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REPORT

TO

HAMMOND CARE PTY LTD

ON

PRELIMINARY GEOTECHNICAL ASSESSMENT

AT

97 TO 115 RIVER ROAD, GREENWICH HOSPITAL, GREENWICH, NSW

19 February 2010

Ref: 23789ZRrpt

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1 INTRODUCTION

This report presents the results of our geotechnical assessment of Greenwich Hospital, 97 to 115 River Road, Greenwich, NSW. The assessment was commissioned by Mr Peter Hamilton of Hammond Care Pty Ltd by signed 'Acceptance of Proposal Form' dated 10 February 2010 in accordance with our proposal (Ref: P32005ZR) dated 9 February 2010.

The site was inspected by the undersigned on 15 February 2010, in order to assess the existing stability of the site, assess current levels of risk to both life and property and to provide recommendations regarding the maintenance of the site and preliminary comments and recommendations in relation to a proposed new development at the site.

2 ASSESSMENT PROCEDURE

The subject site was approximately 130m wide (north-south) BY 250m long (east-west) and comprised the grounds of Greenwich Hospital.

The assessment was completed by an Associate level engineering geologist and comprised a detailed walkover inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. However, we note that in some areas our observations were restricted by thick vegetative cover and/or steep slopes.

Any identified potentially unstable features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the site. The attached Appendix A1 defines the terminology adopted for the risk assessment together with a flow chart illustrating the Risk Management Process based on the guidelines given in AGS 2007(c) (Reference 1).

A summary of our observations are presented in Section 3 below. Our specific recommendations regarding proposed stabilisation measures are discussed in Section 6, following our risk assessment.

The attached Figure 1 presents a geotechnical sketch plan showing the principal geotechnical features present at the site and is based on the provided survey plan (Survey reference 2916 dated 28 March 1994) prepared by K.J. Morrow & R.W. Young Pty Ltd. Additional features on Figure 1 have been based on hand held tape measure, inclinometer and compass techniques. Should any of the features be critical to the proposed stabilisation measures, we recommend they be located more accurately using instrument survey techniques. Figure 2 presents an explanation of geotechnical mapping symbols. Figure 3 presents a General Site Plan indicating the locations of existing hospital buildings. Plates 1 to 3 provide a photographic record of the site and are presented in the attached photographic portfolio.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figures 1 and 3 and the photographic portfolio (Plates 1 to 3). For descriptive purposes, River Road has been assumed to be orientated east-west.

- The site is located within undulating terrain towards the crest of a hillside that slopes and steps down to the south, with localised slopes down to the east and west defining gully features orientated approximately north-south.
- The subject site comprised the grounds of Greenwich Hospital and had northern and eastern street frontages onto River Road and St. Vincents Road, respectively. The southern site boundary and the northern portion of the western site boundary

were lined by the yard areas of residences. The south-western portion of the site boundary was lined by Gore Creek Reserve.

- At the time of the assessment, the northern-central portion of the site was occupied by one to five level (typically one and two level) brick (occassionally fibro) hospital buildings, namely the Main and North Hospital Wings, Blue Gum Lodge, Riverglen Unit, Archinal House and the Service and Maintenace Wings. In this regard, we note that the Riverglen Unit building appears to have been constructed since preparation of the provided survey plan. The central-southern portion of the site was occupied by the two storey brick Pallister House. Based on a cursory inspection, the hospital buildings generally appeared to be in good external condition although some cracking of the rendered walls supporting the verandah over the south-eastern corner of Pallister House was evident. See Plate 1.

- The hospital buildings were connected by bitumen and aspahltic concrete (AC) surfaced driveways, access roads and footpaths. Grassed surfaced and vegetated areas covered much of the remainder of the site with the exception of the bitumen and gravel surfaced car park areas adjacent to the south-western, southern, eastern and north-western ends of the Main and North Hospital Wings. Based on a cursory inspection, the paved surfaces were in variable condition. The AC paved surfaces were in good condition. The bitumen paved surfaces were generally in poor condition and contained a number of diagonal and crocodile cracking of hairline to 10mm width and with numerous signs of previous surface repair and occasional pot-holes evident.

- The site topography was characterised by a relatively flat central and north-eastern portion that extended north-south across the site with approximate surface reduced levels (RLs) ranging between R46m and RL50m. Site surface levels generally stepped and sloped down to the south-east, south, west and south-west from the higher elevation area with approximate minimum surface levels of RL33m, RL32m and RL30m, respectively. The slopes and steps were formed by fill batter slopes, sandstone bedrock outcrops and retaining walls. The building platforms appear to have exploited relatively flat elevated areas with some localised cut and fill earthworks to extend the building platforms and to create parking and landscape areas.

- The retaining walls within the site were typically less than 1.5m high and of sandstone masonry, stacked sandstone, concrete and concrete segmental block construction. Generally, these walls were in good condition, athough some walls below the western side of Archinal House contained hairline to 30mm wide cracks. The larger crack widths appeared to be associated with root growth. The more significant retaining structures comprised:
  - Sandstone masonry walls of maximum 3.5m and 3m heights which supported areas lining the eastern side of the driveway (below the Main Hospital Wing – Extension) and the north-western and western side of Pallister House, respectively and had faces sloping at about 35° to 40°. The walls were generally in reasonably good condition with the exception of occassional missing or displaced blocks (below the Main Hospital Wing – Extension). However our observations were limited as signifcant lengths of the walls were overgrown. See Plate 1.
  - A concrete crib wall supporting the western side of the car park adjacent to the Service Wing which was a maximum of about 2.6m high and with a face that sloped down to the west at about 70°. The wall appeared to be in reasonably good condition with some erosion of the sand backfill (which included gravel to small cobble sized inclusions of brick, sandstone and asbestos sheeting). See Plate 1.
- The fill batter slopes over the western side of the site sloped down to the west, south and south-west and over the eastern side of the site sloped down to the south east. Typically the batters sloped at about 35° with locally flatter batter

slopes. The steeper batter slopes (up to 45°) typically had uneven faces and the trees on the batter slopes generally had curved bases and/or the trunks leaned 'back'. No tension cracks were noted along the crest areas of the fill batters. However, a section of the fence line at the crest of the fill batter to the west of the Main Hospital Wing was misalgned and leaning and the crest of the fill batter over the eastern side of the site contained a number of arcuate features which may well represent localised areas of previous instability. We note that our observations were limted by thick vegetation and tree cover. See Plates 2 and 3.

- The vertical faced sandstone outcrops within the site ranged between about 0.5m and 3m height and were located to the south-west and east of Pallister House and over the north-eastern corner of the site. See Plates 2 and 3.

- Site surface levels were generally similar across the northern and eastern site boundaries. However, a sandstone outcrop (about 0.5mn high) was located at the southern end of the eastern site boundary. In addition, immediately to the south of the eastern driveway entry, the chain/metal fence was leaning and bulging; the base of the fence was supporting sandstone and brick rubble fill. See Plate 2.

- The southern site boundary was lined by residential properties. Brick or rendered one to three level buildings lined or were set-back about 5m from the southern site boundary. Based on a cursory inspection from within the site the neighbouring buildings generally appeared to be in good external condition within only ocassional hairline to 2mm wide cracking evident. Site surface levels were generally similar across the central and eastern portions of the boundary although the sandstone outcrop to the east of Pallister House extended south-west into the neighbouring properties. The central-western portion of the southern site boundary was lined by brick and timber retaining walls (about 1m high) which supported the subject site and appeared to be in reasonable condition, based on limted observations (due to access restrictions and vegetative cover). The western end of the southern site boundary was lined by an abandoned pool. The southern and western sides of the pool area were supported by brick walls of about 1.5m maximum height. The south-western corner of the wall was in poor condition; what appeared to be a previously collapsed section of wall (about 0.5m wide and 1m high) was evident. See Plate 1.

- The northern and central portion of the western site boundary was lined by yard and pool areas of residences; the toe of one of the above mentioned fill batter slopes extended to this portion of the western site boundary. Two and three level rendered and timber houses were set-back at least 1m from the site boundary; the northern end of the western site boundary was lined by a concrete wall (maximum height about 1.5m) which supported the subject site. Based on a cursory inspection from within the site the neighbourng buildings and structures appeared to be in good external condition. The southern portion of the western site boundary was lined by a sub-vertical cliff face (estimated to be about 25m high) which was thickly vegetated and appeared to have a stepped face profile. The toe of the cliff face was lined by Gore Creek which flowed north to south adjacent to the flat grass surfaced Gore Creek Reserve. The crest of the cliff face comprised a flat bench; the toe of the fill batter which extended down from the south-western portion of the site was typically set-back between about 1m and 3m from the crsst of the cliff face. However, access restrictions, thick vegetation and safety considerations prevented further inspection of the cliff face crest. See Plate 3.

