paved foreshore area and a timber boatshed lined the toes of the retaining walls. The western side of the foreshore area was supported by a sandstone masonry seawall (maximum height approximately 1m) which extended north and south beyond the site boundaries. A detached sandstone block (about 0.5m high and 2m long) was located over the southern end of the paved foreshore area. A timber jetty and boat ramp extended west from the paved foreshore area and boat shed, respectively.
- The upslope (eastern) portion of the southern site boundary was lined by a portion of the one and two storey rendered and timber clad house. Immediately to the west, the subject site surface levels were 2.5m above the neighbouring surface levels and were lined by what has been assumed to by a brick retaining wall and the suspended timber walkway lining the southern side of the existing house within the subject site. The remainder of the southern site boundary was lined by the stepped vegetated and landscaped neighbouring yard area with timber and sandstone paved stepped walkways evident. Sandstone masonry walls (maximum height about 2m) formed the stepped yard profile. The

sandstone masonry wall immediately to the west of the neighbouring house extended north into the subject site and terminated at the southern wall of the house. Site surface levels were generally similar across the remainder of the southern site boundary.

- The neighbouring site to the north was occupied by a one to three storey steel framed and steel clad house set-back at least about 1.5m from the northern site boundary. A similar construction neighbouring car port lined the eastern end of the northern site boundary. The front and rear yard areas were sandstone paved with the steps formed by sandstone masonry walls ranging between about 2m and 3.5m height. To the south-west of the neighbouring house a detached sandstone block (at least approximately 1.5m x 1.5m lined the adjacent section of the northern site boundary. The upper portion of the rear yard contained a sandstone outcrop face orientated approximately north-south and about 2m high. The remainder of the rear yard steeply sloped down to the west at between about 60^{o} and 70^{o} to the sandstone paved foreshore area. The base of the slope was supported by sandstone masonry walls between about 1m and 1.5m high. Site surface levels were generally similar over the northern site boundary.
- Based on a cursory inspection from within the site, unless otherwise noted above, the houses and structures within and neighbouring the site appeared to be generally in good external condition.

4 SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone close to the contact with the underlying Newport Formation of the Narrabeen Group, which generally comprises interbedded sandstone and shale. The Hawkesbury Sandstone forms a cap to the hillsides in the area. The outcrop faces within the site and the neighbouring site to the north are interpreted to represent the Hawkesbury Sandstone. Based on our experience in this area we expect that the Newport Formation to be present over the lower portion of the site and the interface between the Hawkesbury Sandstone and the underlying Newport Formation to be at or just above foreshore level.

Based on our experience in this area, we also expect the subsurface conditions across the site to comprise a stepped bedrock surface with a cover of colluvial clayey soils with numerous inclusions of sandstone blocks ranging up to boulder size; the colluvial soils are derived from previous slope instability that has occurred in the recent geological past. We note that some clayey soils were noted across the site which we have interpreted to represent colluvial soils. The detached sandstone blocks described over the rear yard slope and within the neighbouring rear yards to the north and south have been interpreted to represent sandstone blocks within a colluvial soil profile.

The outcrops within the site were trending approximately north-south and extended north into the neighbouring site. The outcrop faces within and neighbouring the site were partially formed by sub-vertical joints trending approximately 340^{o} to 020^{o} with occasional sub-vertical joints trending approximately 100^{o} to 110^{o} also noted.

5 PROPOSED DEVELOPMENT20 August 2019

We understand from the provided architectural drawings (Job No. 1824, Drawing Numbers A010A/P3, A011A/P2, A012A/P3 to A014A/P3, A100A/P7 to A104A/P7, A301A/P6 and A302A /P6, dated 20 August 2019 and A201A to A204A, dated September 2019) prepared by MHDP, that the proposed development will comprise the following:

- Reconfiguration of the existing house, including an additional First Floor Level extending over the existing house that will connect to the existing car port lining the street frontage to the east. The existing Lower Ground and Ground Floor Levels will be extended out to the west.
- The proposed stepped Pool Level (proposed finished floor reduced levels [RLs]) will be formed at between RL21.9m, RL21.2m and RL20.5m. The Pool Level excavations will extend east back into the hillside below the existing house to form an 'outdoor room', bathroom and storage room. To the west (downslope) a new pool and spa will be formed. To achieve design subgrade levels, excavations back into the hillside will locally be required over the proposed Pool Level to maximum depths between about 0.9m ('outdoor room', bathroom and storage room) and 1.4m (the pool).
- Construction of a new stepped walkway down to the Pool Level from the northern side of the house and landscaping of the yard areas.

The footprint of the proposed Pool Level and Lower Ground Floor Level are indicated on Figure 2.

6 GEOTECHNICAL ASSESSMENT

The subject site is characterised by a steeply sloping hillside and appeared to be relatively well drained.

Whilst there are no obvious signs of deep seated slope instability, the inferred colluvial soils located across the site and inferred to extend north, south and east beyond the site boundaries appear to be affected by downslope creep. Downslope creep was indicated by the uneven surface of the steep slopes and occasional leaning trees. In addition, under particularly adverse conditions, the colluvial soils also have the potential to be subject to deeper seated instability.

A sandstone bedrock outcrop face was noted over the upper portion of the rear yard and extended into the rear yard to the north. Within the site the outcrop face contained an overhang feature. A number of detached sandstone blocks ('floaters') were identified over the steep rear yard, at foreshore level and within the neighbouring sites to the north and south.

Bedrock is generally expected at shallow depth across the site and locally deeper where fill may have been locally placed (and supported by retaining walls, in particular the retaining wall below the western side of the house. Bedrock may also be encountered at greater depth where colluvial soils are inferred to be present, particularly over the steeper section of the rear yard below the outcrop and/or where 'floaters' are inferred to be present. Such deposits are expected at a site such as this, which is underlain by bedrock at the inferred interface between the more competent Hawkesbury Sandstone and the weaker underlying Newport Formation. Over time the weaker Newport Formation degrades (due to weathering), erodes and landslides occur. Undercuts form at the base of the more competent Hawkesbury sandstone eventually leading to toppling and collapse of large sandstone blocks ('floaters').

Stress relief effects due to erosion of the landscape to form the current hillsides leads to opening of the defects within the Hawkesbury sandstone and sliding along sub-horizontal defects. In addition, erosion of relatively weak features (extremely weathered seams, bedding partings etc) within the Hawkesbury Sandstone can lead to the formation of overhangs such as those present at the site. Such undercutting of the more competent sandstone above, followed by toppling and/or basal shear would also result in the material collapsing down the slopes and cliff faces to the foreshore. 'Floaters' present along the foreshore and the slope surfaces within and adjacent to the site would also be derived from collapse of such overhangs.

6.1 Potential Landslide Hazards

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

A Instability of existing retaining walls:
- (i) Within the house, car port and storage area;
- (ii) Stacked sandstone, sandstone masonry and concrete panel walls across the site;
- (iii) Variable condition, timber and stacked sandstone landscape walls over the yard areas; and
- (iv) Sandstone masonry seawalls.

B Instability of the natural hillside slopes:
- (i) Downslope (west) of the existing dwelling;
- (ii) Upslope (east) of the existing dwelling;
- (iii) North of the existing dwelling;
- (iv) South of the existing dwelling; and
- (v) Collapse of overhang.

C Instability of temporary excavation batters.

