REPORT

TO

MR JIM NEALE

ON

GEOTECHNICAL ASSESSMENT

FOR

PROPOSED MULTI-STOREY DEVELOPMENT

AT

1,1A & 5 AVON ROAD AND
4 & 8 BEECHWORTH ROAD, PYMBLE, NSW

19 November 2010

Ref: 23513Wrpt Rev1

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FIGURE 3: PROPOSED SITE LAYOUT

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REPORT EXPLANATION NOTES

1 INTRODUCTION

This report presents our geotechnical assessment of the proposed development site comprising the existing numbers 1, 1A and 5 Avon Road, and 4 and 8 Beechworth Road, Pymble, as shown on the attached Figure 1. The assessment was commissioned by Mr Jim Neale in response to our Proposal (Ref: P31535Wemail) dated 2 October 2009.

The geotechnical assessment is based on our walkover survey of the site, together with data available from other development sites nearby. The purpose of the assessment was to provide information required by the Director-General's Requirements (DGR) dated 11 February 2009 (Application No MP 08-0207). DGR Item *'11 Geotechnical and Hydrogeological'* states:

> *"A geotechnical assessment is required to*:
>
> - *Ensure that the land is capable of supporting the proposed development;*
> - *Assess the potential slip hazard on steep slopes;*
> - *Assess the potential impact on ground water flows and downstream ecosystems (including the creeks) and any measures proposed to mitigate them;*
> - *Assess the potential of any development to intersect groundwater flows and the measures proposed to mitigate the impact of the development."*

We note that a meeting was held between the parties on 23 April 2009 to further understand the DGR.

A previous walkover survey and geotechnical assessment was prepared by this company and has been reported in our letter report dated 4 June 1993 (Ref. 9594W/a). This current report updates that previous assessment and addresses the issues raised by the DGR in relation to the development proposals.

We have revised the report by updating Figure 3 as attached, with minor amendment of Section 4.1 to suit. This Rev1 of our report supersedes the original version dated 6 November 2009 (ref. 23513Wrpt).

2 ASSESSMENT PROCEDURE

Our assessment of the site has involved the following:

- A walkover survey across the site area using limited available tracks. This was carried out by a Principal of this company, Mr Bruce Walker on 27 October 2009 after a period of wet weather. We note that access in the central part of the site was limited due to dense vegetation cover.
- During the walkover survey, observation was also made of the adjacent cuttings within the rail corridor for the North Shore Railway Line.
- Inspection of the Sydney geological map (Scale 1:100,000).
- Review of data from nearby sites held in the archives of Jeffery and Katauskas. In particular, we have referred to boreholes completed to the north-east of the project at the southern end of Clydesdale Place and boreholes completed to the south-east of the project at the corner of Avon Road and Pymble Avenue. In addition, other investigations have been carried out at the nearby Pymble Ladies College (PLC), but these are typically at lower elevations than the subject site.
- During the walkover survey, site features were mapped relative to the features shown on the survey plan provided. This survey plan forms the basis for Figure 1 and is understood to have been prepared by Daw & Walton, Surveyors (Job No 800-09) and is based on a survey completed in 2009. It is understood that levels shown on the plan are to Australian Height Datum (AHD). Additional observations of hillside slopes have been carried out using a hand held inclinometer. The resultant ground slopes and morphological features shown on

Figure 1 use the symbols illustrated on the attached Figure 2. Selected photographs have also been included.

A set of explanatory notes attached to this report gives further explanation of some of the terms and methodologies referred to herein.

3 RESULTS OF ASSESSMENT

3.1 Summary of Observations

The site is located on hillside slopes which are generally sloping to the south-west. The North Shore Railway Line forms the north-eastern site boundary. The site is irregular in shape, due to the combination of lots involved for the proposed development (Figure 1). Typically, the main site area for development is about 160m wide (north-west to south-east) by about 150m deep (north-east to south-west).

The principal topographic features are localised ridges on the main hillside slope, which are roughly followed by Beechworth Road to the north-west, and Avon Road to the south-east of the property. Ground slopes on the ridge lines are typically at about 7° to 8° at the upper elevation of the subject site, but flatten at lower elevations beyond the site to less than 5°.

Between these two ridges there is a drainage gully which drains to the south-west. The hillside slopes within the site are from the ridge lines towards the drainage gully and typically are at about 8° to 10° on the ridge lines which are gently rounded. The slopes steepen typically to between about 18° to 26° with some locally steeper slopes immediately adjacent to the drainage gully. These local steeper slopes are at about 25° to 30°. The drainage gully is incised by typically about 3m to 4m below the intermediate hillside slopes. At the time of the walkover, some surface water flow was occurring within the drainage gully due to rain storms in the preceding days. The drainage gully area is heavily overgrown and access was limited to localised tracks.

The North Shore Railway Line has been constructed by cut and fill approximately as indicated on Figure 1, as shown on Photograph 1. Substantial cuts are present at Beechworth Road, exposing an estimated 6m to 7m of weathered shale and residual clays (Photograph 2). The shale at the base of the cutting was estimated to have been formed at about 60° to the horizontal and appeared to be a stronger, less weathered shale. Surface fretting and erosion of the shale bedrock was apparent and shotcrete protection is present on the north-eastern side of the shale cuttings (Photograph 1).

Shale bedrock was also exposed in the railway cutting adjacent to No 1 Avon Road, although the depth of cut was substantially less, being estimated at about 4m to 6m as seen in Photograph 3. The shale appeared to be of poorer quality, being more weathered and fragmented, and is similar to the upper shales at the Beechworth Road cut. The approximate extent of the two cuts is shown in Photograph 1. Between the two cuts, a fill embankment is present and could be observed from the site boundary with the rail corridor. The fill batter appeared to be of about 4m to 5m in overall height and was estimated to be at about 25° to 30° to the horizontal overall. However, the batter profile appeared to be locally steeper nearer the crest. The toe of the fill batter was set back about 3m to 5m (estimated) from the boundary fence marked by rail uprights.

Existing residential dwellings are present on the lots around the perimeter of the development site, as indicated on Figure 1. The condition of these existing dwellings varies and localised cut and fill appears to have taken place during a past development.

It was noted during our 1993 site visit that No 4 Beechworth Road was in a relatively poor condition and had been extended a number of times. Evidence of differential movement of the brick structure was apparent from cracking.

We understand that the dwelling was demolished in about 1994. The area formerly occupied by the dwelling was found to be heavily overgrown, obscuring apparent remnant retaining walls or batters formed after demolition. A steeper batter, estimated at about 30°, appeared to be present along the eastern side of the former dwelling location. A brick retaining wall of about 1.5m height was present on the western side of the old tennis court uphill/west of the former dwelling location. The brick walls were leaning forward at the southern end, and were cracked/bulging at the north-western corner.

No 8 Beechworth Road was a newer dwelling, but in 1993 still appeared to have some evidence of differential movement, particularly around the garage and patio areas. This property was not reinspected during our 2009 walkover survey. It was noted however that the hillside slopes around No 8 were relatively uniform and, as discussed above, steepened towards the drainage gully.

Relatively steep batters are adjacent to No 10A Beechworth Road, located at the corner of the L-shape on the western side. The batter was heavily overgrown and appeared to be formed by fill extending onto the site (Photograph 4). Landscaped lawn and terrace areas are adjacent to the northern side of No 10A.

The dwelling at No 1 Avon Road is located on the ridge line on the eastern corner of the block. Access is provided from the extension of Avon Road parallel to the railway. Localised excavation appears to have been carried out for formation of the roadway, exposing residual clay soils with weathered shale fragments, forming an irregular steep batter of about 2m height. The clays were of medium to high plasticity.