- The day of the site assessment followed a recent period of heavy and prolonged rainfall. It appeared from surface erosion traces over the fill batter slope above the north-eastern corner of the abandoned pool that surface run-off from the nearby car park surface discharges down the fill batter slope. At the time of the assessment significant quantities of water were discharging from a stormwater pipe (about 0.7m diameter) within a sandstone masonry headwall located at the crest of the northern end of the cliff face lining the southern half of the western site boundary. The discharged water cascaded down the cliff face to Gore Creek below.

4 ASSESSED SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. Based on our site observations we have assessed the likely subsurface conditions within the site to comprise:

- A limited thickness of topsoil over landscaped areas.

- Fill of variable composition and compaction forming localised flat platforms across the site. The fill is likely to have been locally sourced from bulk excavations and demolition rubble during the course of the development of the site. The backfill to the concrete crib wall supporting the western side of the car park adjacent to the Service Wing generally appeared to comprise sand with gravel to small cobble sized inclusions of brick, sandstone and asbestos sheeting. It appeared from surface erosion traces over the fill batter slope above the north-eastern corner of the abandoned pool that the fill in this area comprised sand with concrete inclusions.

- The natural soils (where encountered) are expected to comprise a limited thickness of sandy and clayey residual soils.

- The weathered sandstone bedrock revealed in various outcrops and cliff lines was assessed to be distinctly to slightly weathered and generally of medium strength. Defects within the bedrock comprised sub-horizontal bedding partings, cross bedding planes sloping down to the north-east at between about 15° and 20°, and sub-vertical defect planes orientated approximately east-west, north-east to south-west and north-west to south-east. In this regard, we note that the cliff face lining the southern portion of the western site boundary was orientated approximately 300° and, where observations were possible, appeared to be controlled by sub-vertical defect planes. The defects within the outcrops to the south-west and east of Pallister House were typically open between about 0.15mm and 0.25m and a clay filled defect was 1m wide. The outcrop to the south-west of Pallister House contained an undercut feature approximately 4m long, 1m high and extended back a maximum horizontal distance of approximately 3m. Occasional detached sandstone blocks (typically 1m x 1m x 0.5m size) were located along the toe of the outcrop to the east of Pallister House.

- Within the site, traces of recent seepage were evident over some of the sandstone bedrock outcrop faces. We reiterate that the site assessment followed a recent period of heavy and prolonged rainfall.

5 GEOTECHNICAL ASSESSMENT

5.1 Potential Landslide Hazards

The site is located towards the crest of hillside with the regional topography characterised by a stepped slope profile formed by sub-vertical sandstone cliff lines with an intermittent thin cover of natural soils. The hillside slope steps and slopes down to the south-west, south and south-east. The stepped surface profile within the site generally appears to be controlled by the underlying stepped bedrock surface profile associated with the generally east-west, north-west to south-east and north-east to south-west orientated cliff lines and outcrops. As noted in Section 4, above, occasional undercut features were noted within a sandstone bedrock outcrop to the south-west of Pallister House.

As described in Section 3, above, it appears that the hospital has been constructed on the bedrock surface over the relatively flat central and north-eastern portion of the site. Some localised cut and fill earthworks have also been formed to extend the building platforms and to create parking and landscape areas.

The slopes and steps have been formed by fill batter slopes, sandstone bedrock outcrops and cut faces, and retaining walls. The retaining walls support cut faces and selected areas of fill. The sub-vertical sandstone bedrock cut faces and outcrops remain unsupported.

It is evident that the topography of cliff lines and outcrops have been influenced by the approximately north-west to south-east, north-east to south-west and east-west orientated planar joint sets identified during our inspection. The presence of detached blocks along the toe of an outcrop to the south-east of Pallister House indicates that previous instability of bedrock faces has been controlled by sub-vertical planar defects within the bedrock. The differential weathering and/or erosion of relatively weaker extremely weathered seams/clay bands in rock faces would be a likely mechanism of collapse due to undercutting of more competent sandstone bands above. The presence of water within defects and/or the 'jacking' action of tree roots also has the potential to de-stabilise individual blocks.

Crucial to these processes is the rate at which they are occurring. Little evidence is available on the overall rates of occurrence of these forms of instability and the resultant rate of recession of the cut face. Recent published research into erosion of sandstone foreshore cliff faces suggests erosion rates of the order of 1mm per year. However, this is likely to be an over-estimation as the subject rock face will not be subjected to wave erosion or other localised erosion caused by concentrated seepages.

The fill batters slope down to the west, south-west, south and south-east and are typically moderately steep and steep (batter slope angles of the order of 35°) and show signs of on-going near surface creep such as:

- Uneven slope surfaces,
- Curved tree bases,
- Leaning trees,
- A section of uneven fence line at the crest of the fill batter slope to the west of the Main Hospital Wing, and
- Arcuate features lining the crest of the fill batter over the eastern side of the site which may well represent localised areas of previous instability.

Sandstone bedrock is exposed at surface or has been assessed to be generally present at shallow depth and the site appears to be relatively well drained.

Existing retaining walls within the site were in variable but generally good condition. Collapse of walls supporting fill areas and cut faces have the potential to cause relatively extensive damage to adjacent structures. Collapse of the generally low height landscape walls within the site would be relatively localised.

It is important to be mindful that rock falls, soil slumps etc can occur at anytime and it would be difficult or impossible to predict when the identified potential hazards will occur. Also, we cannot predict when an extreme or unusual event may occur (such as an earthquake or 1 in 100 year rainfall event etc) and what impact it would have on the stability of the identified potential hazards. Also, the design life and age of existing retaining walls within the site is pertinent with regard to their on-going performance.

Based on the above, the site may be regarded as stable overall but ongoing down-slope creep of fill batters is expected. In addition, over time collapse of localised wedges or blocks of sandstone, collapse of cliff line/outcrop overhangs and poor condition retaining walls can be expected. On this basis, the potential landslide hazards for the site are associated with potential instability of:

i. Undercut feature within sandstone outcrops and cliff lines,

ii. Wedges and blocks of sandstone within sandstone outcrops and cliff lines,

iii. Existing retaining walls, and

iv. Existing fill batters.

In our opinion, the elements most at risk are:

- Patients, visitors, external maintenance personnel and staff members.
- Vehicles and their occupants.
- Parked vehicles.
- Existing buildings and structures.
- Utility infrastructure.

5.2 Risk Analysis

Our geotechnical assessment of the risk of instability is based on the methodology proposed by Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'* (Reference 1), the relevant site features illustrated on Figure 1, presented in the attached photographic portfolio (Plates 1 to 3) and described in Section 3.

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Based on the above, the qualitative risks to property have been determined.

The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A.

Table A indicates that the assessed risk to property varies between "Very Low", "Low" and "Moderate" under existing conditions which would be considered 'acceptable' and 'tolerable', respectively in accordance with the criteria given in Reference 1. With the recommendations outlined in Section 6 below implemented, the assessed risk to property would be reduced to at least "Low" which would be considered 'acceptable' in accordance with the criteria given in Reference 1.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal, vulnerability evacuation and spatial factors that have been adopted are given in the attached Table B together with the resulting risk calculation.

Our assessed risk to life for the person most at risk, under existing conditions ranges between about 2×10^{-5} and 4×10^{-10} and would be considered to be 'acceptable' in relation to the criteria given in Reference 1.

In general, our adopted temporal, vulnerability evacuation and spatial factors may be regarded as relatively high and as such tending to be conservative. Consequently, our calculated levels of risk to life may be regarded as being at the upper end of what may be reasonably expected.

5.3 Risk Assessment

It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners be made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, risk cannot be completely removed, only reduced, as removing risk is not currently scientifically achievable.

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

Our assessment of the probability of failure of existing structural elements such as retaining walls is based upon a visual appraisal of their type and condition at the time of our inspection together with their approximate age and an assumed design life of say 20 to 30 years. Where appropriate we identify the time period at which reassessment of their longevity seems warranted.

We provide below recommendations regarding stabilisation measures for the identified potential geotechnical hazards which, if adopted in full, would assist in reducing reduce risk to 'acceptable' levels. These recommendations form an integral part of the Landslide Risk Management Process. However, it is a matter for Hammond Care Pty Ltd how they wish to implement the stabilisation measures outlined below.

6 LANDSLIDE RISK MANAGEMENT

6.1 General Comments

Under existing conditions, our risk assessment has indicated that in relation to the criteria given in Reference 1 levels of risk to both life and property are at 'acceptable' and 'acceptable' or 'tolerable' levels, respectively.