D Instability of proposed retaining walls and pool walls:

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

6.2 Risk Analysis

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A indicates that the assessed risk to property varies between 'Moderate' and 'Low' or 'Very Low' under existing conditions, which would be considered 'Tolerable' and 'Acceptable', respectively in accordance with the criteria given in Reference 1 and the Pittwater Council Risk

Management Policy. Following implementation of the recommendations outlined in Section 7, below risk levels would reduce to 'Acceptable' levels within the site.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life during and after completion of the proposed development and implementation of the recommendations outlined in Section 7, below. The temporal, spatial, evacuation and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is less than about 10^{-6}. This would be considered to be 'Acceptable' in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

Based on a review of the Pittwater Council Draft Bush Fire Prone Land Map, dated 20 January 2012 provided on Councils website, we note that the subject site is not designated as bush fire prone. On this basis, we have assessed that there will be negligible impact on slope stability as a result of any potential bushfire under existing conditions. Furthermore, we have assumed that the new residence will be designed and constructed in accordance with best practice for bushfire management. Consequently, we have assessed the levels of risk to life and property due to landslides associated with bushfire to be at 'Acceptable' levels in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy. However, if the site or neighbouring sites along, up or downslope of the site are affected by bushfire then as soon as is practicable after the bushfire, an experienced geotechnical engineer or engineering geologist should inspect the subject site and the adjoining slopes to assess site stability and confirm the scope and extent of any stabilisation measures.

We note that an assessment of risk to life and property due to bushfire is outside our area of expertise and in accordance with Council guidelines, a Bushfire Consultant should complete such an assessment, if required.

6.3 Risk Assessment

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

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

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.
In preparing our recommendations given below we have adopted the above interpretations of the Risk Management Policy requirements. We have also assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all Council's buried services are, and will be regularly maintained to remain, in good condition.

We consider that our risk analysis has shown that the site and existing and proposed development can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy provided that the recommendations given in Section 7 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

7 COMMENTS AND RECOMMENDATIONS

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

7.1 Conditions Recommended to Establish the Design Parameters

7.1.1 The existing retaining walls within the rear yard above the overhang that will remain may be surcharged by the proposed balcony. Vegetation will need to be cleared from around any of the walls that will remain and geotechnical investigation including test pits exposing the wall footings and completion of boreholes and Dynamic Cone Penetration (DCP) testing behind the wall. The test pits will need to be inspected by the geotechnical and structural engineers and the need for wall strengthening or replacing can then be determined. Retaining wall design parameters are provided in Section 7.1.8, below.

7.1.2 The existing rock face contains an overhang that may well require localised stabilisation measures such as mass concrete or masonry underpins and/or permanent rock bolts. The overgrown area around the top surface of the overhang will need to be cleared and any remaining soil carefully removed using hand held tools to expose the bedrock surface. The exposed bedrock will then need to be inspected by a geotechnical engineer to determine if there are open continuous steeply sloping joints that may detrimentally impact the stability of the overhang. Based on the results of the geotechnical inspection, the need and extent of stabilisation measures for the overhang can then be determined and detailed.

7.1.3 The excavations over the eastern side of the Pool Level have the potential to undermine existing house walls. During demolition the wall footings that will remain and potentially undermined will need to be exposed (including the foundation material). The test pits will need to be inspected by the geotechnical and structural engineers to assess the need and extent of underpinning. A similar exercise will need to be completed for all existing footings that will be supporting additional loads. All footings not founded in bedrock will need to be underpinned down to bedrock.

7.1.4 All proposed footings, existing footings supporting additional loads and the new pool and spa must be founded/underpinned in sandstone or possibly shale bedrock. The footings/underpins should be designed for an allowable bearing pressure of 700kPa, subject to inspection by a geotechnical engineer prior to pouring. Footings/underpins close to the crests of natural or cut rock faces should be designed for an allowable bearing pressure of 400kPa on condition that the bedrock faces below the footings/underpins are inspected by a geotechnical engineer to check for the presence of potentially unstable wedges or seams etc which may require support using rock bolts, reinforced shotcrete etc.

7.1.5 Subject to inspection by a geotechnical engineer temporary batters for proposed excavations should be no steeper than 1 Vertical (V) in 1 Horizontal (H) within the clayey soil profile, extremely weathered and/or fractured sandstone or shale bedrock, subject to geotechnical inspection. Temporary vertical cuts in competent sandstone or possibly shale bedrock are expected to be feasible, subject to geotechnical inspection. Sandstone bedrock is expected to be encountered although there is the possibility that interbedded shale and sandstone may be encountered which may be of variable quality and may also need to be battered back. In addition, the western side of the Pool Level excavations may encounter 'floaters' which, where left in place, may also require permanent stabilisation using rock bolts, underpins etc, subject to geotechnical inspection. All surcharge and footing/underpin loads must be kept well clear of the excavation perimeter. Competent sandstone is expected to stand permanently unsupported subject to the geotechnical inspections outlined above. However, competent shale bedrock is expected to deteriorate over time and should therefore be permanently supported. Any bands of extremely weathered and/or fractured bedrock within the more competent bedrock may require localised support using reinforced shotcrete supported by rock bolts.

7.1.6 The above excavation batters are expected to be accommodated within the site and conventional retaining walls then constructed from within the excavation prior to backfilling the walls. This assumes that any adjacent footings that may be undermined by the excavations are underpinned down to bedrock. Any underpins that will be supporting the soil profile will need to be designed in accordance with the advice provided in Section 7.1.8 below, to resist lateral loading.

7.1.7 The surface water discharging from the new and existing roof and paved areas must be diverted to outlets for controlled discharge to the existing stormwater system.

7.1.8 The proposed new retaining walls, pool walls, underpins supporting a soil profile, any existing walls that require strengthening and stabilisation of potentially unstable sections of bedrock and/or 'floaters' should be designed using the following parameters:

– For cantilever walls that will be propped by the proposed structure, pool walls and any underpins supporting a soil profile, adopt a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient (k_o) of 0.55 or 1.35 for the retained soils and extremely weathered bedrock profile, assuming a horizontal backfill surface or a backfill surface sloping at a maximum of 30^o, respectively.

– For cantilever walls where movement is of little concern (say landscape walls), adopt a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.35 or 0.85, for the retained height, assuming a horizontal backfill surface or a backfill surface sloping at a maximum of 30^o, respectively.

– A bulk unit weight of $20kN/m^3$ and $22kN/m^3$ should be adopted for the retained soil and weathered bedrock profile, respectively.

– Any surcharge affecting the walls (e.g. traffic loading, live loading, compaction stresses, etc) should be allowed in the design.

– The retaining walls and underpins supporting a soil profile should be provided with complete and permanent drainage of the ground behind the walls/underpins. The subsoil drains behind the walls should incorporate a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion. The underpins should be provided with PVC pipe weep holes installed at about 1.2m lateral centres just above the adjacent surface level or bedrock level and discharge into the perimeter drainage system. The embedded ends of the weep hole pipes should also be wrapped in a non-woven geotextile fabric (e.g. Bidim A34), to act as a filter against subsoil erosion.

