REPORT

TO
JOHN AND CHRISTINE KELLEHER

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED ALTERATIONS AND ADDITIONS

AT
12 MONTPELIER PLACE, MANLY, NSW

5 January 2018
Ref: 31123ZRrpt

dlcs ISO 9001 certified
AS/NZS 4801 certified
ISO 14001 certified

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Date: 5 January 2018
Report No: 31123ZRrpt
Revision No: 0

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1 INTRODUCTION

This report presents the results of a limited scope geotechnical investigation for the proposed alterations and additions at 12 Montpelier Place, Manly, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by John Kelleher by signed 'Acceptance of Proposal' form dated 28 November 2017. The commission was on the basis of our fee proposal (Ref. P46214ZR) dated 24 November 2017.

We have been provided with the following:

- Architectural drawings of the existing building (Drawing Numbers CC-01 to 04 Issue 1, dated 18 April 2007) prepared by Grose Bradley.
- A copy of architectural drawing CC-01 annotated by Matthew Woodward Architecture indicating the approximate area of the proposed alterations and additions over the rear yard area of the site.
- A geotechnical brief dated 23 November 2017 prepared by Matthew Woodward Architecture.
- A survey plan for 'St Patricks College Estate Precincts 1 & 13 Manly' (Ref. c163b, dated 28 June 2005) prepared by Whelans Operations Pty Ltd.

Based on the provided information and additional details provided by Matthew Woodward Architecture in an email darted 13 December 2017, we understand that the proposed alterations and additions will include:

- Landscaping of the rear yard area to include a new pool, paved areas and terraced garden beds. Excavations to a maximum depth of 1.8m are envisaged to achieve design subgrade levels. The exact location of the pool has not been determined but is expected to be situated over the upper level of the rear yard.
- Reconfiguration of the ground floor level of the existing building.

Structural loads have not been supplied, and we have assumed typical loadings for this type of development.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, retention, footings, site stability, external paved areas and the geotechnical aspects of the likely pool location.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 8 December 2017 and was limited by access constraints to the use of portable, manually operated equipment. The investigation comprised:

- Three boreholes (BH1, BH2 and BH5) hand auger drilled to respective refusal depths of 0.3m, 1.05m and 0.2m below existing surface levels.
- Four Dynamic Cone Penetration (DCP) tests (DCP1 to DCP4) completed adjacent to selected boreholes and at additional locations across the rear yard. The DCP tests were extended to refusal depths ranging between about 0.2m (DCP1) and 1m (DCP2) below existing surface levels.

The test locations, as indicated on the attached Figure 2, were set out using taped measurements from existing surface features. Architectural drawing CC-01 forms the basis of Figure 2. The surface RLs at the test locations were estimated by interpolation between spot heights shown on the supplied survey plan. The survey datum is the Australian Height Datum (AHD).

The state of compaction of the fill and the strength and density of the clayey and sandy residual soils were assessed from the DCP test results, which were supplemented by hand penetrometer readings carried out on clayey samples recovered from the hand auger. The refusal depth of DCP tests can also provide an indicative depth to rock, though we note that refusal can also occur on obstructions in fill, ‘floaters’ and other hard layers.

Groundwater observations were made in the boreholes during, and on completion of, hand auger drilling. Longer term groundwater monitoring was not carried out.

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

The fieldwork for the investigation was carried out under the direction of our geotechnical engineer (Arthur Kourtesis), who was present full-time on site and set out the test locations, logged the encountered subsurface profile, and nominated in-situ testing and sampling. In addition, our geotechnical engineer completed a visual assessment of the sandstone outcrops and cut faces within the site. Our visual assessment of the identified pertinent geotechnical features was supplemented using hand held inclinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. The borehole logs (which include field test results and groundwater observations) and the DCP test results sheet are attached, together with a glossary of logging terms and symbols used.

Geotechnical laboratory testing was not carried out as it was not deemed appropriate. A contamination screen of the site soils was not within the agreed scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located on a hillside that slopes down to the north towards the southern Cabbage Tree Bay foreshore at a maximum of approximately 5°. The site has northern and eastern frontages onto Montpelier Place.