However with regard to existing retaining structures (potential geotechnical hazard iii) although we have assumed that they have been engineer designed, we are unaware of their design lives. A design life of 20 to 30 years may be regarded as a reasonable assumption of the design life of retaining structures. We also note that in some instances (particularly the sandstone masonry walls below the western side of Archinal House) cracks assessed to be due to root growth were evident. We also note that we have no information regarding the design or construction of the retaining structures within the site.

With no stabilisation measures implemented, as time goes on the likelihood of instability will increase and consequently, over time, risk levels may be assumed to increase from 'acceptable' to 'tolerable' levels or from 'tolerable' to 'unacceptable' levels. We therefore provide and outline of what we consider to be an appropriate Landslide Risk Management Process which includes annual monitoring by site staff, assessment of retaining structures by a structural engineer, 5 yearly geotechnical assessments, checking of water carry pipelines for leaks and, in specific areas, repair of retaining walls. The results of the monitoring will assist in the future assessment of whether further stabilisation measures are required.

6.2 Monitoring

We recommend that the following areas of the site be inspected by staff members on an annual basis and after periods of heavy or prolonged rainfall:

- The crest areas of the fill batter slopes adjacent to the car parks over the western and south-western portions of the site. The inspection should check for signs of tension cracking within the car park paved surface, leaning trees or fences and/or areas of disturbed fill batter slope surface.

- The stormwater pipeline located over the western side of the site and lining a portion of the western site boundary. The inspection should check for visible signs of damaged and/or leaking pipes which could cause erosion of the fill batter slope and initiate slope failure.

- The sandstone masonry walls which support areas lining the eastern side of the driveway (below the Main Hospital Wing – Extension) and the north-western and western side of Pallister House.

- The concrete crib wall supporting the western side of the car park adjacent to the Service Wing.

- Unless repaired in accordance with the advice provided in Section 6.3, below:
  - The cracked sandstone masonry walls below the western side of Archinal House, and
  - The poor condition brick wall supporting the south-western corner of the abandoned pool area.

We also recommend that within the next 5 years a structural condition assessment of all the retaining walls within the site be undertaken.

Should the inspections by staff members reveal any signs of instability, such as outlined above and, in relation to the existing retaining walls, signs of fresh cracking, displaced blocks or crib elements, visibly leaning walls then further geotechnical advice should be immediately sought. The monitoring inspection must be formally documented and include the date of the inspection, any comments/observations and photographs to be provided. A copy must be provided to the geotechnical consultant for review.

In addition, on a 5 yearly basis, a detailed assessment of the site should be undertaken by an experienced engineering geologist/geotechnical engineer to assess current conditions with regard to the previous inspection and the on-going inspection monitoring reports.

6.2 Water Carrying Services

All existing surface (including roof) and subsurface drains, sewers and other water carrying pipelines must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than five yearly intervals, commencing within 12 months of issue of this report; including provision of a written report confirming scope of work completed and identifying any required remedial measures.

6.3 Retaining Wall Repairs

Consideration should be given to repair of the cracked sandstone masonry walls below the western side of Archinal House. Removal of large roots systems from the planter bed area supported by the wall and localised re-construction of the wall is recommended. The wall re-construction should be designed by a structural engineer, in accordance with the advice provided in Section 7.3.2.

Re-planting of the planter bed should incorporate plants with small root systems; advice from a landscape consultant should be sought.

The poor condition brick wall supporting the south-western corner of the abandoned pool area should be strengthened, re-constructed or removed. Further advice should be sought from a structural engineer and any wall strengthening or re-construction should be completed in accordance with the advice provided in Section 7.3.2.

7 COMMENTS AND RECOMMENDATIONS

7.1 Likely Proposed Development

We understand from information provided by Richard Smyth (Smyth Planning) that the following new developments within the site are being considered:

- Demolition of Blue Gum Lodge,
- Construction of say an 8 level building over 1 or 2 or 3 levels of basement car park over the central-eastern portion of the site in the vicinity of the Blue Gum Lodge and the car park area to the south. Access would be from the existing driveway that extends down to St. Vincents Road, and
- Re-landscaping of the fill batter slope over the south-western corner of the site which would likely include modification of the batter slope to form a stepped terraced profile.

We provide below preliminary comments and recommendations in relation to the proposed new development. However, we recommend that once further details are determined, a comprehensive geotechnical investigation be undertaken; further details are provided in Section 7.7, below.

7.2 Demolition and Excavation

All excavation work will need to be complemented by reference to the Code of Practice, 'Excavation Work', Cat. No 312 dated 31 March 2000 by WorkCover.

Demolition and excavation will need to be undertaken with care so as not to damage or de-stabilise any adjacent existing buildings and structures that will remain.

Based on the likely locations of the proposed development, it is unlikely that dilapidation surveys of nearby buildings and structures will be required. However, Council may require dilapidation surveys of the adjoining road pavements and footpaths. The owners should be asked to sign any dilapidation survey reports that are prepared and agree that they are a fair assessment of existing conditions, as these can then be used as a benchmark in assessing potential future damage claims (due to ground surface movements and/or vibration damage).

Following demolition and stripping of vegetation and root affected soils, we expect the excavations will extend through a shallow soil cover and into the weathered sandstone bedrock. Excavation through the soils and extremely low strength sandstone bedrock should be able to be completed using conventional earthmoving equipment such as tracked excavators. The excavation of low, or higher, strength sandstone is likely to require a rock breaker and/or or ripping tyne attachments to hydraulic excavators (say 20 tonne to 30 tonne size) or Dozers (say D9 size). Rock breakers may also be required for demolition of existing concrete footings and paved surfaces.

We note that rotary grinders and rock saws may also be used to create 'smooth' finishes on cut rock faces and aid in detailed rock excavation of footings, services trenches etc.

Care should be taken when using rock breakers so that ground vibrations do not adversely affect nearby structures or existing fill batter slopes. In this regard we recommend that a rock saw cut be provided around the excavation perimeter prior to using rock breakers. The base of the saw cut slot must be maintained below the adjacent bulk excavation level at all times. If there are concerns with regard to vibration damage of nearby existing hospital buildings, we recommend that periodic quantitative vibration monitoring of these buildings be undertaken while the rock hammers are being used to confirm that peak particle velocities fall within acceptable limits. We recommend that the peak particle velocities do not exceed 5mm/sec at the adjacent buildings; the peak particle velocity will need to be reduced to 3mm/sec adjacent to the heritage Pallister House. We note that these vibration limits will reduce the risk of vibration damage to the nearby buildings and structures. However, these vibrations may still result in discomfort to occupants of the buildings. If potentially damaging vibrations are occurring it will be necessary to use lower energy equipment such as smaller hammers or grinder attachments on hydraulic excavators. Alternatively grid-sawing techniques can be used to dampen ground vibrations.

The following procedures are recommended to reduce vibrations if rock hammers are used.

- Maintain a sharp moil.
- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Operate hammers one at a time in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with a competent supervisor who is aware of vibration risks, possible rock face instability issues etc. The contractor should be

provided with a copy of this report and have all appropriate statutory and public liability insurances.

Groundwater seepage is expected within the excavation at or below the contact between the soil profile and the sandstone bedrock below. However, concentrated flows along the surface or discrete defect planes within the sandstone bedrock may also occur, particularly after heavy or prolonged rainfall. We expect the inflows could be controlled by conventional sump and pump techniques and/or gravity drainage. Inspection and monitoring of groundwater seepage during bulk excavation is recommended, so that appropriate drainage may be detailed. Some soil loss may occur at the soil rock interface, especially after rain periods and sand bagging may be required to stabilise the toe of batter slopes through the soils.

7.3 Retention

7.3.1 Temporary and Permanent Cut Batter Slopes

Temporary batters through the soil profile may be excavated no steeper than 1 Vertical (V) in 1.5 Horizontal (H) (sandy soils) and 1V in 1H (clay soils and extremely weathered sandstone). Any permanent batters should be formed at no steeper than 1V in 2.5H and should be protected against erosion by means of rapid growing vegetation, shotcrete or similar.

Competent sandstone bedrock of low or higher strength may be cut vertically and depending on the excavation depth we recommend that a 0.3m to 1.5m wide berm be provided between the toe of the soil batter and the crest of the vertical rock face below. A geotechnical engineer/engineering geologist should however progressively inspect the rock face as excavation proceeds (at no greater than 1.5m height increments) to identify adverse defects and to propose stabilisation measures where necessary. Provision must be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

The presence of clay seams and/or weakly cemented (extremely weathered) seams within the sandstone bedrock may adversely affect the stability of the cut faces and will probably require shotcreting and bolting. If full height retaining walls are to be constructed for long term stability the shotcrete is unlikely to be required. Clay seams occurring in permanently exposed faces may also require 'dental' treatment.