The building on No 1 has not been examined for evidence of cracking. The area surrounding this dwelling has been extensively modified by past landscaping. Some of the landscaping features are shown on the survey plan (Figure 1). Sandstone block retaining walls are present around the terraced areas on the western side of the dwelling and, where visible, appeared to be in reasonable condition.

A disused tennis court is located in the gully area towards the southern end of the lot. The tennis court area appears to have been formed by past cut into the hillside slopes and filled close to the drainage gully. The tennis court is severely overgrown. The cut batters were supported by terraced garden-type areas supported by sandstone flagstones, as shown in Photograph 5. The overall cut height was estimated to be about 3m at about 36° to the horizontal.

The dwelling at No 5 Avon Road straddled the ridge line location. Ground slopes at the front were towards the south-east onto Avon Road, typically at about 6° to the south and south-east. To the west of the dwelling, hillside slopes increased from about 7° to 10° to the west and south-west and increased to 14° to 20° on the steeper slopes at lower elevations. The dwelling at No 5 appeared to be vacant and was fire-damaged.

The drainage gully in the centre part of the site appears to be fed from a drainage culvert passing beneath the southern end of the railway fill batter. Discharge from this culvert has caused local scour erosion at the foot of the fill batter and within the adjoining gully area on the subject site. The main drainage gully passed adjacent to the dwelling on No 1 Arilla Avenue into a piped culvert.

3.2 Anticipated Subsurface Conditions

The 1:100000 geological map identifies the shale bedrock at the site to be Ashfield Shale of the Wianamatta Group. The shales are shown to extend to lower elevations beyond the site where the transition to the underlying Hawkesbury Sandstone occurs. Past investigations carried out for the nearby PLC school have encountered predominantly shale bedrock below the area of the main school buildings close to the oval. However, further to the south, sandstone bedrock has also been encountered. The clay soil profiles vary from about 1m to 4m in depth and confirmed the clays to be of moderate to high reactivity to seasonal changes in moisture content.

A preliminary investigation was carried out at the south-western end of Clydesdale Place about 70m north-east from No 1 Avon Road. The investigation comprised one augered borehole which encountered a relatively shallow residual clay profile of about 0.3m grading into distinctly weathered shale of very low to low strength. This relatively poor quality shale extended to about 8.5m depth and thereafter the shale became of low to medium strength. Groundwater was not encountered during drilling.

More extensive investigations have been carried out for the development at the corner of Avon Road and Pymble Avenue about 300m to 400m south-east from No 1 Avon Road. Relatively deep cored boreholes were completed on this site, though it is noted that there were significant changes in elevation due to the steep topography and drainage gully present at that site. The residual clays of medium to high plasticity were encountered to depths of about 2m to 3.6m below existing ground levels. The underlying weathered shale bedrock was extremely weathered and of extremely low to low strength at first contact. With depth, the degree of weathering decreased and there is a commensurate increase in rock strength. The upper shales of poor quality, being typically Class 5 to Class 4 shales, extended to depths varying between about 8m to 15m below existing ground levels.

The underlying shale was typically of better quality, being of Class 3 to Class 1, depending on elevation and location across the site. Groundwater seepage was encountered within the upper poor quality shale, typically at depths between about 5m to 7m below existing grade. No long-term groundwater monitoring had been completed at the time of our investigation report. It is noted that observations during excavation on that site indicate only localised groundwater seepages from the cut faces.

From these adjacent investigations and the cut batters on the North Shore Railway Line, a subsurface profile of residual clays of medium to high plasticity grading into the underlying weathered shale can be anticipated on the development site. The depth of clays is likely to be about 2m to 3m on the ridge lines, and may be less in the drainage gully. Upper shales will be of relatively poor quality and very low to low strength.

Due to the hillside location and topography of the site, it is considered that the site, including the gully, would be subject to surface erosion over geological time. It is considered unlikely there would be any deposition over the site, even within the gully, over geological time.

4 COMMENTS AND PRELIMINARY RECOMMENDATIONS

4.1 Proposed Development

It is understood that the proposed development will comprise a multi-storey residential development with five building blocks. The Site Concept Plan prepared by Anchor Mortlock Woolley (Project No 0909, Dwg No CP-100 Rev A), is presented on Figure 3. Also shown on this layout is the indicative staging for the development, which we understand will progress from Block 1 to Block 5.

It is understood the building heights will range from three to 11 storeys, with the maximum height adjacent to the gully, due to the drop in ground level towards the gully. Basement carparking is proposed beneath the buildings, being typically of two to three carparking levels. Excavation below existing ground level of between about 6m to 9m is anticipated. We note that the development is predominantly on the south-eastern side of the existing drainage gully with the Stage 5 block being more removed from the drainage gully on the north-western side off Beechworth Road. Access to the blocks is via a driveway system, as indicated on Figure 3. In the absence of specific design loads, we would assume moderate to high column loads.

4.2 Suitability for Proposed Development

Although the site has topographic constraints associated with the two ridgelines and the central gully, the overall hillside slopes are only up to moderate in slope, and are similar to other slopes along the North Shore ridgeline already developed for residential purposes, such as in Turramurra, Gordon and Killara. In addition, the anticipated subsurface profile is typical of most of the North Shore ridgeline, being residual clays grading into the underlying weathered shales.

The nature of the proposed development is relatively routine and proven engineering solutions are available for the proposed extent of excavation. We outline in the sections following the appropriate engineering recommendations to cater for the anticipated subsurface conditions, given the scope of the proposed development.

We consider the site is suitable from a geotechnical perspective for the proposed development.

4.3 Landslide Risk Assessment

Landslides are known to occur in the Ashfield Shales found in the West Pennant Hills to Castle Hill area on westerly facing slopes. The accepted cause of these landslides is a combination of deforestation together with geological features associated with minor folding of the strata. To our knowledge, no such landslide features have been identified on the North Shore ridgeline. There is no indication from the site topography and site performance that such landslide features are found on the subject site. Therefore, we consider the site is not subject to large scale instability associated with the geological setting.

Landslide hazards can be present or caused by development such as due to cuts and fills. Proven long term stability along the North Shore Railway Line illustrates the overall stability of cuts formed typically at about 1 Vertical (V) in 1.5 Horizontal (H) to 1V:1.25H within the residual clay and upper weathered shale profile.

Similarly, instability of the rail embankments is generally not problematic, though it is understood that past instability has occurred on a fill embankment just south of Turramurra Station. There was no evidence noted of such instability on the rail embankment adjacent to the subject site. Therefore, we have considered the site topographic features in accordance with the Australian Geomechanics Society *'Practice Note – Guidelines for Landslide Risk Management (AGS2007c)'* as discussed below.

Our landslide risk assessment has been prepared on a qualitative basis from our assessment of the potential landslide hazards at the site and the indicative consequences to property should be landslide hazard occur. (We note that we have not attempted any quantification of the cost of consequences, but have adopted the qualitative description given in Appendix A tables.) Based on this, the qualitative risk to property has been determined. The terminology adopted for the qualitative assessment is in accordance with Table A1 given in Appendix A.

We have considered five indicative landslide hazards relevant for the site and subject development are considered in the attached Table A. These hazards comprise:

- Instability of the natural hillside slopes (A).
- Instability of existing fill batters – either associated with the existing North Shore Railway Line (B) or the localised batter at No 10A Beechworth Road (C).
- Instability of cut batters where unsupported by properly engineered retaining walls, but batters are assumed to be cut to 1V:2H as recommended below (D).
- Instability of proposed retaining walls, which are assumed to be properly engineered in accordance with recommendations given below (E).

We note that our evaluation of the frequency of the landslides is based on two considerations.