– Toe resistance of the wall may be achieved by keying the footing into bedrock. An allowable lateral stress of 200kPa may be adopted for key design assuming horizontal ground in front of the toe. However, the presence of a step down in the bedrock in front of the key cannot be discounted and geotechnical inspections are recommended

. – Toe restraint for low height (say <1m) landscape walls founded in the soil profile may be provided by the passive pressure of the soil below bulk adjacent surface levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections that are associated with achieving a full passive case. Localised excavations in front of the walls e.g. for buried services etc must also be taken into account in the design. Where these footings are on bedrock, the lateral resistance may be calculated using a friction angle of 30^o for the footing/bedrock interface. We warn that downslope creep of the soil profile across the hillside may occur and that full height construction joints should be provided to accommodate any such potential movements that may affect walls founded in the soil profile.

– Rock bolts within the weathered bedrock of at least low strength should be designed for an allowable bond strength of 100kPa. Where appropriate, the bolt heads should be engaged with the reinforcement mesh and encapsulated in the shotcrete with sufficient cover to achieve corrosion protection.

7.1.9 Where rock bolts are to run below adjoining properties (believed to be unlikely), then the permission of the owners must be obtained before installation.

7.1.10 The Pool Level excavation subgrade is likely to comprise a mix of bedrock and natural soils and may encounter a step down in the bedrock profile. Whilst slab-on-grade construction is theoretically feasible for the floor slab, it will be difficult to complete high quality earthworks on a confined site such as this, as it would require removing and re-compacting the existing fill. Also, on-grade floor slabs would be in contact with a mix of existing sandy and clayey fill and natural sands and possibly a sub-vertical bedrock face. This mix of subgrade conditions would result in differential deflections, and potential cracking of the lower section of the re-profiled driveway slabs. Over soil subgrade areas our recommendation is to suspend floor slabs between footings founded in bedrock.

7.1.11 Over any soil subgrade areas beneath external paved areas, after completion of any bulk excavations to achieve design subgrade levels, the subgrade must be proof rolled. The proof rolling should be carried out using a small (say 2 tonne) smooth drum vibratory roller. However, due to access constraints a hand held vibrating plate compactor ("whacker packer") is likely to be used. During proof rolling, adjoining structures must be closely monitored by the site supervisor and if there are causes for concern then the static (no-vibration) mode should be used or work immediately stop and this office be contacted for further advice. The aim of the proof rolling is to identify any soft or unstable areas, which if detected should be excavated down to a sound base and backfilled with thoroughly compacted engineered fill.

7.1.12 Engineered fill must be free of organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted to achieve a Minimum Density Index (ID) of 70% (sandy fill materials) or at least 98% of Standard Maximum Dry Density (clayey fill materials and excavated bedrock). For any fill over landscaped areas, where movements are expected to be of little concern, the above compaction criteria may be reduced to an ID of 65% (sandy fill materials) or at least 95% of SMDD (clayey fill materials and excavated bedrock). Backfill to conventional retaining walls should also comprise engineered fill. Well graded granular materials such as ripped or crushed sandstone and demolition rubble would be suitable for this purpose. Compaction of backfill behind retaining walls is likely to require the use of "whacker packers". Confirmatory in-situ density testing should be undertaken to assess compaction. Any areas of insufficient compaction will require reworking.

7.1.13 The effluent system should be piped and discharged to the main sewer system.

7.1.14 The sections of the pool and spa that are not suspended over the slope should be provided with a one-way or non-return valve at their lowest points so as to prevent buoyancy in the event that external groundwater levels rise above the water level within the pool or spa, unless such groundwater could be fed into a gravity drain. The pool and spa backwash systems should be piped and discharged to the main sewer system.

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

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

7.2.1 The additional geotechnical inspections referred to in the preceding Sections 7.1.1, 7.1.2, and 7.1.3 are to be noted on the design drawings and appropriate contingency details provided on the design drawings.

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

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

7.2.4 Dilapidation surveys must be carried out on the neighbouring buildings and structures to the north and south. A copy of the dilapidation report must be provided to the neighbours and Council or the Principle Certifying Authority

7.3 Conditions Recommended During the Construction Period

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

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

7.3.3 The approved excavation/retention methodology must be followed.

7.3.4 The additional geotechnical inspections referred to in the preceding Sections 7.1.1, 7.1.2, and 7.1.3 are to be completed.

7.3.5 Periodic vibration monitoring must be carried out during rock excavations using rock hammer attachments to excavators. The ground vibration measured as peak particle velocity (PPV) must not exceed 5mm/sec at the northern, southern and eastern site boundaries. The PPV will need to be confirmed following review of the dilapidation survey reports recommended in Section 7.2.4, above.

7.3.6 Localised stabilisation measures to the rock cut faces and sandstone bedrock overhang in the rear yard (where required) must be witnessed by the geotechnical engineer.

7.3.7 The geotechnical engineer must inspect all footing and underpin excavations prior to placing reinforcement or pouring the concrete.

7.3.8 Bulk excavations must be progressively inspected by the geotechnical engineer as excavation proceeds. We recommend inspections at maximum 1.5m vertical depth intervals and on completion.

7.3.9 Proposed material to be used for backfilling behind retaining walls in critical areas (e.g. backfill that will be supporting paved areas etc) must be approved by the geotechnical engineer prior to placement. The engineered fill specification and compaction requirements should comply with the advice provided in Section 7.1.12, above.

7.3.10 Compaction density of the backfill material must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer.

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

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

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

7.3.14 The geotechnical engineer must confirm that the proposed alterations and additions have been completed in accordance with the geotechnical reports.

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

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

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

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

7.4.2 All existing rock faces and detached sandstone blocks remaining across the site must be inspected by an experienced engineer/engineering geologist at no more than ten yearly intervals; including provision of a written report confirming scope of work completed and identifying any required remedial measures.

7.4.3 Existing retaining walls that are to remain over the site must be inspected by a structural engineer at no more than ten yearly intervals; including the provision of a written report confirming scope of work completed and identifying any required remedial measures.