We note the presence of planar defects recorded in the sandstone outcrops within the site. These defects (and others which may be encountered in the excavation) may form potentially unstable wedges that also require stabilisation during excavation. Stabilisation may take the form of rock bolts and/or shotcrete.

If rock bolts are used and extend below adjoining properties (believed to be unlikely), permission will be required from the respective property owners to allow their installation. The actual amount of stabilisation which will be required cannot be quantified at this stage and can only be determined at the time of excavation. Once the need for stabilisation of any particular stage has been identified, no further excavation below this zone should be completed in that area until the stabilisation has been completed.

Retaining walls may be founded on the top of the sandstone bedrock, which is expected to be encountered within the majority of the excavations. For retaining walls founded on top of the sandstone bedrock within the excavation face, lateral restraint may be provided by starter bars drilled and grouted to a depth of at least 0.5m into the sandstone bedrock. The starter bars should be installed at a downward angle into the rock and be provided with a vertical cogged length. If cross bedded units within the sandstone bedrock are identified and slope down into the excavation, then the starter bars may have to be extended to stabilise the potentially unstable cross bedded units. For long term corrosion considerations we recommend that all permanent starter bars be hot dipped galvanised. Alternatively, if the bedrock is in relatively poor condition and/or is encountered at or below bulk excavation level, then full height retaining walls to support the cut faces will be required.

7.3.2 Retaining Wall Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of basement retaining walls, any landscape retaining walls and for wall re-construction/repairs:

- We assume that the basement retaining walls will be propped by the structure and subsequently backfilled. We therefore recommend the use of a triangular lateral earth pressure distribution and an "at rest" earth pressure coefficient (k_o) of 0.55 for the soil and extremely weathered sandstone profile, assuming a horizontal backfill surface.
- Where some movements of retaining walls may be tolerated (such as landscaped areas), and for repair of existing poor condition walls, they may be designed for a coefficient of 'active' earth pressure, K_a, of 0.35 for the soil and extremely weathered sandstone profile, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the retained profile.
- Any surcharge affecting the walls (e.g. adjacent footings, traffic loads, construction loads, compaction stresses, sloping backfill surface etc) should be allowed for in the design, using the appropriate earth pressure coefficient from above.
- Complete and permanent drainage of ground behind the retaining walls must be provided. The subsurface drains must incorporate a geofabric (e.g. Bidim A34) to act as a filter against subsoil drainage.

- Lateral toe restraint of the retaining walls founded below bulk excavation level should be achieved by keying the wall footing into the underlying bedrock below the base of nearby footings or service trench excavations. An allowable lateral stress of 200kPa may be adopted for key design.
- Rock dowels, bolts or anchors should be designed for an allowable bond strength of 200kPa and installed into sandstone bedrock of at least low strength. Permanent rock dowels, bolts or anchors will need to be designed with due regard for long term corrosion resistance.

7.4 Footings

Excavations for the new building are expected to expose sandstone bedrock over the base of the excavation. We expect that retaining walls, load bearing walls and internal columns may be supported on pad or strip footings founded on sandstone bedrock exposed at bulk excavation level.

For uniformity of support, all footings should be founded within the underlying weathered sandstone bedrock. Footings socketted a nominal depth of 0.1m into weathered sandstone bedrock of at least low strength may be designed for an allowable bearing pressure of 1,000kPa; higher bearing pressures up to 3,500kPa may also be achievable. However, the geotechnical investigation outlined in Section 7.7 will need to be completed in order to confirm the allowable bearing pressure that may be adopted for footing design.

In addition, geotechnical inspection of the footing bases will be required and possibly spoon testing of footing bases where relatively high allowable bearing pressures are adopted. In addition, where any footings are located close to steps in the bulk excavation level or close to nearby lift pit and/or service trench excavations, then the sandstone below the nearby footing should also be inspected by a geotechnical engineer to check for the presence of any potentially unstable wedges of sandstone. Any such wedges will require stabilisation using permanent rock bolts or underpins.

All footings should be excavated, inspected and poured with minimal delay. All footings should be free from all loose or softened materials prior to pouring. If water ponds in the base of the footings it should first be pumped dry and then re-excavated to remove all loose and softened materials. If a delay in pouring is anticipated we recommend that a blinding layer of concrete be placed to protect the base of footing excavations in poor quality bedrock.

7.5 Subgrade Preparation and Engineered Fill

7.5.1 Subgrade Preparation

Prior to the placement of any fill or pavements over a soil subgrade the following site preparation should be undertaken:

- Strip all existing pavements, vegetation, topsoil or root affected zones.
- Proof roll the exposed subgrade with a minimum of eight passes of a five tonne minimum deadweight smooth drum vibratory roller. The purpose of proof rolling is to increase the near surface density of the subgrade and to identify any soft or unstable areas. All proof rolling should be conducted under the direction of an experienced earthworks superintendent or geotechnical engineer. Care should be taken when proof rolling under vibration if movement sensitive structures are located nearby. If transmitted vibrations are considered excessive, proof rolling should be completed using the static mode with no vibration.
- All soft or heaving areas identified during proof rolling should be excavated to a sound base and reinstated with engineered fill as described below.
- Any fill placed to raise site levels should also comprise engineered fill complying the specification outlined in Section 7.5.2, below.

7.5.2 Engineered Fill

Engineered fill should preferably comprise a well-graded, durable select granular material with a maximum particle size not exceeding 50mm and containing no organic or other deleterious inclusions. The sandstone being excavated from the site may be suitable for this purpose provided oversize material is removed or crushed. All fill should be compacted in layers no greater than 150mm loose thickness and should be compacted to at least 98% of Standard Maximum Dry Density (SMDD).

7.5.3 Compaction Control

Where fill is placed as engineered fill, testing should be undertaken to confirm that the target density is being achieved. The frequency of density tests should be at least one test per layer per $500m^2$, or three tests per visit, whichever requires the greater number of tests. We recommend that the density testing be in accordance with at least Level 2 testing as defined in AS3798-2007. The earthworks recommendations provided here should be complemented by reference to AS3798.

7.5.4 Re-Profiling Existing Fill Batter Slope

We note the proposal to re-profile the existing fill batter slope over the south-western corner of the site. In this regard we note the current visible signs of near surface creep of the fill batter slope. We concur with the intention to re-profile the existing fill batter slope as this will improve long term stability provided the following recommendations are adopted in the design:

- New permanent fill batter slopes should be no-steeper than 1V in 2.5H and planted with rapid growing vegetation. To assist with maintenance, consideration should be given to permanent slopes of 1V in 3H, where space permits.

- The re-profiling of the slope will require removal of vegetation. Such work in a moderately steeply sloping environment (and close to the crest of a sub-vertical cliff line will need to be completed with care and with appropriate rope and harness safety equipment. In addition, measures will need to be taken to control potential debris falling downslope over the edge of the cliff and possibly entering the creek line below. We recommend that a further geotechnical inspection be completed following vegetation clearance to check for additional signs of slope instability. The contractor should prepare a Safe Work Method Statement (SWMS) prior to completing this work. The SWMS must include but not be limited to proposed demolition and excavation techniques, the proposed demolition and excavation equipment, excavation sequencing, hold points and/or geotechnical inspections, work safety and slope monitoring procedures, contingency plans in case of slope instability. The geotechnical engineer should review and approve the SWMS.

- New landscape walls to form the proposed terraced profile should be designed in accordance with the advice provided in Section 7.3.2, above. In addition, the new landscape walls may be founded in the existing fill provided it has been prepared in accordance with the advice provided in Section 7.5.1. However, we note that due to access restrictions, the subgrade preparation will need to be completed with a hand held vibrating pate compactor. We note that care will also need to be exercised adjacent to the downslope edge of the fill benches so as not to cause localised instability.

- Provided the fill subgrade has been prepared in accordance with the above advice, the landscape wall footings may be designed for an allowable bearing pressure of 50kPa. In addition, the bench widths and footing embedment should be designed such that the landscape walls do not surcharge the landscape wall supporting the bench below.

7.6 Pavements, Basement Floor Slabs and Drainage

Prior to the placement of any pavements and slabs on ground, the comments and recommendations contained in the preceding Section 7.5 should be carefully followed.

Where used, concrete driveways should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or similar good quality, durable, fine crushed rock) compacted to at least 100% of SMDD. Concrete pavements should be designed with doweled or keyed joints to transfer shear forces but not bending moments.