Firstly, the probability of landsliding is expected to be significantly less than that experienced in the Pittwater area, since there is no known or reported landslides in the vicinity of the site or in similar settings elsewhere on the North Shire ridgeline. Therefore, the indicative annual likelihood of instability of the natural hillside slopes is considered to be less than 10^{-5}pa. Similarly, for unsupported cut batters, the likelihood of instability would be related to the batter angle and the recommended batters have a proven low probability of failures.

Secondly, evidence from the cuts formed for the North Shore Railway Line is that typically the cut batters have remained stable since excavation in the late 1890s or early 1900s, though localised surface fretting does occur in the weathered shales. In addition, some localised earth slumps can occur in the residual profile in the vicinity of the crest of the relatively steep slopes which are typically at about 35° to 40° to the horizontal.

From our site mapping, the cut batters on the rail line will not directly affect the subject site. In addition, we anticipate that all required basement excavations will be provided with properly engineered retaining walls as discussed below. If required for access roads or landscaping, cut batters would be recommended at no steeper than 1V:2H through the residual soils.

The existing fill batter at the rail line is also considered to have a relatively low probability of failure based on its performance since construction in the 1890s. In addition, such embankments have generally remained stable along the North Shore Railway Line, with the exception noted above. Therefore, we anticipate that the annual probability of failure of this fill embankment would be about or less than 10^{-4}pa. At this stage, we have conservatively assessed a higher probability of instability of the apparent fill batter associated with No 10A. However, we note this batter is relatively minor and will not directly affect any of the proposed development.

The fill batters are anticipated to be comprised of clay and weathered shale derived from nearby cuts/ excavation. Should instability occur, the run-out distance for such material and size/ height of fill embankment would be expected to be limited to between about 3m and 20m from the embankment toe.

We have assessed the consequences to property, as summarised in Table A, considering the likely scale of the instability relative to the layout shown on Figure 3.

The attached Table A indicates that the assessed risk to property would be Low to Very Low, which would be considered to be 'acceptable' in accordance with the criteria given in Reference 1. These criteria have also been adopted by other development authorities, such as Pittwater Council and for the Kosciusko National Park.

We have also used indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. We have assumed that the person most at risk is either a person walking regularly through the landscaped garden areas or a nominal person accessing carparking within the basements or site access driveways. The temporal and vulnerability factors that have been adopted are given in the attached Table B, together with the resulting risk calculation. Our assessed risk to life for the person most of risk is about $5x10^{-7}$pa. This also would be considered to be 'acceptable' in relation to the criteria given in Reference 1.

In preparing the above assessment we recognise that, due to the many complex factors that can affect the site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, that the risk of instability of a site and/or development cannot be completely removed. It is, however, essential that that risk be reduced to at least that which can be reasonably anticipated by the community in everyday life, and that landowners be made aware of reasonable and practical measures available to reduce risk as far as possible. Therefore, the recommendations given below have been proposed to achieve this aim.

We consider that our risk analysis has shown that the site and proposed development can achieve the acceptable risk management criteria usually adopted for residential developments, provided the recommendations given below are adopted. These recommendations form an integral part of the landslide risk management process.

4.4 Impact on Groundwater Flows

The site is located near the major topographic feature of the North Shore ridgeline. Although it is located on the western side slopes, the site is still at a relatively high elevation. Nonetheless, there is still a groundwater catchment area to the east which may be controlling the groundwater levels present on site. Groundwater levels would be expected to fluctuate seasonally by about 1m to 2m between wet and dry periods.

Experience on the adjacent developments to the north-east and south-east indicate that groundwater is not a major constraint and only localised seepages would be anticipated in the upper weathered shale horizons. It is anticipated that such groundwater seepage would naturally be flowing to the main drainage gully passing through the subject site, as this gully is at a lower elevation than the adjacent ridgelines bordering the south-eastern and north-western sides of the site.

The proposed basement excavations may intersect some of the groundwater flows where the basement is locally deeper than, say, 6m. The flow quantity would be relatively minor and readily controllable using conventional subsurface drains associated with basement construction. Discharge from these drainage provisions would be via the stormwater system to the gully. Therefore the net effect of these basement excavations on flows into the drainage gully would be very limited and would not have any measurable effect. Some localised lowering of groundwater levels may be associated with the basement excavation, with areas adjacent to the deeper basement excavation possibly having a local drawdown of about 1m to 2m on the high side (typically to the north-east). As the site is predominantly bordered by the North Shore Railway Line, which already has excavations of a similar depth, the anticipated effect is relatively minor, if any.

4.5 Excavation

Excavations into the soil and more weathered shale of up to low strength should be readily achieved using conventional tracked excavators with increasing use of a ripping tyne to loosen materials as they become stronger, less weathered with depth. For excavations up to about 6m depth, it is likely that 'hard rock' excavation techniques will not be required. For the locally deeper excavations between, say 6m to 9m, some hard rock excavation may be anticipated. Such excavation would be readily accomplished by means of rock hammers, where the volume of excavation is limited.

Proximity of excavations to adjoining structures usually determines the need for extensive dilapidation reports and vibration monitoring. Given the basement set back from adjacent existing development, we anticipate that rock excavation techniques will be possible using conventional rock hammers typically of up to 600kg size. Use of vibration monitoring would be recommended on any existing development within, say, 15m from the hard rock excavation. Such a set back may apply to No 3 or No 7 Avon Road, and No 6 Beechworth Road.

We note that for larger excavation volumes, use could be made of large dozer tractors, such as Caterpillar D10 or equivalent. The need or appropriate use of such equipment may be better established once the geotechnical conditions are better defined by the detailed site investigations.

4.6 Excavation Support

The options for excavation support will involve a combination of the following:

- Temporary batter slopes, which may be formed in the residual soils and extremely weathered, extremely low to very low strength shale, at 1H:1V. Permanent batter slopes through these materials should be 1V:2H and the batter faces should be protected from erosion by revegetation, stone pitching or similar. Furthermore, allowance should be made for horizontal berms at the base of such temporary or permanent batter slopes where there is to be a steeper rock cut below, in order to enable barriers to be placed to collect loose material which otherwise may cause health and safety issues for workers within the excavation below.

- Temporary batter slopes in low to medium strength shale can sometimes be cut vertically to a reasonably stable condition. However, it should be anticipated there will be some steeply dipping joints and it would be more appropriate to adopt a batter of 2V:1H to avoid the need for most temporary stabilisation works, such as rock bolts, shotcrete and mesh, or combinations thereof. Permanent batters of 1V:1H should be adopted together with surface protection against long term fretting/ erosion such as by shotcrete or stone pitching.

- Permanent support to cuts in weathered shale in which temporary batters have been formed, would normally be provided by means of retaining walls braced by the building structures within the excavations. Even low to medium strength shale requiring rock excavation methods is likely to require some support, and allowance should be made in costing accordingly. Where basement retaining walls are supporting backfill to temporary excavations, then the walls should be designed for a coefficient of lateral earth pressure 'at rest', K_o = 0.6, together with a bulk density for the backfill of $20kN/m^3$. Allowance should be made for surcharge loads in addition to the above. Provision must be included for permanent and complete subsurface drainage measures behind such walls.

- Where temporary batters cannot be formed or are not desirable, then excavation should be supported by anchored soldier pile walls, assuming there are no movement sensitive structures within the zone of influence of the excavation. The zone of influence from the excavation is a horizontal distance equal to twice the vertical depth of excavation at the basement line. If there are movement sensitive structures within the zone of influence, then contiguous pile walls may be preferred. It is anticipated that the piles for either the soldier pile or contiguous pile wall would be most economically formed by means of conventional bored piles. The use of ground anchors where excavation depths are greater than about 2m to 3m is normal practise. Where such anchors may extend beyond property boundaries, then the permission of adjoining property owners must be sought. This approval may become a design constraint for any basements adjacent to the rail corridor, as railway authorities have been known to refuse permission for even temporary anchorages. There could also be an issue with installation of anchorages adjoining the steeply sloping creek banks where locally ground slopes are as steep as 30°, and ground anchors would be inclined at very steep angles to obtain any kind of anchorage and may therefore be inefficient.