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

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

8 OVERVIEW

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

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

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

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

TABLE A

SUMMARY OF RISK ASSESSMENT TO PROPERTY

Potential Landslide Hazard	EXISTING CONDITIONS									DURING AND AFTER COMPLETION OF PROPOSED DEVELOPMENT AND IMPLEMENTATION OF RECOMMENDATIONS OUTLINED IN SECTION 7										
	A: Instability of existing retaining walls				B: Instability of the hillside slopes					A: Instability of existing retaining walls				B: Instability of the hillside slopes					C: Instability of Temporary Excavation Batters	D: Instability of Proposed Retaining Walls Supporting the Excavation Faces and pool walls
	(i) Within the house, car port and storage area	(ii) Stacked sandstone, sandstone masonry and concrete panel walls across the site	(iii) Variable condition, timber and stacked sandstone landscape walls over the yard areas	(iv) Sandstone masonry seawalls	(i) Downslope of the existing dwelling	(ii) Upslope of the existing dwelling	(iii) North of the existing dwelling	(iv) South of the existing dwelling	(iv) Collapse of overhang	(i) Within the house, car port and storage area	(ii) Stacked sandstone, sandstone masonry and concrete panel walls across the site	(iii) Variable condition, timber and stacked sandstone landscape walls over the yard areas	(iv) Sandstone masonry seawalls	(i) Downslope of the existing dwelling	(ii) Upslope of the existing dwelling	(iii) North of the existing dwelling	(iv) South of the existing dwelling	(v) Collapse of overhang		
Assessed Likelihood	Unlikely	Unlikely	Possible	Unlikely	Unlikely (Deep Seated) Almost Certain (Creep)	Unlikely (Deep Seated) Almost Certain (Creep)	Unlikely (Deep Seated) Almost Certain (Creep)		Possible	Rare	Rare	Rare	Rare	Unlikely	Unlikely	Unlikely		Unlikely	Unlikely	Rare
Assessed Consequences	Minor	Insignificant (yard areas) Minor (adjacent to the house or car port)	Insignificant	Minor	Insignificant	Insignificant (Creep) Minor (Deep Seated)	Insignificant		Minor	Minor	Insignificant (yard areas) Minor (adjacent to the house or car port)	Insignificant	Minor	Insignificant (yard areas) Medium (new pool)	Insignificant	Insignificant		Minor	Minor	Minor (retaining walls) Major (pool walls)
Risk	Low	Very Low (yard areas) Low (adjacent to the house or car port)	Very Low	Low	Low (Creep) Very Low (Deep Seated)	Low (Creep) Low (Deep Seated)	Low (Creep) Very Low (Deep Seated)		Moderate	Very Low	Very Low	Very Low	Very Low	Very Low (yard areas) Low (new pool)	Very Low (Creep) Low (Deep Seated)	Very Low		Low	Low	Very Low (retaining walls) Lowr (pool walls)
Comments	A (i) to (iv): Assumes localised instability. B (i) to B (iv); deep seated instability: assumes localised instability. A (i), (ii) and (iv): Assumes walls have been engineer designed.									A (iii): Assumes walls will be removed as part of works or assessed and repaired as appropriate. A (i), (ii) and (iv): Assumes walls have been engineer designed, assessed and repaired and/or removed as appropriate. B (i) to B (iv): Assumes remaining slopes will be re-profiled and/or stabilised as per the recommendations in this report. B (v): Assumes overhang assessed and stabilised as required. C: Assumes recommended batter slopes will be adopted and excavation batters inspected by geotechnical engineer D: Assumes the retaining walls and pool walls will be properly engineered.										

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

DURING AND AFTER COMPLETION OF PROPOSED DEVELOPMENT AND IMPLEMENTATION OF RECOMMENDATIONS OUTLINED IN SECTION 7

Potential Landslide Hazard	A: Instability of existing retaining walls				B: Instability of the hillside slopes					C: Instability of Temporary Excavation Batters	D: Instability of Proposed Retaining Walls Supporting the Excavation Faces and Pool Walls
	(i) Within the house, car port and storage area	(ii) Stacked sandstone, sandstone masonry and concrete panel walls across the site	(iii) Variable condition, timber and stacked sandstone landscape walls over the yard areas	(iv) Sandstone masonry seawalls	(i) Downslope of the existing dwelling	(ii) Upslope of the existing dwelling	(iii) North of the existing dwelling	(iv) South of the existing dwelling	(v) Collapse of overhang		
Assessed Likelihood	Rare	Rare	Rare	Rare	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Unlikely	Rare
Indicative Annual Probability	10^{-5}	10^{-5}	10^{-5}	10^{-5}	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-5}
Persons at Risk	Persons in house and yard areas within and neighbouring the site	Persons in yard areas			Persons in house, pool area, and yard areas within and neighbouring the site				Persons in yard areas	Persons at crest Workers within excavation	Persons in house and yard and pool areas
Number of Persons Considered	2	2			2				2	2	2
Duration of Use of Area Affected (Temporal Probability)	1hr/day each i.e. 0.04 (yard areas/car port) 20hr/day each i.e. 0.8 (living areas) 6hrs/day each i.e. 0.25 (bedroom)	1hr/day			1hr/day each i.e. 0.04 (yard areas) 20hr/day each i.e. 0.8 (living areas) 6hrs/day each i.e. 0.25 (bedroom) 4hrs/day, 2 days/week, 6 months/year each i.e. 0.024 (pool area)				1hr/day each i.e. 0.04 (yard areas)	1hr/day each over say 6 weeks i.e. 4.6×10^{-3} 6hrs/day each over say 6 weeks i.e. 0.03	1hr/day each i.e. 0.04 (yard areas) 20hr/day each i.e. 0.8 (living areas) 6hrs/day each i.e. 0.25 (bedroom) 4hrs/day, 2 days/week, 6 months/year each i.e. 0.024 (pool area)
Probability of Not Evacuating Area Affected	0.2 (yard areas) 0.4 (house)	0.2			0.2				0.2	0.4	0.2 (yard and pool areas) 0.4 (house)
Spatial Probability	1m failure over 5m length of wall i.e. 0.2				2m failure over 10m length of slope i.e. 0.2				3m failure over 5m length of wall i.e. 0.6	1m failure over 5m length of excavation i.e. 0.2	1m failure over 5m length of wall i.e. 0.2
Vulnerability to Life if Failure Occurs Whilst Person Present	0.1 (yard areas/car port) 0.4 (house)	0.1 (yard areas)			0.1 (yard and pool areas) 0.4 (house)				0.1 (yard areas)	0.1	0.1 (yard and pool areas) 0.4 (house)
Risk for Person Most at Risk	1.6×10^{-9} (yard areas) 2.6×10^{-7} (living areas) 8×10^{-8} (bedroom)	1.6×10^{-9}			1.6×10^{-8} (yard areas) 1.3×10^{-6} (living areas) 4×10^{-7} (bedroom) 9.6×10^{-9} (pool area)				4.8×10^{-8}	3.7×10^{-9} 2.4×10^{-8}	1.6×10^{-9} (yard areas) 2.6×10^{-7} (living areas) 8×10^{-8} (bedroom) 9.6×10^{-10} (pool area)
Total Risk	3.2×10^{-9} (yard areas) 5.2×10^{-7} (living areas) 1.6×10^{-7} (bedroom)	3.2×10^{-9}			3.2×10^{-8} (yard areas) 2.6×10^{-6} (living areas) 8×10^{-7} (bedroom) 1.92×10^{-8} (pool area)				9.6×10^{-8}	7.4×10^{-9} 4.8×10^{-8}	3.2×10^{-9} (yard areas) 5.2×10^{-7} (living areas) 1.6×10^{-7} (bedroom) 1.92×10^{-9} (pool area)