At the time of the fieldwork, the site was occupied by a two storey rendered house, constructed over a car parking level, with stone paver and artificial grass surfaced surrounds. The flat building platform appeared to have been excavated back into the hillside; a sandstone bedrock cut face was exposed over the southern end of the car parking level. A lap pool lined the western side of the house. The site surface levels over the rear (southern) yard area stepped down to the north. The steps were formed by a timber retaining wall (about 0.2m high) situated close to the western site boundary and sandstone masonry retaining wall and sloping planter beds (about 1.7m overall height). Intermittent sandstone outcrops were noted over the upper portion of the rear yard.

The landscaped stepped front yard was supported by a sandstone masonry retaining wall which was founded on a sandstone outcrop; the overall height was about 0.8m.

A similar neighbouring three storey house was set back approximately 1.5m to 2m from the western site boundary. A rendered retaining wall (0.9m high) lined the majority of the western side boundary and supported the subject site. The southern end of the western site boundary was lined by the crest of a sandstone outcrop. The outcrop face (2m to 3m high) extended south-west to the southern site boundary then returned to the west and extended westwards along the neighbouring southern site boundary. The outcrop face (with occasional boulders over the crest area) along the neighbouring southern site boundary was about 4m to 5m high.

To the east of the site, across a continuation of Montpelier Place, was a similar house to that within the subject site, set back an estimated 6m from the eastern site boundary.

Upslope (to the south) of the site, a neighbouring bushland area, containing medium sized trees, sloped and stepped down to the north to the southern site boundary.

Based on a cursory inspection from within the subject site, the buildings and structures within and neighbouring the site appeared to be in good condition.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. Where sandstone does not outcrop, the investigation has revealed a subsurface profile comprising a limited thickness of fill and residual soil overlying sandstone bedrock at shallow depth. Groundwater seepage was intermittently encountered at shallow depth. For detailed subsurface conditions at the borehole locations, reference should be made to the attached borehole logs. A graphical borehole summary is presented as Figure 3. A summary of some of the more pertinent subsurface issues or considerations is outlined below:

Fill

Sandy fill (BH1 and BH5) and clayey fill assessed to be of medium or high plasticity (BH2) with varying gravel content were encountered from surface level. The fill extended to depths of 0.2m (BH1) and 0.55m (BH2) and the refusal depth of BH5 at 0.2m depth. Based on the results of DCP1 and DCP2, the fill was assessed to be poorly compacted. Hand auger refusal in BH5 has been interpreted to be due to the presence of sandstone bedrock, although the presence of a large inclusion within the fill cannot be discounted. Assuming a similar subsurface profile across the site, the results of DCP3 and DCP4 have been interpreted to indicate that similar poorly compacted fill extended to about 0.3m depth.

Residual Soils

Residual sand was encountered beneath the fill in BH1. Residual silty clayey sand overlying sandy clay assessed to be of medium plasticity firm to stiff strength was encountered beneath the fill in BH2. The residual soils extended to the respective refusal depths of BH1 and BH2 at 0.3m and 1.05m. Based on the DCP test results, the sands were assessed to be loose or very loose. Assuming a similar subsurface profile across the site, below 0.3m depth the results of DCP3 and DCP4 have been interpreted to indicate residual medium dense sands or stiff to very stiff sandy clay extending to the respective refusal depths of about 0.6m and 0.7m.

Weathered Sandstone Bedrock

Based on the presence of sandstone bedrock outcrops and cut faces within and neighbouring the site to the west, the refusal of the DCP tests and BH5 at depths of approximately 0.2m (DCP1), 1m, (DCP2), 0.6m (DCP3), 0.7m (DCP4) and 0.2m (BH5), have been interpreted to indicate the top surface of weathered sandstone bedrock. However, we note that refusal can occur due to the presence of large inclusions within the fill or 'floaters' within the residual soils. The bedrock surface has therefore been assumed to be stepping down to the north from the southern end of the site from approximately RL43.8m (DCP2) and RL43.4m (DCP3) to RL42m (DCP1) and RL42.1m (BH5).

We note that the outcrop and cut faces comprised sandstone bedrock assessed to be distinctly weathered and of very low and low strength.

Groundwater

Groundwater seepage was encountered in BH2 within the residual sand at 0.6m depth. However, BH2 was 'dry' for a period of 1.5 hours after completion until the borehole was backfilled. No groundwater seepage was encountered during, or on completion of, hand auger drilling BH1 or BH5. However, a standing water level was recorded at the base of BH1 (0.3m depth) 2 hours after completion. No seepage was observed over the sandstone outcrop or cut faces within the site or the neighbouring site to the west. No longer term groundwater monitoring has been carried out.