- For preliminary design purposes, anchored soldier pile walls not adjacent to movement sensitive structures, may be designed on the basis of a trapezoidal pressure as shown in the attached Figure 4. It is anticipated that shotcrete infill would be provided in vertical stages between the anchored soldier piles. Depending on the operational requirements of the building, a dry wall is sometimes constructed in front of the shoring system, since it is possible there may be some dampness or long term seepage. Subsurface drainage measures would usually be installed behind the shotcrete panels as each panel is constructed.
- Consideration could also be given to soil nail walls comprising a grid of ground anchors typically on about 1.5m spacing both vertically and horizontally, tied into a reinforced shotcrete facing that can be used as both temporary and permanent support for excavations.
- Further advice on suitable retention schemes and design values should be provided based on additional geotechnical investigations.

4.7 Footing Design

It is anticipated that the moderate column loads will be located over the basement excavations. Therefore the most likely footing system would be conventional pad footings excavated at basement level. For uniformity of support, the footings should all be founded on the shale bedrock, unless allowance is made for differential movements. For pad footings within the basement excavations, preliminary design may be based on an allowable bearing pressure of 1,000kPa, provided low to medium strength shale is encountered at the appropriate founding depth. Higher bearing pressures would be possible in the better quality shale and deeper basement excavations. Alternatively, where the basement excavation is relatively shallow, bored pier footings may be required for uniformity of support. Some localised groundwater flows may occur into the bored piers and may require either use of dewatering equipment or, alternatively, placement of concrete using tremie techniques.

For any relatively small structures founded on the residual clay soils, high level footings may be adopted. Such footings should be in accordance with the requirements of a Class H site in accordance with AS2870. Stiffened raft slabs or even pier and beam construction may be preferred. It is noted that the effect of existing large trees would also have to be taken into account and may result in a Class P classification at specific areas of the site.

4.8 Basement Floor Slabs and Drainage

As noted above, subsurface drains would be provided around the perimeter basement walls. In addition, a grid of subsurface drains would be required over the basement floor area. Flows to these drains would be relatively minor and should be easily handled using either gravity discharge or conventional pump sumps. The adoption of a tanked basement design would not be normal for these type of site conditions. As discussed in Section 4.4 above, the disposal of any groundwater seepage would not have any significant effect on the drainage gully. Some localised lowering of groundwater may occur on the high side of excavations. As the groundwater levels are anticipated to be within the weathered shale bedrock, we do not anticipate that this would have a significant effect in terms of ground settlements. Nonetheless, some settlement may arise from a 'drying out' of adjacent ground, including the reactive clay soil.

The basement floor slabs would usually be isolated from the walls and column footings. Floor slabs should be provided with a subbase of durable, well graded material, such as DGB20 or similar. Basement floor slab joints should be capable of resisting shear but not bending.

4.9 Pavement Design

The residual clay soils are likely to provide a relatively poor subgrade with design CBR values typically of 2% to 3%. As predominantly residential traffic is anticipated, this should not have a major effect.

As the residual clay subgrade deteriorates rapidly in strength when exposed to moisture, the use of capping layers of good quality granular materials such as crushed concrete are commonly used to provide good all-weather platform during construction. We anticipate that the main delivery driveways would also be subject to some heavier vehicle loads, such as due to removals, vans, etc.

From the preliminary design layout provided, we do not anticipate that significant fill batters will be required. However, if site levels do require the formation of fill batters, then properly engineered fill will be required. It is noted that the residual clay soils and upper weathered shale form a poor quality fill material, such that careful design and construction supervision is required. If site won materials are used as structural fill, compaction should be to between 98% and 102% of Standard Maximum Dry Density (SMDD) at moisture contents within 2% of Standard Optimum Moisture Content (SOMC). Consideration should be given to the use of select imported granular fill materials such as crushed sandstone.

Reference should also be made to AS3796 for further guidance and appropriate testing to at least Level 2.

4.10 Further Investigation

Further detailed investigation using boreholes would be required to confirm subsurface conditions and appropriate design recommendations for retaining wall and footing design. To optimise these design requirements, consideration should be given to adopting diamond cored boreholes. Typical borehole spacing would be about 30m to 40m.

For basement excavations close to the adjoining rail corridor, consideration should be given to early discussions with the rail authorities as they frequently have requirements for submissions of designs for their approval. This may require further detailed investigation and design including detailed computer modelling of the retention system to predict any likely affects on the rail corridor.

We would be pleased to provide a further detailed scope of investigations if requested.

5 GENERAL COMMENTS

This report provides preliminary advice on geotechnical aspects for the proposed civil and structural design. Further detailed geotechnical investigations will be required to confirm the preliminary advice.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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 Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

Bruce F. Walker

B F WALKER
Principal

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

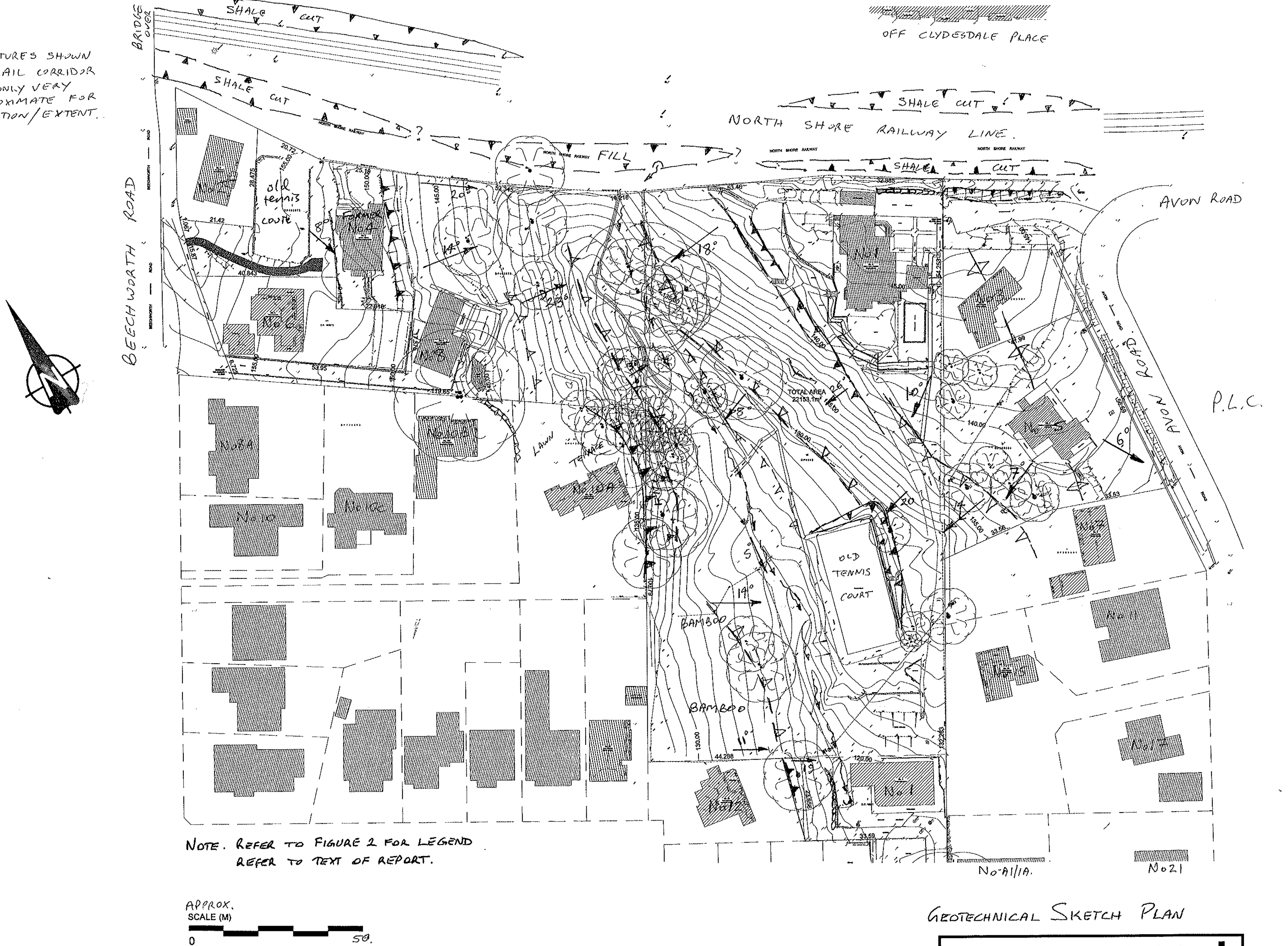
TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A Overall Instability of Existing Natural Hillside Slope	B Instability of Railway Fill Embankment	C Instability of Existing Fill Batter on North-West Side (No 10A)	D Instability of Possible Cut Batters at 1V:2H	E Instability of Proposed Retaining Walls
Assessed Likelihood	$<10^{-5}$ pa Say, RARE to BARELY CREDIBLE	$\approx 10^{-4}$pa UNLIKELY	$\approx 10^{-3}$pa POSSIBLE	$\approx 10^{-5}$pa RARE	10^{-5}pa to 10^{-6}pa RARE to BARELY CREDIBLE
Assessed Consequences	Ranges from localised impact on landscaping to impacting on development. MINOR to MAJOR	May impact part of development, if forms flow slide, more likely to be located in gully area and therefore would not impact development. INSIGNIFICANT to MEDIUM	Will not impact proposed development, only impacts landscaped areas. INSIGNIFICANT	Only localised impact on proposed development. MINOR to MEDIUM	If occurs, may require strengthening of proposed walls or stabilisation by drainage. MINOR to MEDIUM
Risk	VERY LOW to LOW	VERY LOW to LOW	VERY LOW	VERY LOW to LOW	VERY LOW to LOW
Comments	Development to be founded on rock and will therefore have reduced vulnerability.	If occurs, may form a flow slide, or may only be a slump/ debris slide. Run-out distance estimated from 3m to 20m.	Only considers property within subject site. Assume dwelling at No 10A would not be affected due to founding on piles.	No cuts currently proposed without support by retaining wall.	Basement walls will be properly engineered.