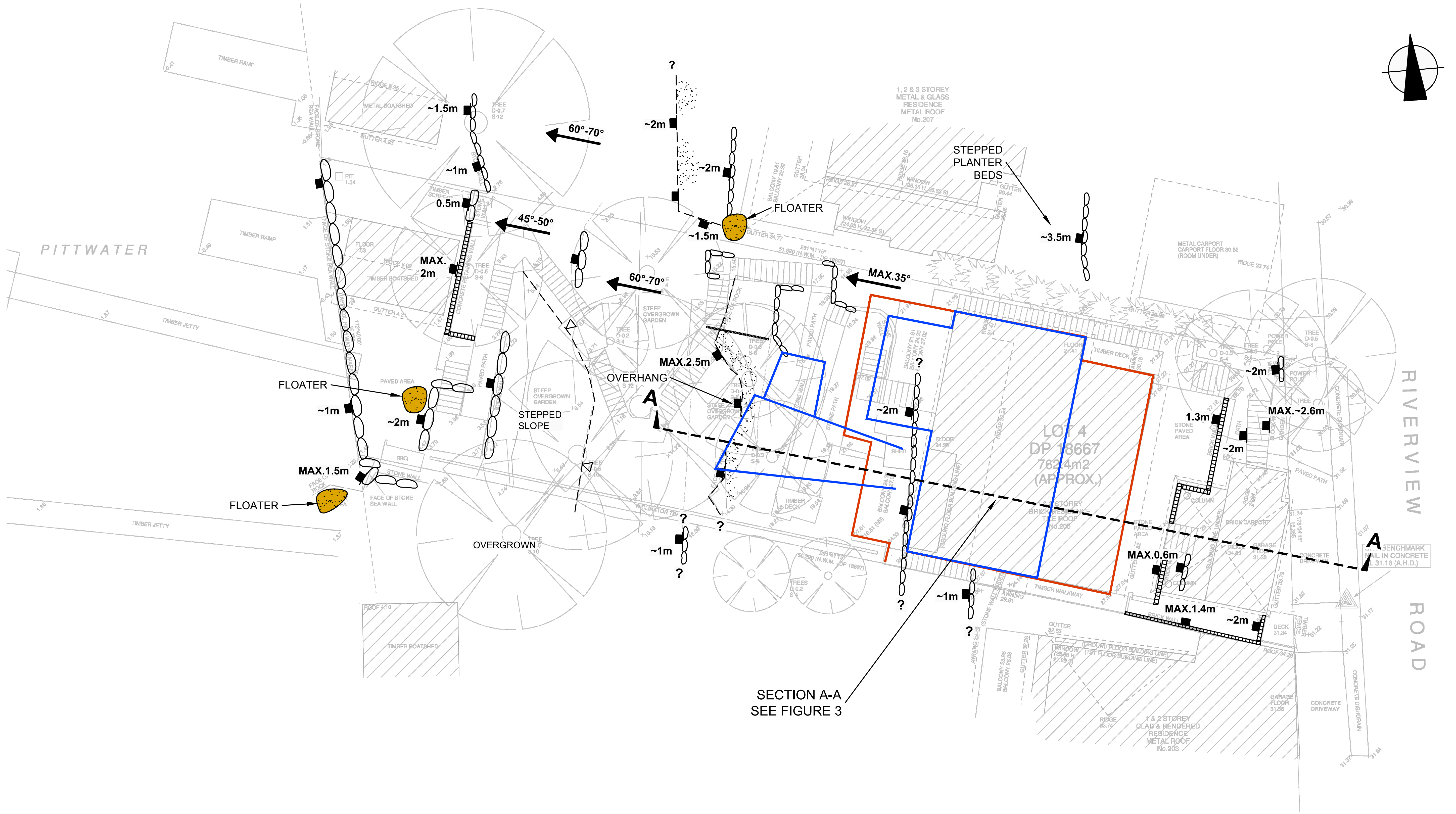
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 14 MAY 2019.

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

Title:	**SITE LOCATION PLAN**
Location:	205 RIVERVIEW ROAD AVALON BEACH, NSW
Report No: 27796R	Figure No: 1

JKGeotechnics

PLOT DATE: 3/07/2019 2:01:16 PM   DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\27000'S\27796ZR CLAREVILLE\CAD\27796R.DWG



PROPOSED LOWER GROUND FLOOR LEVEL

PROPOSED POOL LEVEL

FOR AN EXPLANATION OF MAPPING SYMBOLS SEE FIGURE 4

Title:	GEOTECHNICAL SKETCH PLAN
Location:	205 RIVERVIEW ROAD AVALON BEACH, NSW
Report No: 27796R	Figure No: 2

0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

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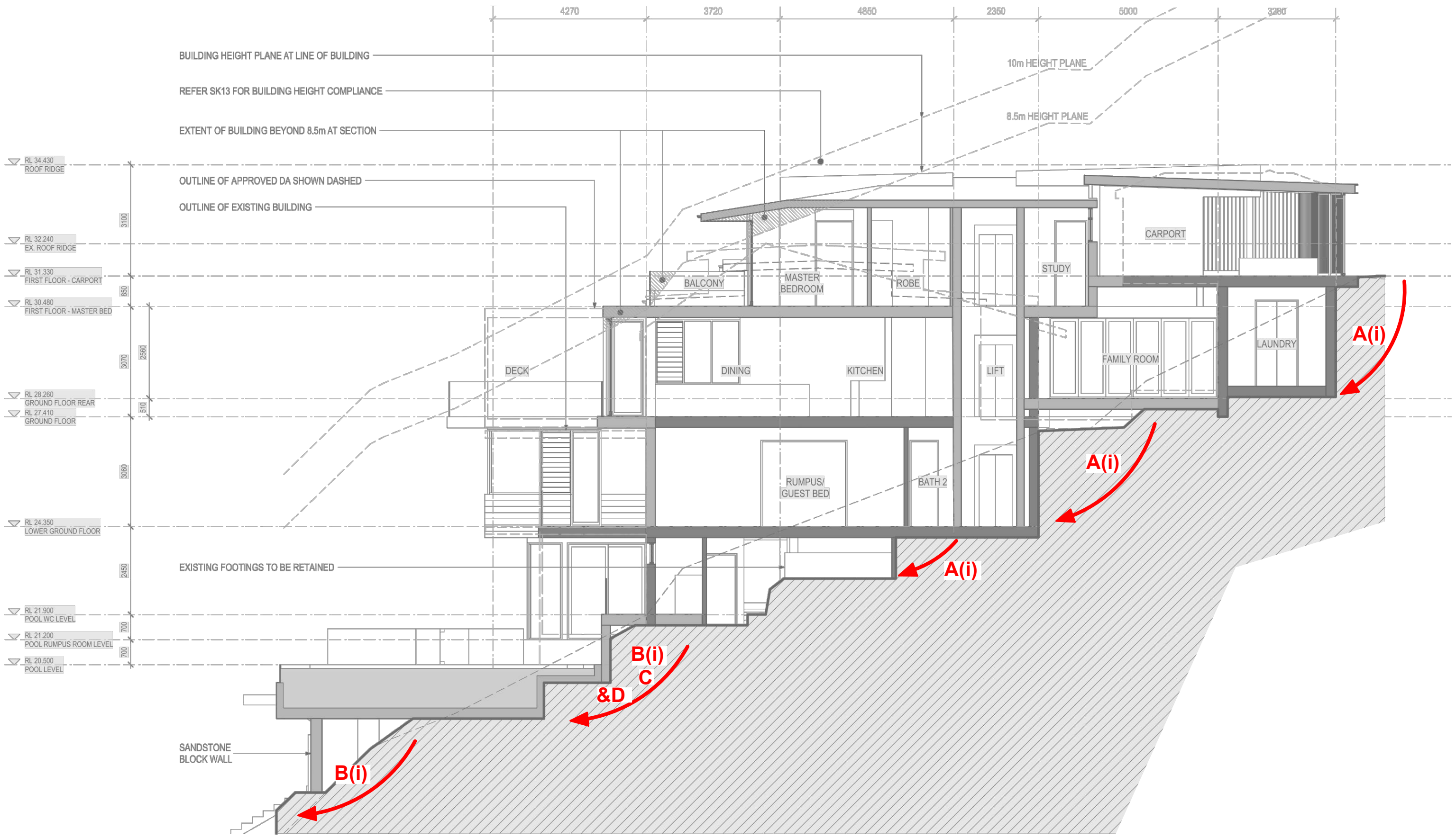


0 2 4 6 8 10 SCALE 1:200 @A4 METRES	Title: **SECTION A-A SHOWING POTENTIAL LANDSLIDE HAZARDS**
	Location: 205 RIVERVIEW ROAD AVALON BEACH, NSW
	Report No: 27796R — Figure No: 3
This plan should be read in conjunction with the JK Geotechnics report.	JKGeotechnics