At this stage, pavement subgrade conditions may comprise bedrock and/or sandy or clayey fill materials. Provided areas of soil subgrade have been prepared in accordance with the advice provided in section 7.5, we provide the following preliminary advice:

- An estimated CBR value of 5% may be adopted for pavements constructed over a sandy fill and/or engineered fill subgrade. For the design of rigid pavements an equivalent modulus of subgrade reaction of 35kPa/mm (750mm diameter plate) or a Youngs Modulus (E) value of 25MPa may be adopted.

- An estimated CBR value of 2% may be adopted for pavements constructed over a clayey fill or natural clay soil (including extremely weathered bedrock) subgrade. For the design of rigid pavements an equivalent modulus of subgrade reaction of 20kPa/mm (750mm diameter plate) or a Youngs Modulus (E) value of 15MPa may be adopted.

- Where pavements are placed over sandstone bedrock we recommend that an estimated CBR value of 10% be adopted. For design of rigid pavements an equivalent modulus of subgrade reaction of 50kPa/mm (for a 750mm diameter plate) or a Youngs Modulus (E) value of 30MPa may be adopted.

Subsurface drains will be required along the high side of external pavements and on both sides of external pavements located in low spots.

We recommend that proposed basement floor slabs be provided with under-floor drainage. The under-floor drainage should comprise a high strength, durable, single sized washed aggregate, such as 'blue metal' gravel. The under-floor drainage should connect with the wall drainage and lead to a sump for disposal to the stormwater system.

On-grade concrete floor slabs should be separated from all walls, footings etc (i.e. designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys. In addition, close to the interface between soil and bedrock subgrade conditions, additional joints and dowels will be required.

We expect that groundwater may flow along the top of the sandstone bedrock surface or along bedding partings or joint planes within the sandstone bedrock, particularly following periods of heavy or prolonged rainfall. The groundwater seepage should be monitored during bulk excavation so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design. In the long term we would recommend that drains be constructed around the basement excavation to collect all groundwater inflow and discharge this at the lower end of the site.

Consideration may be given to waterproofing subsurface walls and the underside of the basement floor slab to reduce the chance of possible dampness.

7.7 Further Geotechnical and Environmental Input

We recommend that the following further geotechnical investigations/inspections and environmental input be completed:

- A cored borehole investigation over the footprint of the proposed new building. The results of the investigation may allow optimisation of allowable bearing pressures for footing design, provide additional information of sandstone bedrock quality and strength to assist the earthworks contractor and provide information on groundwater levels.
- Geotechnical inspection of the batter slope over the south-western corner of the site following clearing of vegetation.
- Periodic quantitative vibration monitoring during rock excavation.
- Monitoring of groundwater levels during excavation.
- Inspection of each 1.5m of cut faces to identify any unfavourable joints and/or defects, along with recommending appropriate stabilisation methods.
- Geotechnical inspection and/or spoon testing of footing bases to confirm sandstone bedrock quality.
- Inspection of proof rolling of soil subgrade below all pavements or fill placed.
- Density testing of engineered fill in accordance with AS3798.
- Completion of a contamination assessment report together with a waste classification of soils that are likely to be excavated from the site prior to offsite disposal. Further advice should be sought from our specialist Environmental Investigation Services (EIS) division.

8 GENERAL COMMENTS

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

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

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

A waste classification 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.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of 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.

Paul Roberts

Paul Roberts
Associate

Agi Zenon
Senior Associate

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

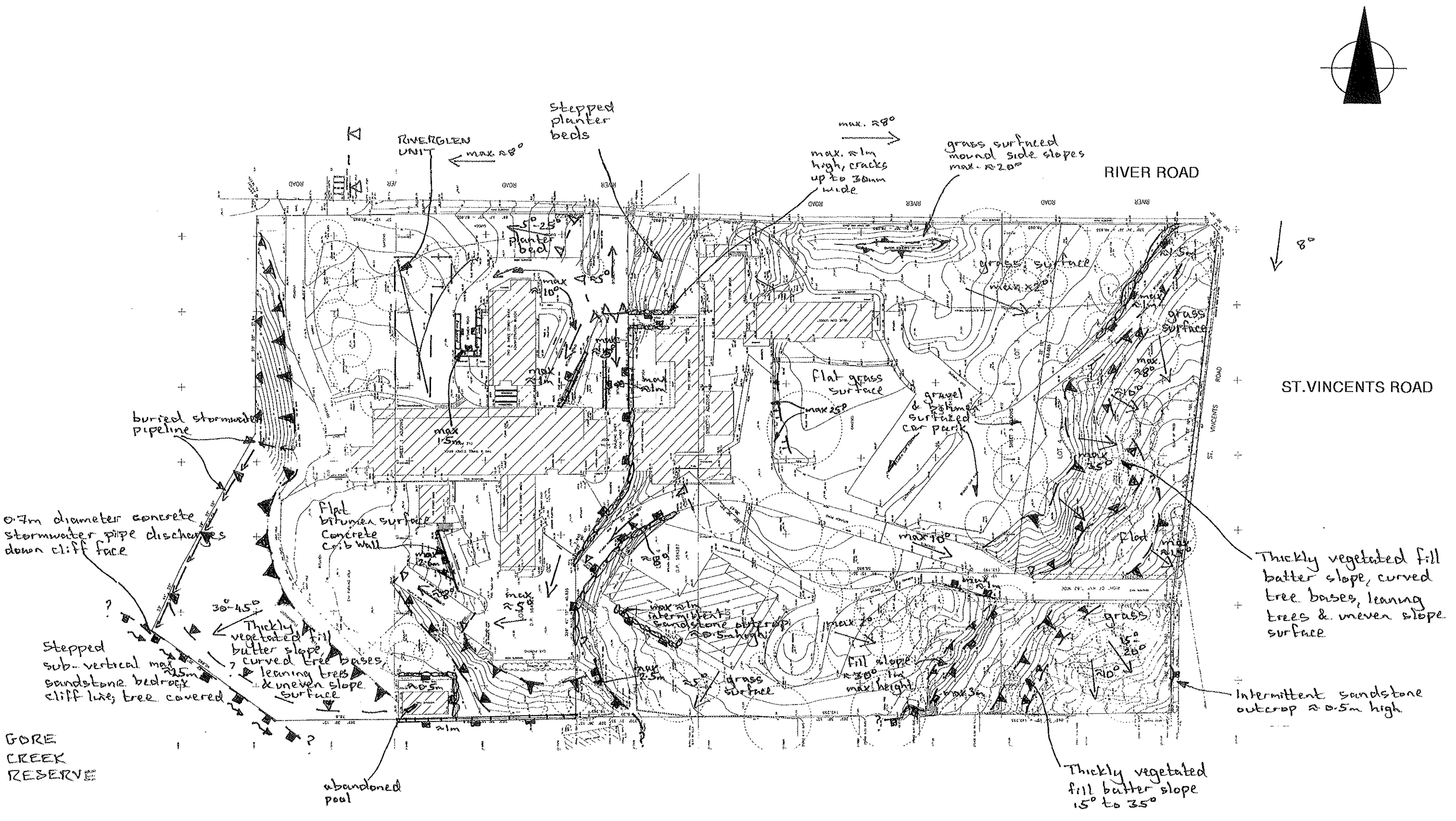
Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

TABLE A
RISK TO PROPERTY UNDER EXISTING CONDITIONS

Potential Geotechnical Hazard	i Instability of cliff line/outcrop overhangs.	ii Instability of wedges and blocks of sandstone.	iii Instability of existing retaining structures.	iv Instability of existing fill batters. On-going creep	iv Instability of existing fill batters. Rotational failure
Assessed Likelihood	**Possible**	**Possible**	**Unlikely** – walls generally in good condition **Possible** – walls in poor condition	**Almost Certain**	**Possible**
Assessed Consequences	**Insignificant** (vehicles, utilities, buildings & structures)		**Insignificant** (moving Vehicles) **Minor** (buildings) **Medium** (utilities, parked vehicle, & structures)	**Insignificant** (vehicles, utilities, buildings & structures)	**Insignificant** (moving vehicle & buildings) **Minor** (utilities & structures) **Medium** (parked vehicle & stormwater pipe western side of site)
Risk	**Very Low** (vehicles, utilities, buildings & structures)		Walls generally in good condition **Very Low** (moving Vehicles) **Low** (buildings. utilities, parked vehicles, & structures) Walls in poor condition **Very Low** (moving Vehicles) **Moderate** (buildings. utilities, parked vehicles, & structures)	**Low** (vehicles, utilities, buildings & structures)	**Very Low** (moving vehicle & buildings) **Moderate** (utilities & structures, parked vehicles & stormwater pipe western side of site)
Comments	Instability of length of overhang assumed to be a maximum of about 2m. Cliff lines and outcrops some distance from elements at risk.	Block or wedge assumed to be a maximum of about 1m length.	All retaining walls assumed to be engineer designed. Instability of localised section of wall, i.e. <5m length.		Rotational failure assumed to extend over short length, i.e. about 5m long and less than 3m thick. Parked vehicle adjacent to area of instability.