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A Overall Instability of Existing Natural Hillside Slope	B Instability of Railway Fill Embankment	C Instability of Existing Fill Batter on North-West Side (No 10A)	D Instability of Possible Cut Batters at 1V:2H	E Instability of Proposed Retaining Walls
Assessed Likelihood	RARE to BARELY CREDIBLE	UNLIKELY	POSSIBLE	RARE	RARE to BARELY CREDIBLE
Indicative Annual Probability (pa)	Say 10^{-5}	10^{-4}	10^{-3}	10^{-5}	Say, 10^{-5}
Person at Risk	a) Person walking regularly within landscaped areas. b) Persons accessing carparking.	a) Person walking regularly within landscaped areas. b) Persons accessing carparking.	a) Person walking regularly within landscaped area.	a) Person walking regularly within landscaped area.	b) Persons accessing carparking.
Number of Persons Considered	a) One – person most at risk. b) One – person most at risk.	a) One – persons most at risk. b) One – person most at risk.	a) One – person most at risk.	a) One – person most at risk.	b) One – person most at risk.
Duration of Use of Area (Temporal Probability, Part A)	a) 0.5 hour per day = 0.02 b) 10 mins x 4 times per day = 0.03	a) 0.5 hour per day = 0.02 b) 10 mins x 4 times per day = 0.03	a) 10 mins per day = 0.007	a) 10 mins per day = 0.007	b) 10 mins x 4 times per day = 0.03
Probability of being within area affected when event occurs (Temporal Probability, Part B)	a) Say, 0.1 b) Say, 0.1	a) Say, 0.1 b) Say, 0.1	a) Say, 0.1	a) Say, 0.1	b) Say, 0.1
Probability of Not Evacuating Area Affected	a) May not have warning, 1.0 b) May have warning from cracking, 0.5	a) May not have warning, 1.0 b) May have warning from cracking, 0.5	a) Unlikely to have warning, 1.0	a) May have warning, 0.5	b) May have warning from cracking, 0.5
Vulnerability to Life if Failure Occurs Whilst Person Present	a) Unlikely to be buried, 0.1 b) Unlikely to be buried; building unlikely to collapse, 0.05	a) May be buried, 0.5 b) Unlikely to be buried; building unlikely to collapse, 0.05	a) May be buried, 0.5	a) Unlikely to be buried, 0.1	b) Unlikely to be buried; building unlikely to collapse, 0.05
Risk for Person Most at Risk	a) $2x10^{-9}$ b) $7.5x10^{-10}$	a) 10^{-7} b) $7.5x10^{-9}$	a) $3.5x10^{-7}$	a) $3.5x10^{-10}$	b) $7.5x10^{-10}$
Combined Risk for Person Most at Risk	About $5x10^{-7}$				

NOTE: FEATURES SHOWN IN RAIL CORRIDOR ARE ONLY VERY APPROXIMATE FOR LOCATION/EXTENT.



Note. REFER TO FIGURE 2 FOR LEGEND
REFER TO TEXT OF REPORT.