4 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The location of the pool and landscaping have not been confirmed and the comments and recommendations provided below are therefore preliminary. Once the details of the pool and landscaping have been finalised, we recommend that the advice provided below be reviewed and revised as necessary.

4.1 Site Stability

Based on Schedule 1 - Map C – Potential Geotechnical Landslip Hazard Areas presented in the Manly Development Control Plan 2013 Amendment 11 (last amended 28 August 2017), the site has been assessed to lie within a G2 Geotechnical Area.

The site has been assessed to be underlain at shallow depth by a sandstone bedrock surface that steps down to the north to the Cabbage Tree Bay foreshore.

The outcrop faces within and neighbouring the site have been controlled by sub-vertical joint planes (orientated approximately east-west and north-east to south-west). Typically, for slopes underlain by sandstone bedrock, there is the potential for 'floaters' to be present, which represent previous rock falls that have occurred over the recent geological past as erosion of the landscape occurred and formed the valleys, gullies and creek lines. Such 'floaters' were noted within the neighbouring site to the west, just beyond the southern portion of the western site boundary.

There were no obvious signs of instability across the site, such as tension cracks behind retaining wall crests, bulging/leaning retaining walls, blocks or wedges of sandstone exposed in outcrop or cut faces.

The soil profile over the footprint of the proposed pool and landscaping is relatively thin (maximum of about 1m thickness) and covers the stepped bedrock surface. The sandstone bedrock represents a suitable founding stratum for the pool, subject to geotechnical inspection. The presence of potentially unstable 'floaters' and/or blocks or wedges of sandstone within outcrop faces cannot be discounted. In order to confirm the stability of the sandstone bedrock, the following measures will be required:

1. Excavate the soil profile over the footprint of the proposed pool and over adjacent buried bedrock faces in order to expose the sandstone bedrock surface and identify any 'floaters'. The exposed sandstone bedrock surface will then need to be inspected by an experienced geotechnical engineer or engineering geologist to check for the presence of potentially unstable features and the need for stabilisation measures.
2. Once the pool excavation into the sandstone bedrock has been completed, carry out a further geotechnical inspection as described in item 1, above.

Further advice in relation to items 1 and 2 is provided in Section 4.3, below.

With the above measures implemented, and the design and construction of the proposed pool and landscaping completed in accordance with the intent of the advice provided in the following sections of this report, the stability of the site will be maintained. On this basis, the levels of risk to property and life for the proposed pool and landscaping may be regarded as 'Acceptable' in accordance with the criteria given in Reference 1.

4.2 Excavation

4.2.1 General

Excavations to a maximum depth of about 1.8m are envisaged to achieve design subgrade levels. At this stage the actual pool location with the rear yard has not been finalised but is expected to be over the upper portion of the rear yard.

Based on our observations, we consider that the existing house has been founded on bedrock. If any existing footings are required to support additional loads, then the footings should be exposed by excavating test pits to confirm this assumption. The test pits should be inspected by the geotechnical and structural engineers to assess the need for underpinning down to bedrock. Some temporary propping of the sections of the existing house that will remain may also be required and will need to be detailed by the structural engineer.

4.2.2 Excavation Methods

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

On the basis of the investigation results, the proposed excavations for the pool and landscaping will encounter the soil profile and penetrate weathered sandstone bedrock. Topsoil and/or root affected soils should be stripped and separately stockpiled for re-use in landscape areas as such soils are not suitable for re-use as engineered fill.

Such excavations we expect to be readily completed using small sized tracked excavators and/or hand held equipment, subject to access constraints. It may be that a crane is required to place a tracked excavator in the rear ayrd; further advice form contractors would need to be sought.

Excavation of low or higher strength sandstone bedrock may be achieved using rock breakers. Rock grinders and ripping attachments to the tracked excavator may be used but their effectiveness would be limited in the confined pool excavation. If access for a tracked excavator is not feasible then hand held 'kango' hammers would need to be used and this would considerably reduce the productivity of the excavation.

Care will be required to control ground vibrations associated with the use of rock breakers, such as the provision of rock saw cuts (see Section 4.2.3, below). Rock saws may also be used to create 'smooth' finishes on cut faces and aid in detailed excavation of footings, services trenches etc.

Where rock breaker, saw or grinder attachments and/or hand held ‘kango’ hammers are used, the resulting dust should be suppressed with water.