TABLE B
RISK TO LIFE UNDER EXISTING CONDITIONS

Potential Geotechnical Hazard	**i** Instability of cliff line/outcrop overhangs.	**ii** Instability of wedges and blocks of sandstone.	**iii** Instability of existing retaining structures.	**iv** Instability of existing fill batters.	
Assessed Likelihood	**Possible**	**Possible**	**Unlikely** – walls generally in good condition **Possible** – walls in poor condition	**Almost Certain** On-going creep	**Possible** Rotational failure Possible
Indicative Annual Probability	10^{-3}	10^{-3}	10^{-4} 10^{-3}	10^{-1}	10^{-3}
Persons at Risk	Patients, visitors, maintenance personnel and staff.				
Number of Persons Considered	2				
Duration of Use of Area Affected (Temporal Probability)	1. Patients: 24hrs/day i.e. 1 2. People in car parks & on walkways: Maximum 0.25hours/day i.e. 0.01 3. Visitors: Maximum say 4 hours/day/week: 0.024 4. Maintenance Personnel: Maximum 2 hours/day/month: 2.7×10^{-3} 5. Staff: say 8hrs/day shift: 0.33				
Probability of Not Evacuating Area Affected	All affected persons: 0.01		1. Patients, staff, visitors & maintenance personnel in buildings: 0.4. 2. People in car parks & on walkways: 0.2.	All affected persons: 0.001	1. Patients, staff, visitors & maintenance personnel in buildings: 0.01. 2. People in car parks: 0.2.
Spatial Probability	Assume maximum 2m over a 10m length of cliff line, slope or retaining structure (**i, iii, iv & v**) i.e. 0.2 Assume 1m failure over 10m length of rock face (**ii**) i.e. 0.1				
Vulnerability to Life if Failure Occurs Whilst Person Present	0.8 (immediately above or below overhang)	0.3	0.3	0.2	
Approximate Risk for Person Most at Risk	1. Patients: 2×10^{-6} 2. People in car parks etc: 2×10^{-8} 3. Visitors: 4×10^{-8} 4. Maint. Personnel: 4×10^{-9} 5. Staff: 5×10^{-7}	1. Patients: 3×10^{-7} 2. People in car parks etc: 3×10^{-9} 3. Visitors: 7×10^{-9} 4. Maint. Personnel: 8×10^{-10} 5. Staff: 1×10^{-7}	Good Condition 1. Patients: 2×10^{-6} 2. People in car parks etc: 1×10^{-8} 3. Visitors: 6×10^{-8} 4. Maint. Personnel: 7×10^{-9} 5. Staff: 8×10^{-7} Poor Condition 1. Patients: 2×10^{-5} 2. People in car parks etc: 1×10^{-7} 3. Visitors: 6×10^{-7} 4. Maint. Personnel: 7×10^{-8} 5. Staff: 8×10^{-6}	1. Patients: 4×10^{-6} 2. People in car parks etc: 4×10^{-8} 3. Visitors: 5×10^{-8} 4. Maint. Personnel: 1×10^{-7} 5. Staff: 1×10^{-6}	1. Patients: 4×10^{-8} 2. People in car parks etc: 4×10^{-10} 3. Visitors: 5×10^{-10} 4. Maint. Personnel: 1×10^{-9} 5. Staff: 1×10^{-8}



NOTES

TO BE READ IN CONJUNCTION WITH TEXT OF REPORT

FOR EXPLANATION OF MAPPNG SYMBOLS SEE FIGURE 2

SCALE (m)

0 50

GEOTECHNICAL SKETCH PLAN

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 23789ZR Figure No. 1

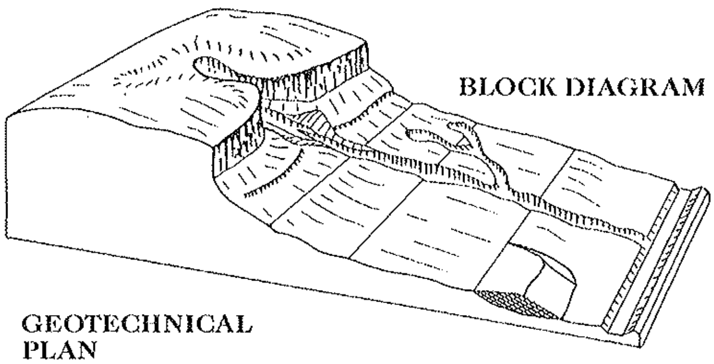
TOPOGRAPHY

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

OTHER FEATURES

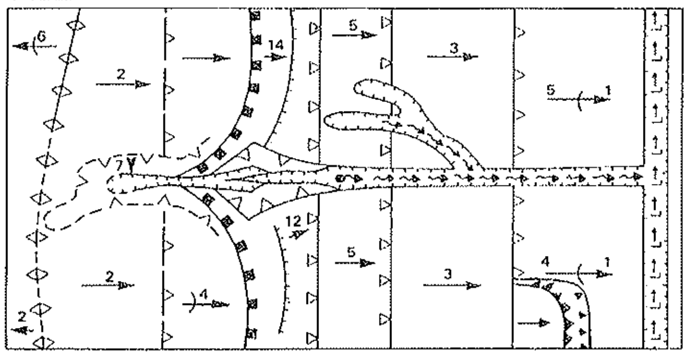
- Boulder
- Seepage/spring
- Swallow hole for runoff
- Natural water course
- Open drain, unlined
- Open drain, lined
- Fenceline
- Property boundary
- Dry Stone Wall
- J —— J 200 — Major joint in rock face (opening in millimetres)
- - T - T - 10 — Tension crack (opening in millimetres)
- Masonry or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