GEOTECHNICAL SKETCH PLAN

Jeffery & Katauskas Pty Ltd

Report No. 23513W Figure No. 1

TOPOGRAPHY

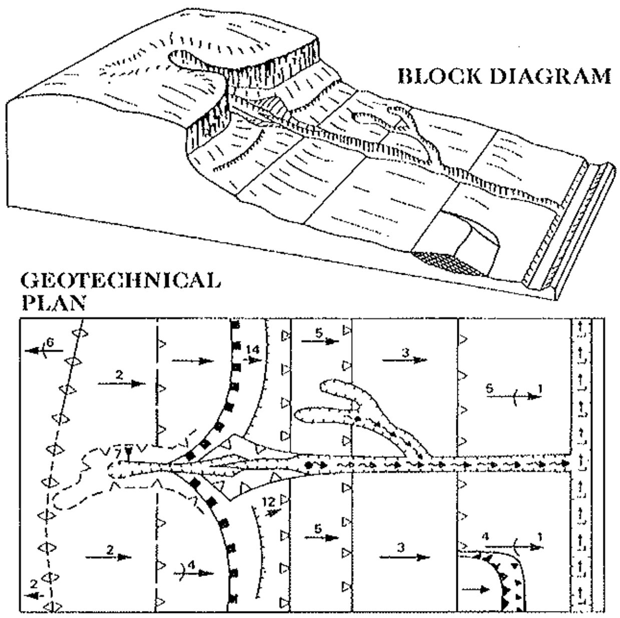
Symbol *Ground Profile*

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

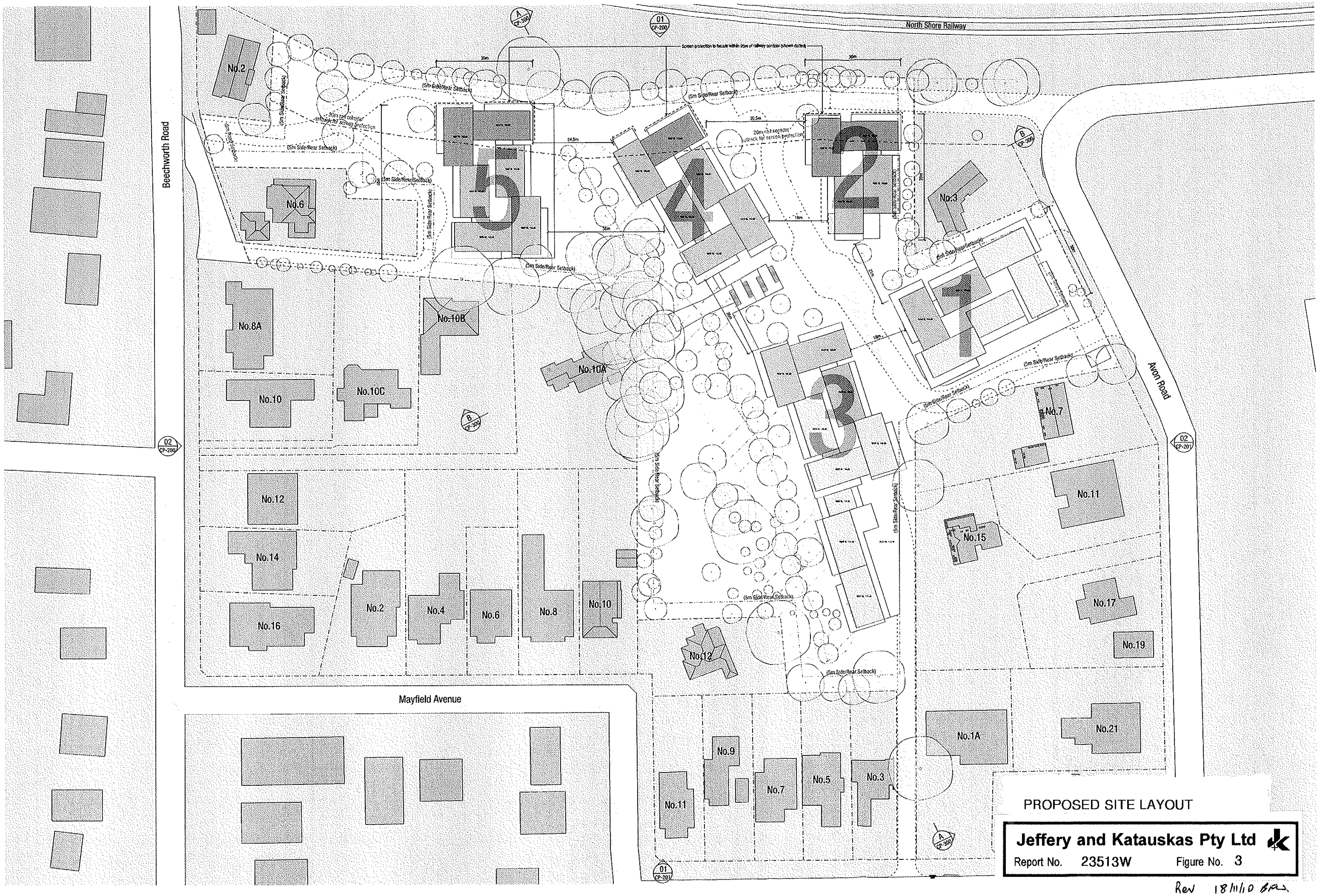
EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



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

GEOTECHNICAL MAPPING SYMBOLS

Jeffery and Katauskas Pty Ltd
Report No. 23513W Figure No. 2



PROPOSED SITE LAYOUT

Jeffery and Katauskas Pty Ltd

Report No. 23513W Figure No. 3

Rev 18/11/10

revision	date	description	by	verification	initial	signature	date
A	7 DEC 2009	CONCEPT PLAN - ENVIRONMENTAL ASSESSMENT	SC, MO	checked by	SC		19.11.09
				approved by	DS		19.11.09

note: Do not scale from drawings. All dimensions to be checked on site before commencement of work. All discrepancies to be brought to the attention of the Architect. This drawing is copyright and the property of the author, and must not be retained, copied or used without the express authority of Ancher Mortlock Woolley Pty. Ltd.

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project: EA - CONCEPT PLAN APPLICATION AVON ROAD, PYMBLE