4.2.3 Potential Vibration and Ground Surface Movement Risks

Due to the presence of poorly compacted fill which we expect will extend across the site boundaries, we advise that sudden stop/start movements of tracked equipment should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures within and neighbouring the site. This is of particular concern in relation to the retaining wall lining the western site boundary. If there is any cause for concern then demolition and/or excavation should cease and further geotechnical advice sought.

Care should be taken where rock breakers are used during excavation of sandstone bedrock so that ground vibrations do not adversely affect sections of the existing house, the western site boundary retaining wall and the nearby neighbouring building to the west.

Whilst rock breakers are being used, periodic quantitative vibration monitoring of the south-eastern portion of the neighbouring building to the west will be required, to confirm that peak particle velocities (PPV) fall within acceptable limits. Consideration should also be given to the southern portion of the existing house within the site. Subject to the results of the dilapidation reports (see Section 4.2.4, below), we would recommend that the PPV along the western site boundary and on the house within the site do not exceed 5mm/sec during bedrock excavation using rock breakers. Should higher vibrations be measured they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the vibration frequency. We note that this vibration limit will reduce the risk of vibration damage to the surrounding building and structures. However, these vibrations may still result in discomfort to occupants of the surrounding buildings. If excessive vibrations are confirmed, it will be necessary to use lower energy equipment such as smaller rock breakers and/or use rock saw cuts with the base of the slot maintained below the level at which the rock breaker is being used.

Where rock breakers are used, to reduce vibrations we recommend that the rock breaker be continually orientated towards the face, and be operated one at a time and in short bursts only to reduce amplification of vibrations.

4.2.4 Dilapidation Surveys

Prior to excavation commencing, a detailed dilapidation report should be compiled on at least the southern portion of the neighbouring building to the west and the southern portion of the retaining wall lining the western site boundary. Consideration should also be given to the southern portion of the house within the site. In addition, Council may also require that a dilapidation survey report be completed on their assets lining the street frontage, i.e. the paved footpath, the roadway and kerb and gutter. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the client in defending themselves from unfair damage claims. The report may also assist the client in assessing any possible damage to the existing house caused by the contractor.

4.2.5 Groundwater Seepage

Localised groundwater seepage inflow may be expected within the excavations within the sandy soil profile close to, or at, the contact with the underlying bedrock, particularly after periods of heavy rain. In addition, concentrated flows along defects within the rock mass may also be encountered. In general, we expect the inflows to be of small volume, ephemeral and managed by conventional sump and pump techniques. Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design.

4.3 Temporary Excavation Support and Retention

4.3.1 Temporary Batters and Retention

For the generally limited depth of soil excavation, temporary batter slopes through the soil profile no steeper than 1 Vertical (V) in 1 Horizontal (H) are considered to be appropriate. We expect that the above batter slopes will generally be achievable within the site geometry, provided the pool is located at least a horizontal distance equivalent to the depth of the excavation from the site boundaries. The excavations over the western side of the pool will need to be completed with care where they are adjacent to the boundary retaining wall so as to avoid damage to the wall.

Some instability of temporary sand batters may occur at the soil-bedrock interface, especially after rain periods and sand bagging may be required to stabilise the toe of batter slopes through the soils. Conventional landscape retaining walls and pool walls may then be constructed at the base of the above temporary batters and subsequently backfilled.

4.3.2 Sandstone Cut Face and Outcrop Stability

Competent sandstone bedrock of low or higher strength may be cut vertically, subject to geotechnical inspection. As noted in Section 4.1 above, the geotechnical inspection of the outcrop and cut faces over the area of the proposed pool and landscaping should be completed as follows:

1. Once the soil profile over the footprint of the proposed pool and adjacent buried outcrop faces has been removed to expose the sandstone bedrock surface.
2. Once the pool excavation into the sandstone bedrock has been completed.

For landscape walls founded at the crest of outcrop faces, 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 face and be provided with a vertical cogged length. Where cross bedded units within the sandstone bedrock are identified during geotechnical inspections and slope down to the north, then the starter bars may have to be extended to stabilise the potentially unstable cross bedded units.

The presence of potentially unstable wedges, clay seams and extremely weathered seams within the sandstone bedrock and/or 'floaters' on the bedrock surface, may adversely affect the stability of the cut faces, the proposed pool and/or footings located close to the crests of cut faces or sandstone outcrops. Such features may require shotcreting and bolting, underpinning, removal etc. However, in some instances the prompt construction of full height landscape retaining walls may remove the need for use of shotcrete and rock bolts, although this would only be confirmed following geotechnical inspection. With regard to the pool walls, locally additional rock bolts and/or shotcrete may be required to support potentially unstable features identified by the geotechnical inspections. Provision should be made in the contract documents (budget and programme) for such inspections and stabilisation measures.