PLOT DATE: 4/09/2019 8:01:38 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\27000'S\27796ZR CLAREVILLE\CAD\27796R.DWG

TOPOGRAPHY

Symbol *Ground Profile*

convex
concave
} well defined or angular break of slope

convex
concave
} poorly defined or smooth change of slope

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

sharp
rounded
} ridge crest

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

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

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

Hummocky or irregular ground

OTHER FEATURES

Boulder

Seepage/spring

Swallow hole for runoff

Natural water course

Open drain, unlined

Open drain, lined

Fenceline

Property boundary

Dry Stone Wall

J —— J
200
Major joint in rock face (opening in millimetres)

– T – T –
10
Tension crack (opening in millimetres)

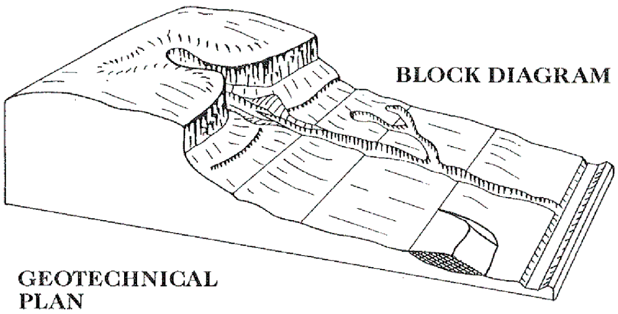
Masonry or concrete wall

Ponding water

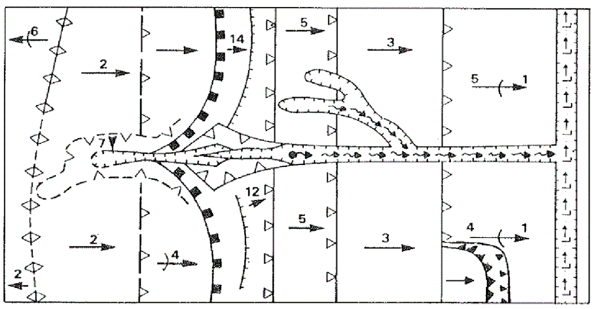
Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:

BLOCK DIAGRAM



GEOTECHNICAL PLAN



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

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

Title:	GEOTECHNICAL MAPPING SYMBOLS
Location:	205 RIVERVIEW ROAD AVALON BEACH, NSW
Report No: 27796R	Figure No: 4

JKGeotechnics

APPENDIX A

LANDSLIDE RISK

MANAGEMENT

TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

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

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

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

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

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

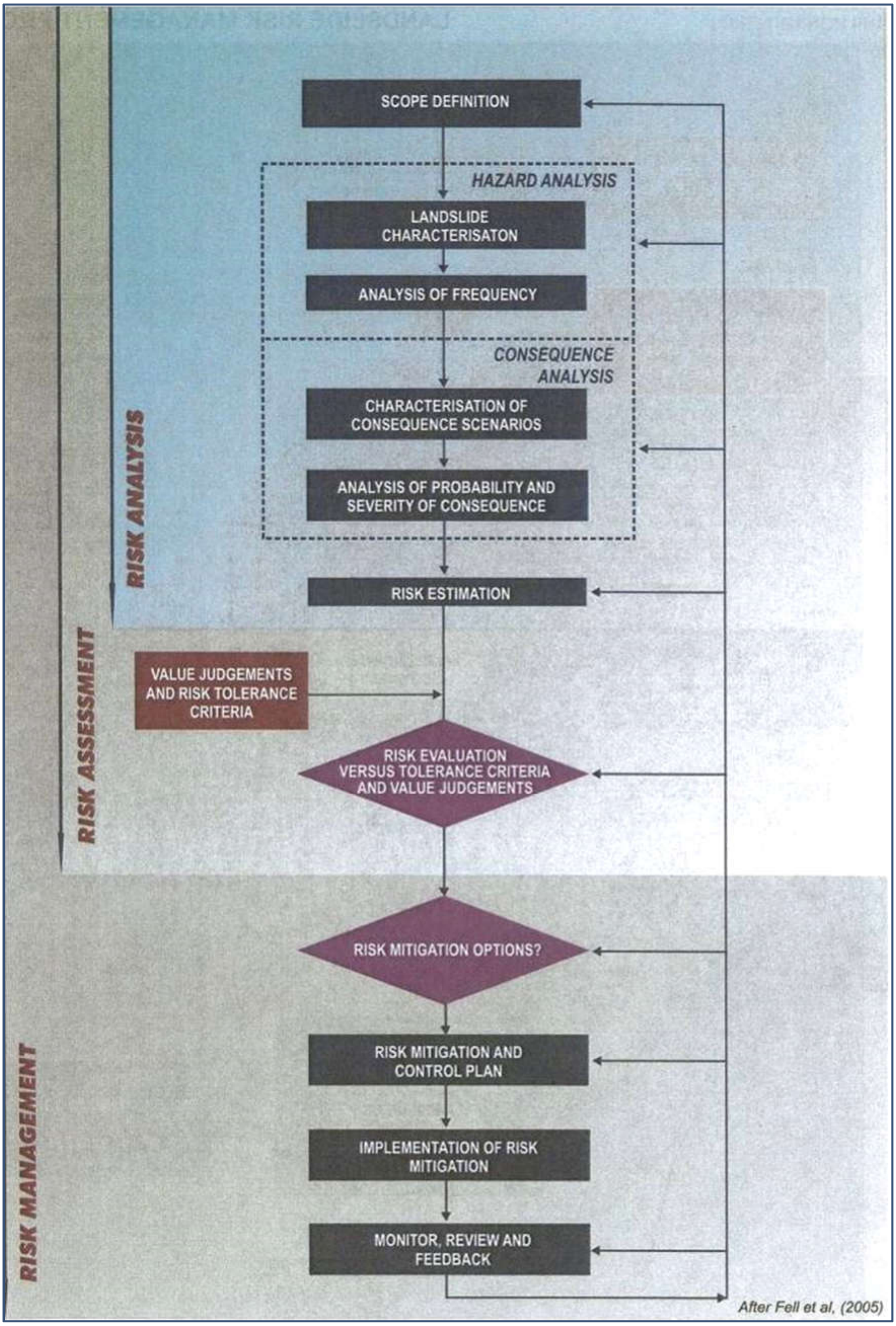


FIGURE A1: Flowchart for Landslide Risk Management.

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

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

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

RISK LEVEL IMPLICATIONS

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

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

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

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

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

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

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

What Causes a Landslide?

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

Does a Landslide Affect You?