client: Jim Neale

drawing: SITE CONCEPT PLAN

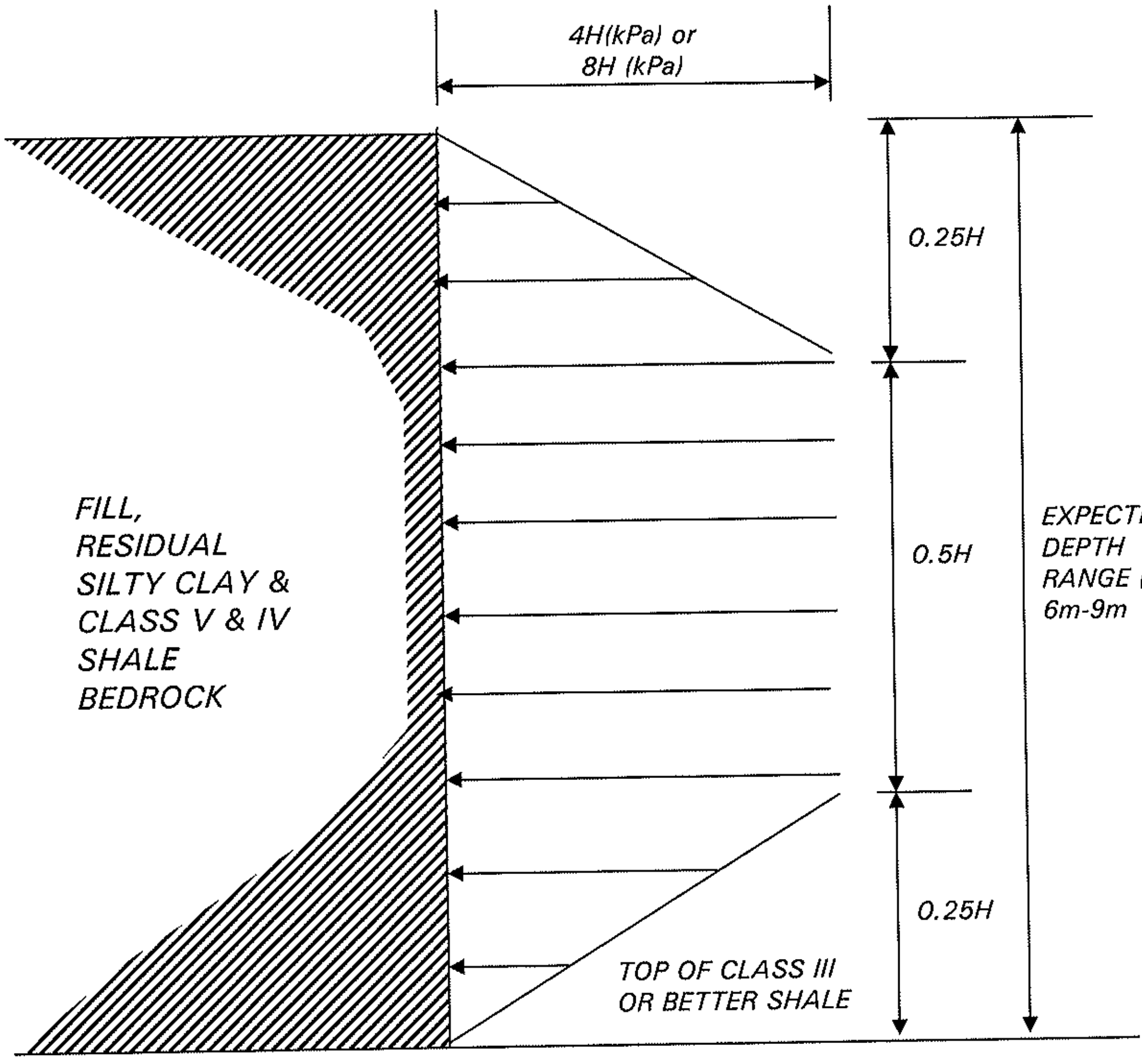
scale 1:500@A1, 1:1000@A3

drawn by SC,MO

project no 0909

drawing no CP-100

revision A

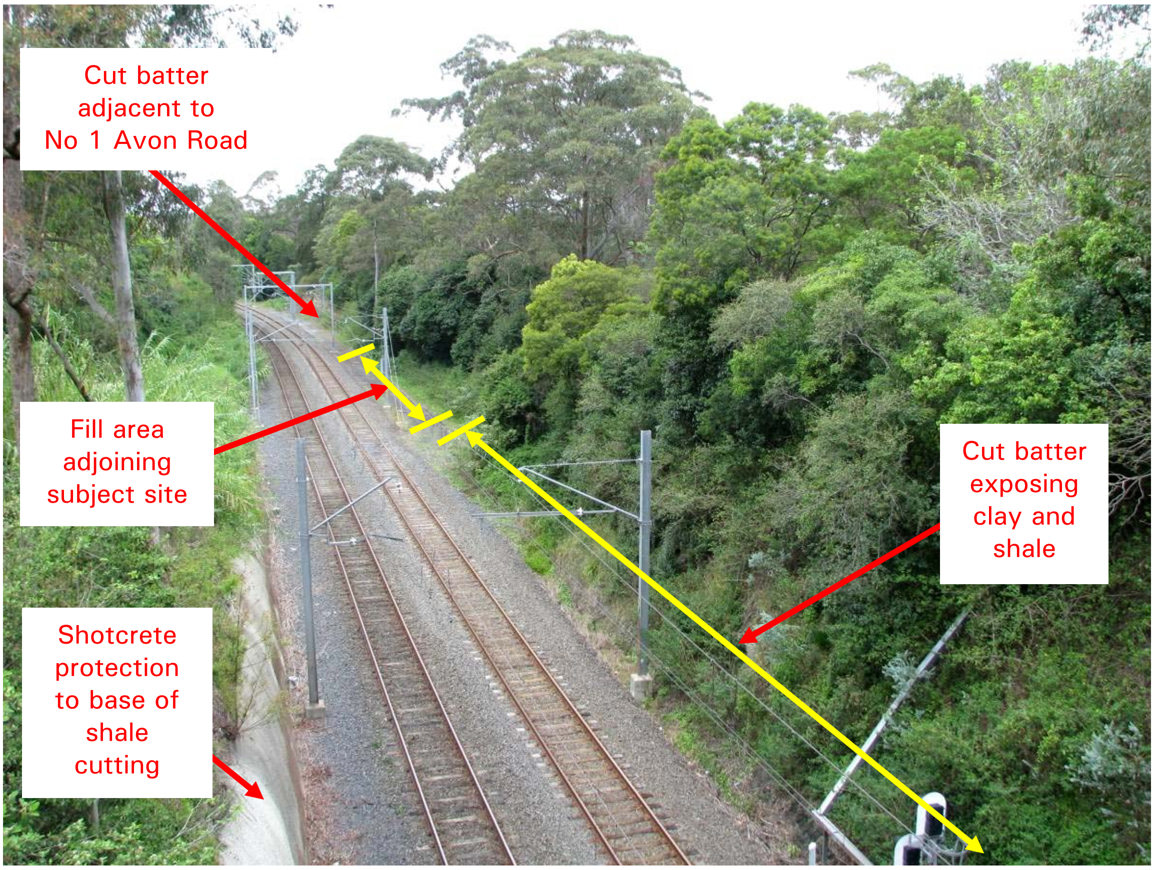


NOTES:

1. USE 4H WHERE THERE ARE NO MOVEMENT SENSITIVE STRUCTURES WITHIN 2H FROM THE LINE OF BASEMENT WALLS.
2. USE 8H WHERE THERE ARE MOVEMENT SENSITIVE STRUCTURES WITHIN 2H FROM THE LINE OF BASEMENT WALLS.
3. SURCHARGE AND GROUNDWATER PRESSURES MUST BE ADDED TO THE ABOVE IF APPLICABLE.
4. REFER TO TEXT OF REPORT

RECOMMENDED PRELIMINARY DESIGN PRESSURES FOR ANCHORED OR PROPPED RETAINING WALLS

Jeffery & Katauskas Pty Ltd	
Report No. 23513W	Figure No. 4



PHOTOGRAPH 1: View to south-east from Beechworth Road bridge along North Shore Railway line.

PHOTOGRAPH 2: View of cut batter on south-eastern side of Beechworth Road bridge, showing shale exposed at base of cutting.

PHOTOGRAPH 3: View of shale exposed in North Shore Railway line cutting on north-eastern side, adjacent to No 1 Avon Road.

PHOTOGRAPH 4: Possible fill batter adjoining No 10A Beechworth Road.

PHOTOGRAPH 5: view of terraced garden walls at north-eastern corner of the disused tennis court on No 1 Avon Road.

APPENDIX A

LANDSLIDE RISK

MANAGEMENT

TERMINOLOGY

APPENDIX A
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.