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



BLOCK DIAGRAM

GEOTECHNICAL PLAN

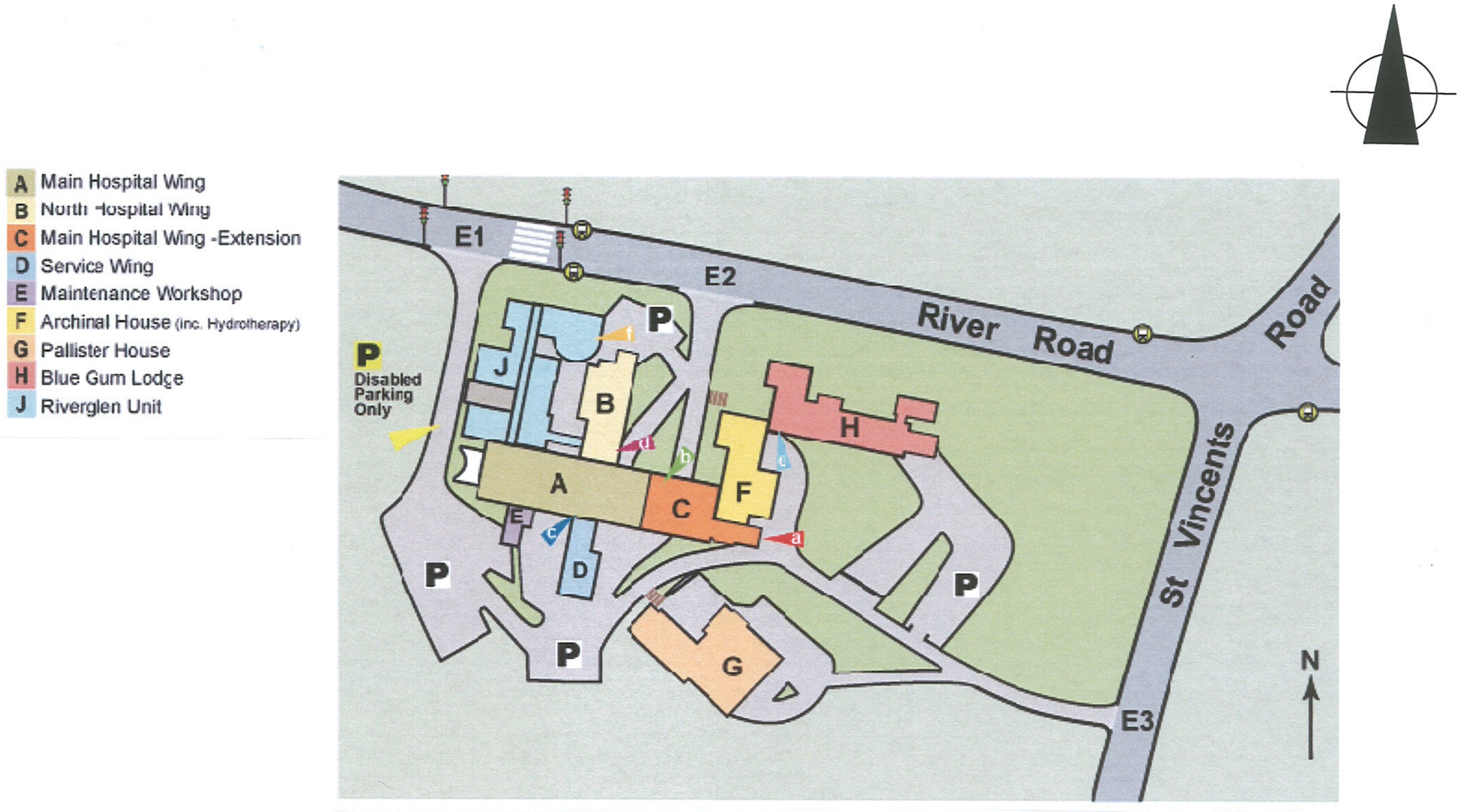


(After Gardiner, V & Dackombe, R.V. (1983), Geomorphological Field Manual; George Allen & Unwin).

GEOTECHNICAL MAPPING SYMBOLS

Jeffery and Katauskas Pty Ltd

Report No. 23789ZR Figure No. 2



GENERAL SITE PLAN

NOT TO SCALE

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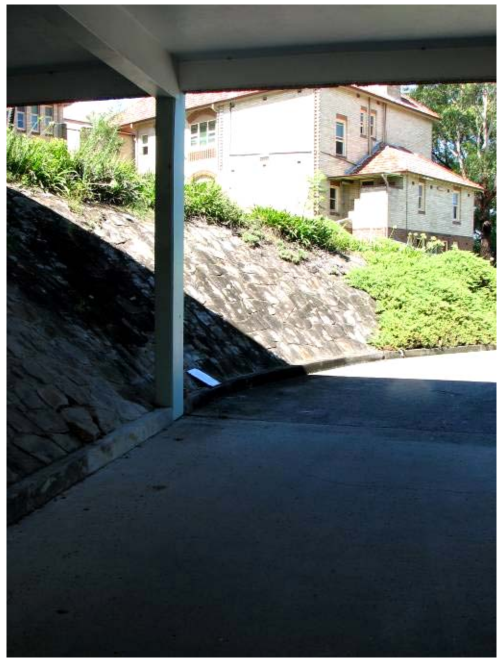
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

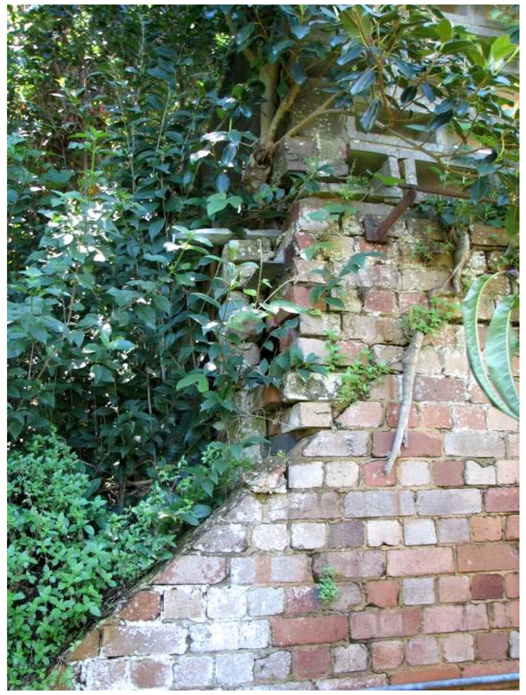
Report No. 23789ZR **Figure No. 3**



Sloping faced sandstone masonry wall below central portion of main hospital wing.

South-western corner of Pallister House

Crib wall supporting western side of car park lining western side of service wing.



Brick wall supporting south-western corner of abandoned pool.

Selected Photographs of Buildings and Structures

To be read in conjunction with text of report.

Driveway leading down to St Vincents Road.

Crest of fill slope.

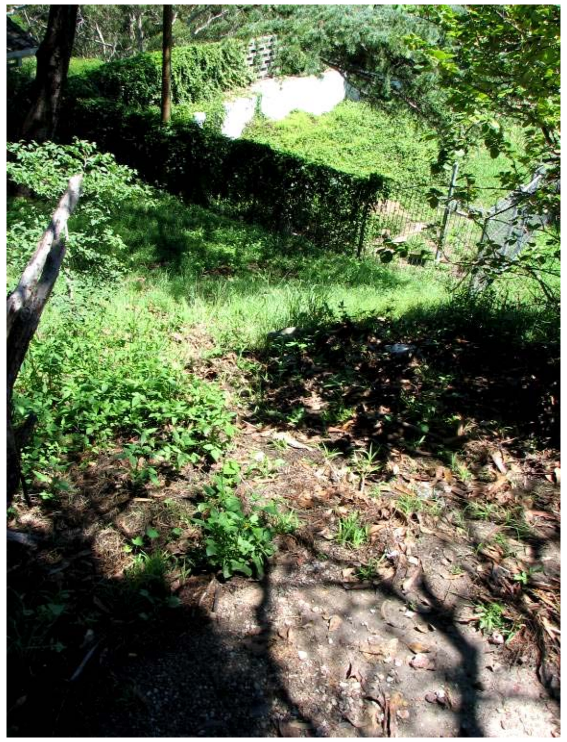
Sandstone outcrop and stacked sandstone wall lining pathway over north-eastern corner of the site.

Looking south from north-eastern corner of site boundary (St Vincents Road).

South-eastern corner of site (St Vincents Road).

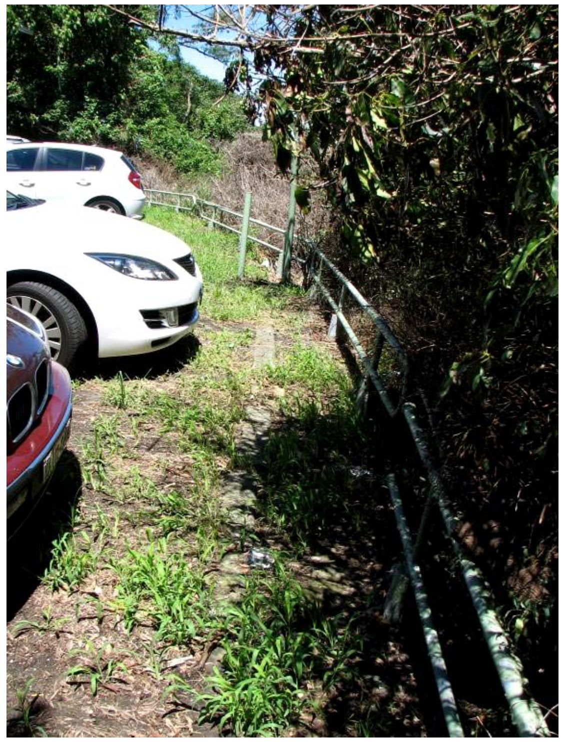
Selected Photographs of Eastern Portion of the Site

To be read in conjunction with text of report.

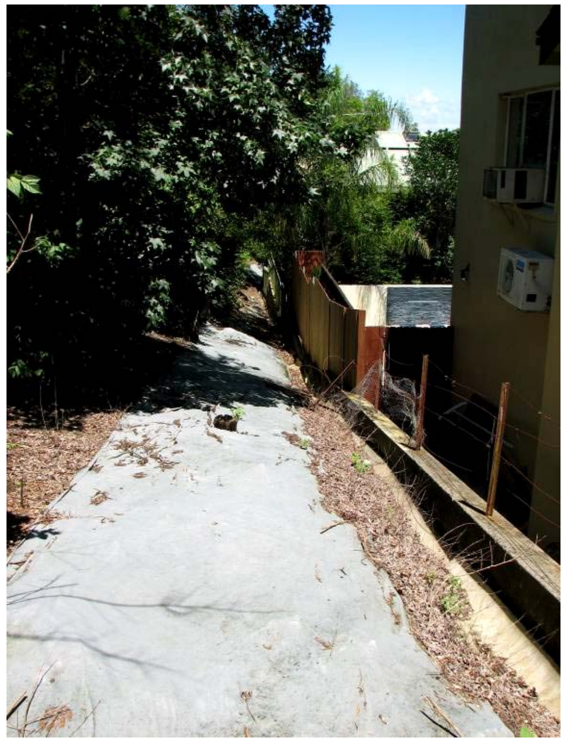


Abandoned inground pool south-western corner of the site.