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

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

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

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

TABLE 1 – Slope Descriptions

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

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

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

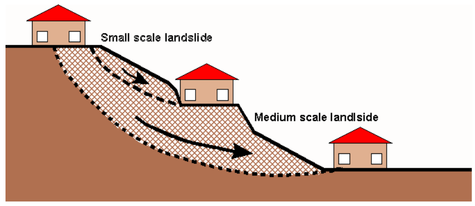


Figure 1

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

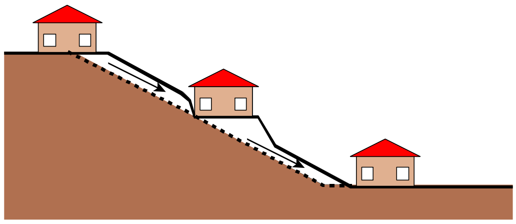


Figure 2

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

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

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

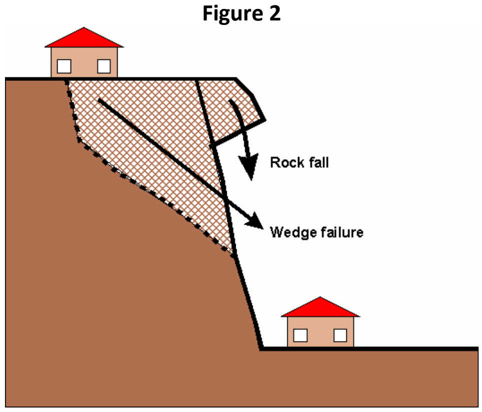


Figure 3

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

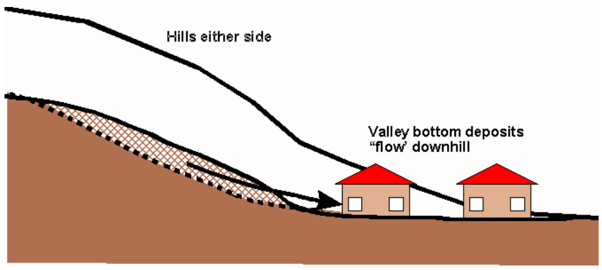


Figure 4

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

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

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AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

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

Landslide Risk Assessment

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

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

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

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

Risk to Property

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

TABLE 2 – LIKELIHOOD

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

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

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

TABLE 1 – RISK TO PROPERTY

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

Risk to Life

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

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

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

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

TABLE 3 – RISK TO LIFE

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

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
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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.

APPENDIX B

SOME GUIDELINES

FOR

HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

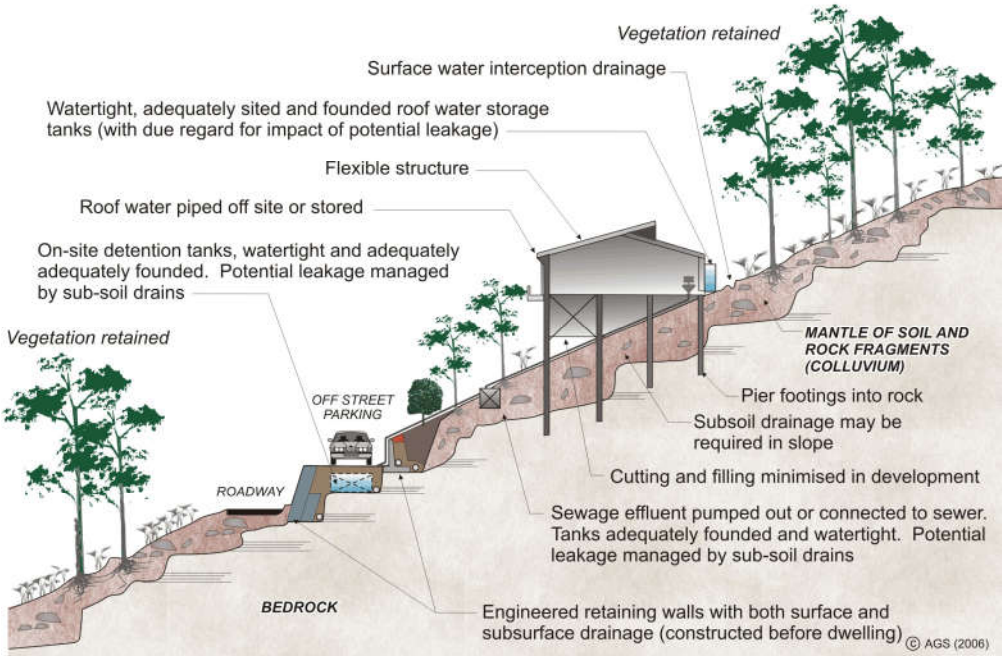
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having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

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

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

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

EXAMPLES FOR **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

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

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

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

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

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

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

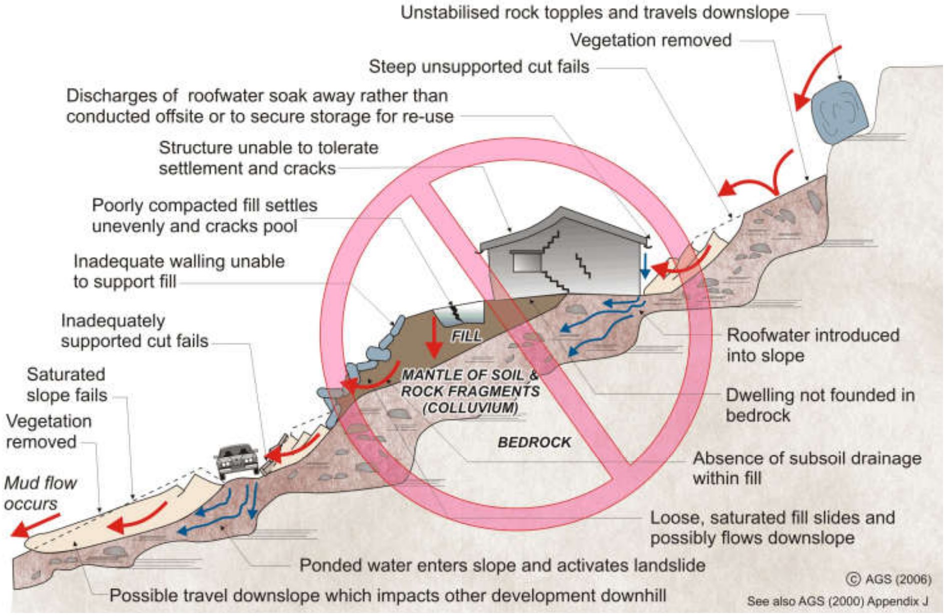
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

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

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

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

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

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

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

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

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

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

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

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

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

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
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APPENDIX C

31 July 2015
Ref: E27796Klet-rev1

K. Rooney
205 Riverview Road
CLAREVILLE NSW 2107

PRELIMINARY DESKTOP ACID SULFATE SOIL ASSESSMENT

PROPOSED RESIDENTIAL DEVELOPMENT

205 RIVERVIEW ROAD, CLAREVILLE, NSW

1 INTRODUCTION

Vitale Design, on behalf of K. Rooney ('the client') commissioned Environmental Investigation Services (EIS)[1] to undertake a preliminary desktop acid sulfate soil (ASS) assessment for the proposed residential development at 205 Riverview Road, Clareville ('the site'). The site is identified as Lot 4 in DP18667 and at the time of this investigation was occupied by a 1-2 storey brick house, a brick carport and a steeply sloping backyard that led down to a timber boatshed, ramp and jetty providing access to Pittwater.

The investigation was undertaken generally in accordance with an EIS proposal (Ref: EP8381K) of 16/9/14 and written acceptance from Vitale Design on behalf of K. Rooney of 26/9/14.

This report describes the investigation procedures and presents the results of the ASS assessment, together with comments, discussion and recommendations.