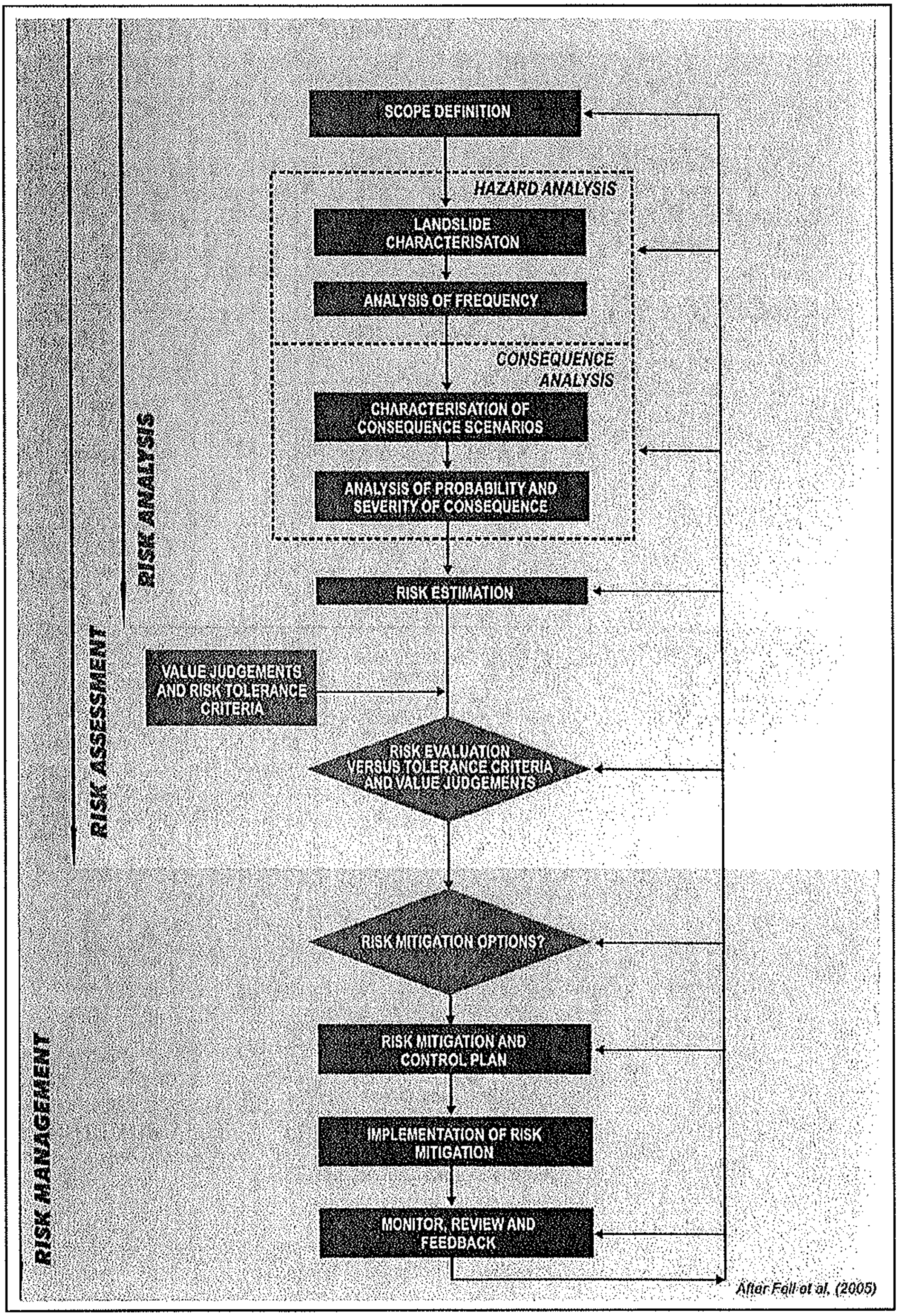


FIGURE A1: Flowchart for Landslide Risk Management.

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: Indicative Value	Approximate Annual Probability: Notional Boundary	Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
10^{-1}		10 years		The event is expected to occur over the design life.	ALMOST CERTAIN	A
	5×10^{-2}		20 years			
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
	5×10^{-3}		200 years			
10^{-3}		1000 years		The event could occur under adverse conditions over the design life.	POSSIBLE	C
	5×10^{-4}		2000 years			
10^{-4}		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
	5×10^{-5}		20,000 years			
10^{-5}		100,000 years		The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
	5×10^{-6}		200,000 years			
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: Indicative Value	Approximate Cost of Damage: Notional Boundary	Description	Descriptor	Level
200%		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
	100%			
60%		Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
	40%			
20%		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
	10%			
5%		Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
	1%			
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. **<u>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.</u>**

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.

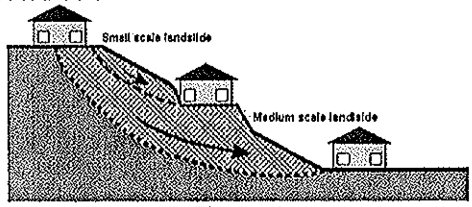


Figure 1

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.

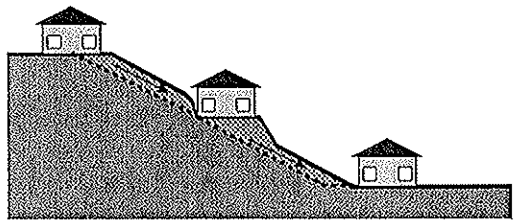


Figure 2

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.

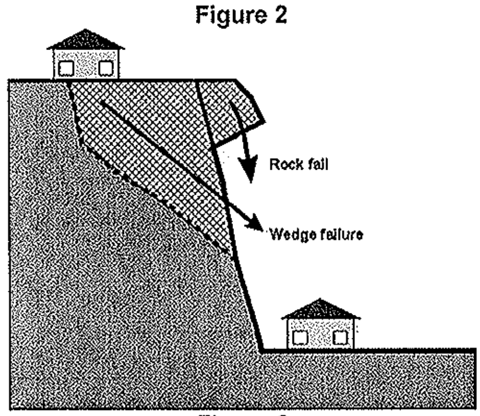


Figure 3

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.

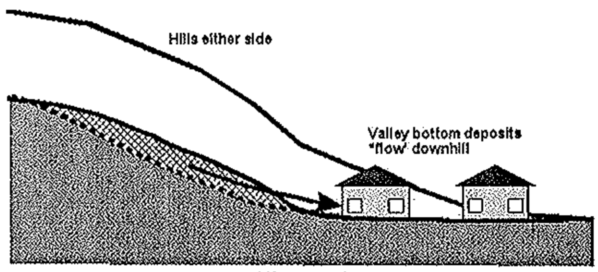


Figure 4

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. If you have any concern that you could be dealing with a landslide hazard that your local council is not aware of you should seek advice from a geotechnical practitioner.

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 inevitably lacks 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 perhaps temporary loss of use. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

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.

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", "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. This may be lower than you feel is reasonable for your block but it is, nonetheless, a pre-requisite for development. Reasons for this include the fact that a landslide on your block may pose a risk to neighbours and passers-by and that , should you sell, subsequent owners of the block may be more risk averse than you.

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 LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
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- GeoGuide LR6 - Retaining Walls
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
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Jeffery and Katauskas Pty Ltd

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ABN 17 003 550 801

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

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

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

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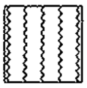
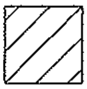
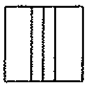
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL


FILL


TOPSOIL

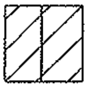

CLAY (CL, CH)


SILT (ML, MH)


SAND (SP, SW)

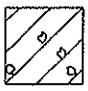

GRAVEL (GP, GW)


SANDY CLAY (CL, CH)

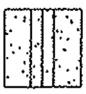

SILTY CLAY (CL, CH)


CLAYEY SAND (SC)


SILTY SAND (SM)


GRAVELLY CLAY (CL, CH)

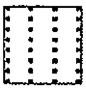

CLAYEY GRAVEL (GC)


SANDY SILT (ML)


PEAT AND ORGANIC SOILS

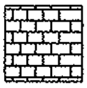
ROCK


CONGLOMERATE


SANDSTONE


SHALE

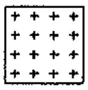

SILTSTONE, MUDSTONE, CLAYSTONE


LIMESTONE


PHYLLITE, SCHIST


TUFF

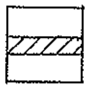

GRANITE, GABBRO

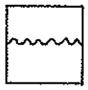

DOLERITE, DIORITE

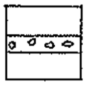

BASALT, ANDESITE

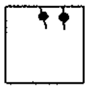

QUARTZITE

DEFECTS AND INCLUSIONS


CLAY SEAM


SHEARED OR CRUSHED SEAM


BRECCIATED OR SHATTERED SEAM/ZONE


IRONSTONE GRAVEL


ORGANIC MATERIAL

OTHER MATERIALS


CONCRETE


BITUMINOUS CONCRETE, COAL


COLLUVIUM

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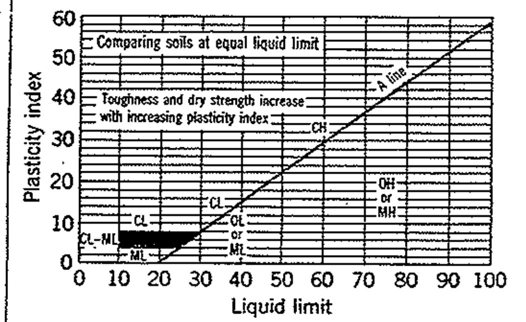
UNIFIED SOIL CLASSIFICATION TABLE

Major divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits below "A" line with *PI* greater than 7

Major divisions		Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	Example: *Clayey silt*, brown: slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil 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 screening.
	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. 'R' as noted below.
	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 degree 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 (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless 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 less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

Ref: Standard Sheets/Log Symbols
November 2007

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
		0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
		0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
		0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
		1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
		3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
		10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	