Fill batter lining southern side of car park to south of service wing.



Leaning fence at crest of fill batter to west of main hospital wing.



Toe of fill batter lining northern portion of western site boundary.

Sandstone outcrop below south-western corner of Pallister House.

Vegetated cliff face below south-western portion of the site.

Selected Photographs of Western Portion of the Site

To be read in conjunction with text of report.

APPENDIX A

LANDSLIDE RISK MANAGEMENT TERMINOLOGY

APPENDIX A

LANDSLIDE RISK MANAGEMENT

DEFINITION OF TERMS

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.

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.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

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.

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.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

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 judgements 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 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 Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – 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.

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.

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.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

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, kinetic energy per unit area.

Note: Reference should also 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.

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

Qualitative Measures of Likelihood

Level	Descriptor	Description	Indicative Annual Probability
A	ALMOST CERTAIN	The event is expected to occur.	$>\approx 10^{-1}$
B	LIKELY	The event will probably occur under adverse conditions.	$\approx 10^{-2}$
C	POSSIBLE	The event could occur under adverse conditions.	$\approx 10^{-3}$
D	UNLIKELY	The event might occur under very adverse circumstances.	$\approx 10^{-4}$
E	RARE	The event is conceivable but only under exceptional circumstances.	$\approx 10^{-5}$
F	NOT CREDIBLE	The event is inconceivable or fanciful.	$< 10^{-6}$

Note: "≈" means that the indicative value may vary by say ±½ order of magnitude, or more.

Qualitative Measures of Consequences to Property

Level	Descriptor	Description
1	CATASTROPHIC	Structure completely destroyed or large scale damage requiring major engineering works for stabilisation.
2	MAJOR	Extensive damage to most of structure, or extending beyond site boundaries requiring significant stabilisation works.
3	MEDIUM	Moderate damage to some of structure, or significant part of site requiring large stabilisation works.
4	MINOR	Limited damage to part of structure, or part of site requiring some reinstatement/stabilisation works.
5	INSIGNIFICANT	Little damage.

Note: The "Description" may be edited to suit a particular case.

Qualitative Risk Analysis Matrix – Level of Risk to Property

LIKELIHOOD	CONSEQUENCES to PROPERTY				
	1: CATASTROPHIC	2: MAJOR	3: MEDIUM	4: MINOR	5: INSIGNIFICANT
A – ALMOST CERTAIN	VH	VH	H	H	M
B – LIKELY	VH	H	H	M	L-M
C – POSSIBLE	H	H	M	L-M	VL-L
D – UNLIKELY	M-H	M	L-M	VL-L	VL
E – RARE	M-L	L-M	VL-L	VL	VL
F – NOT CREDIBLE	VL	VL	VL	VL	VL

Risk Level Implications

Risk Level		Example Implications(1)
VH	VERY HIGH RISK	Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to acceptable levels; may be too expensive and not practical.
H	HIGH RISK	Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable levels.
M	MODERATE RISK	Tolerable provided treatment plan is implemented to maintain or reduce risks. May be accepted. May require investigation and planning of treatment options.
L	LOW RISK	Usually accepted. Treatment requirements and responsibility to be defined to maintain or reduce risk.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (1) The implications for a particular situation are to be determined by all parties to the risk assessment; these are only given as a general guide.

(2) Judicious use of dual descriptors for Likelihood, Consequence and Risk to reflect the uncertainty of the estimate may be appropriate in some cases.

These tables are an extract from LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES as presented in *Australian Geomechanics*, Vol. 35, No. 1, 2000 which discusses the matter more fully.

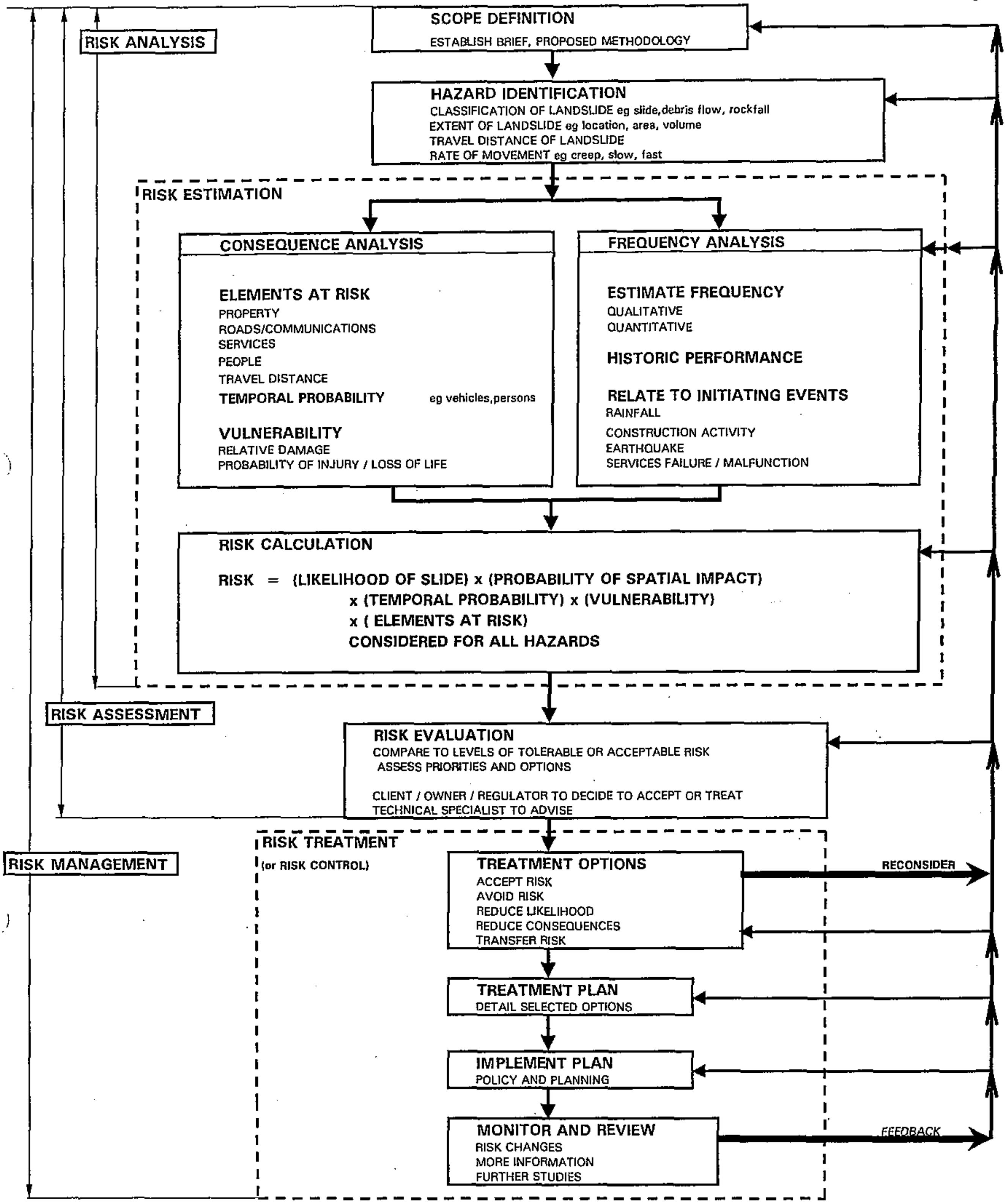


FIGURE A1: FLOWCHART FOR LANDSLIDE RISK MANAGEMENT

This figure is an extract from LANDSLIDE RISK MANAGEMENT CONCEPTS AND GUIDELINES as presented in Australian Geomechanics Vol35, No1, 2000 which discusses the matter more fully.

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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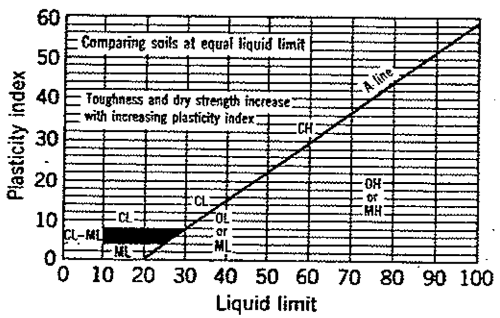
UNIFIED SOIL CLASSIFICATION TABLE

Major division	Division	Subdivision	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 division	Division	Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
		Identification Procedures on Fraction Smaller than 380 µm Sieve Size					
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	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	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	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		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.
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
August 2001

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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	