A geotechnical assessment was undertaken in conjunction with the ASS assessment by JK Geotechnics[2] and the results are presented in the report *Geotechnical Assessment for Proposed Alterations and Additions at 205 Riverview Road, Clareville, NSW* (Ref. 27796ZRrpt rev1, dated 31/7/2015). This *Preliminary Desktop Acid Sulfate Soil Assessment* is presented as an appendix to the geotechnical assessment report and should be read in conjunction with that report.

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[1] Environmental consulting division of Jeffery & Katauskas Pty Ltd (J&K)

[2] Geotechnical consulting division of J&K

2 BACKGROUND ON ACID SULFATE SOILS (ASS)

ASS are formed from iron-rich alluvial sediments and sulfate (found in seawater) in the presence of sulfate-reducing bacteria and plentiful organic matter. These conditions are generally found in mangroves, salt marsh vegetation or tidal areas and at the bottom of coastal rivers and lakes. These soils include those that are producing acid (termed actual ASS) and those that can become acid producing (termed potential ASS or 'PASS'). PASS are naturally occurring soils and sediment that contain iron sulfides (pyrite) which, when exposed to oxygen generate sulfuric acid.

The NSW government in 1994 formed the Acid Sulfate Soils Management Advisory Committee (ASSMAC) to coordinate a response to ASS issues. In 1998 this group released the Acid Sulfate Soil Manual[3] providing best practice advice for planning, assessment, management, laboratory methods, drainage, groundwater and the preparation of ASS management plans (ASSMP).

In 1997 the Department of Land and Soil Conservation (now part of the Office of Environment and Heritage[4]) developed two series of maps with respect to ASS for use by council and technical staff implementing the ASS Manual 1998:

- ASS Planning Maps – issued to councils and government units; and
- ASS Risk Maps – issued to interested parties.

The ASS planning maps provide an indication of the relative potential for disturbance of ASS to occur at locations within the council area. These maps do not provide an indication of the actual occurrence of ASS at a site or the likely severity of the conditions. The maps are divided into five classes dependent upon the type of activities/works that if undertaken, may represent an environmental risk through the development of acidic conditions associated with ASS:

Table 2-1: Risk Classes

Risk Class	Description
Class 1	All works.
Class 2	All works below existing ground level and works by which the water table is likely to be lowered.
Class 3	Works at depths beyond 1m below existing ground level or works by which the water table is likely to be lowered beyond 1m below existing ground level.
Class 4	Works at depths beyond 2m below existing ground level or works by which the water table is likely to be lowered beyond 2m below existing ground level.

[3] *Acid Sulfate Soils Manual,* Acid Sulfate Soils Management Advisory Committee (ASSMAC), 1998 (ASS Manual 1998)

[4] http://www.environment.nsw.gov.au/acidsulfatesoil/index.htm

Risk Class	Description
Class 5	Works within 500m of adjacent Class 1,2,3,4 land which are likely to lower the water table below 1m AHD on the adjacent land.

The ASS risk maps provide an indication of the probability of occurrence of PASS at a particular location based on interpretation from geological and soil landscape maps. The maps provide classes based on high probability, low probability, no known occurrence and areas of disturbed terrain (site specific assessment necessary) and the likely depth at which ASS may to be encountered.

3 REGIONAL GEOLOGY AND SOIL INFORMATION

3.1 Broken Bay Council Planning Map

The ASS planning map for the area available on the Pittwater Council website[5] indicates that the site is located in a Class 5 risk area.

3.2 ASS Risk Map

The ASS risk map for Broken Bay prepared by the Department of Land and Soil Conservation (1997[6]) indicates that the site is located in an area classed as having no known occurrence of acid sulfate soil materials.

4 CONCLUSION AND RECOMMENDATIONS

The site is situated on a steep slope. The proposed development is confined to the eastern section of the site near the top of the slope.

Based on the available information the risk of encountering acid sulfate soil is considered to be very low for the following reasons:

- The ASS risk map indicates that the site is located in an area classed as having no known occurrence of acid sulfate soil materials;
- Sandstone outcrops were observed across the site. Acid sulfate soils are usually associated with estuarine swamps and coastal wetlands, not residual soils derived from weathered bedrock;
- The Lower Level Three of the proposed development is at an elevation of approximately RL17.95m. Acid sulfate soils are usually associated with soil horizons less than 5m AHD.

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[5] http://www.legislation.nsw.gov.au/mapindex?type=epi&year=2014&no=320#ASS visited on 13/10/14

[6] *1:25,000 Acid Sulfate Soil Risk Map (Series 9130N1, Ed 2),* Department of Land and Soil Conservation (1997)

An Acid Sulfate Soil Management Plan is not considered necessary for the proposed development at the site.

5 LIMITATIONS

The report limitations are outlined below:

- EIS adopts no responsibility for any unidentified ASS that may be located at the site. Any unexpected problems/subsurface features that may be encountered during development works should be inspected by an environmental consultant as soon as possible;
- This report has been prepared based on site conditions which existed at the time of the investigation; scope of work and limitation outlined in the EIS proposal; and terms of contract between EIS and the client (as applicable);
- The conclusions presented in this report are based on investigation of conditions at specific locations, chosen to be as representative as possible under the given circumstances, visual observations of the site and immediate surrounds and documents reviewed as described in the report;
- Subsurface soil and rock conditions encountered between investigation locations may be found to be different from those expected. Groundwater conditions may also vary, especially after climatic changes;
- The investigation and preparation of this report have been undertaken in accordance with accepted practice for environmental consultants, with reference to applicable environmental regulatory authority and industry standards, guidelines and the assessment criteria outlined in the report;
- Where information has been provided by third parties, EIS has not undertaken any verification process, except where specifically stated in the report;
- EIS has not undertaken any assessment of off-site areas, except where specifically stated in the report;
- EIS accept no responsibility for potentially asbestos containing materials that may exist at the site. These materials may be associated with demolition of pre-1990 constructed buildings or fill material at the site;
- EIS have not and will not make any determination regarding finances associated with the site;
- Additional investigation work may be required in the event of changes to the proposed development or land use. EIS should be contacted immediately in such circumstances;
- Material considered to be suitable from a geotechnical point of view may be unsatisfactory from an environmental viewpoint, and vice versa;
- 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;
- Copyright in this report is the property of EIS. EIS has used a degree of care, skill and diligence normally exercised by consulting professionals 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;

- If the client, or any person, provides a copy of this report to any third party, such third party must not rely on this report except with the express written consent of EIS; and
- Any third party who seeks to rely on this report without the express written consent of EIS does so entirely at their own risk and to the fullest extent permitted by law, EIS accepts no liability whatsoever, in respect of any loss or damage suffered by any such third party.

If you have any questions concerning the contents of this letter please do not hesitate to contact us.

Yours faithfully
ENVIRONMENTAL INVESTIGATION SERVICES

Rob Muller
Environmental Scientist

Adrian Kingswell
Principal

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

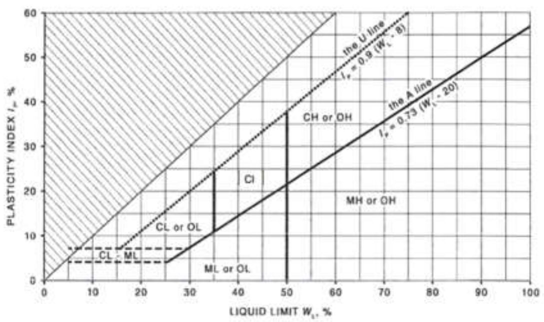
$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

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

NOTES:

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

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



LOG SYMBOLS

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

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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