4.3.3 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the design of conventional landscape retaining walls and pool walls:

- For design of pool walls, we recommend the use of a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient (k_o) of 0.55 for the soil profile, assuming a horizontal backfill surface. However, an 'active' earth pressure coefficient, (k_a), of 0.3 may be used for the pool walls if reduced stresses are adopted for the steel reinforcing so as to control concrete cracking, in accordance with the relevant Standard.

- Where some minor movements of retaining walls may be tolerated (e.g. landscape walls), they may be designed using a triangular lateral earth pressure distribution and a coefficient of 'active' earth pressure, (k_a), of 0.3 for the soil, 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. nearby footings, compaction stresses, sloping retained surfaces, construction loads etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- Conventional retaining walls should be designed as drained and provision made for permanent and effective drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile fabric, such as Bidim A34, to act as a filter against subsoil erosion. The subsoil drains should discharge into the stormwater system.
- For conventional retaining wall footings socketed or keyed into the sandstone bedrock below design subgrade level, an allowable lateral stress of 200kPa may be adopted for sandstone of at least low strength. The socket/key depth should commence below the base of any nearby excavations such as for service trenches or footings.
- Lateral restraint of landscape walls founded in the soil profile below bulk excavation level and/or adjacent surface levels may be provided by the passive pressure of the soil below these levels. A 'passive' earth pressure coefficient, K_p, of 3 may be adopted, using a triangular lateral earth pressure distribution, provided a Factor of Safety of 2 is used in order to reduce deflections. Localised excavations in front of the walls e.g. for buried services etc should also be taken into account in the design.
- For any proposed terraced landscape retaining walls, the interaction of horizontal and vertical loads impacting the lower wall will need to be taken into account.
- Any rock bolts and/or starter bars that may be required to provide lateral restraint for retaining walls at the crests of outcrop faces, should be designed for an allowable bond strength of 200kPa assuming they are installed into sandstone bedrock of at least low strength. Rock bolts should be 'nipped' tight.
- Where rock bolts, starter bars etc extend beyond the site boundaries, permission from the neighbouring property owners will be required. Permanent rock bolts and starter bars will need to be designed with due regard for long term corrosion protection in this 'marine' environment.

4.4 Footings

Based on the shallow depth to bedrock, we recommend that the pool base and landscape retaining walls be founded in weathered sandstone bedrock.

Landscape retaining wall footings, the pool base and any new footings or underpins required in relation to reconfiguration of the existing house founded in weathered sandstone bedrock may be designed for an allowable bearing pressure of 800kPa, subject to geotechnical inspection. However, for footings founded at the crest of sandstone outcrop faces, they should be designed using a maximum allowable bearing pressure of 400kPa. Vertical sandstone faces below nearby footings 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 they should be pumped dry and then re-excavated to remove all loose and any water softened materials.

Excavations for pad or strip footings that extend through sands should be supported with formwork, as vertical cuts will be potentially unstable.

4.5 Subgrade Preparation and Engineered Fill

The following earthworks recommendations should be complemented by reference to AS3798-2007 “Guidelines on Earthworks for Commercial and Residential Developments”.

4.5.1 Subgrade Preparation

Prior to construction of external paved areas and/or placement of fill over areas of soil subgrade, preparation of the soil subgrade should consist of the following:

- Proof roll the soil subgrade with a minimum 2 tonne deadweight smooth drum vibratory roller or a hand held vibration plate compactor, where access is restricted.
- Proof rolling should be closely monitored by the site supervisor to detect soft or unstable areas which should be removed and replaced with engineered fill (as outlined below).
- Care should also be taken when using vibrating equipment not to cause damage to any adjacent structures. The vibrations should be qualitatively monitored by site personnel and if there is

any cause for concern then proof rolling should cease and further advice sought. Alternatively, where appropriate, the static (non-vibration) mode may be used.

4.5.2 Engineered Fill

Any engineered fill required to treat areas of poor subgrade and/or raise site surface levels, should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the excavated soils and weathered bedrock may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned equipment to achieve a minimum density index (I_D) of 75% for the sandy soils. However, the I_D may be reduced to 70% in landscaped areas. For clayey materials (including weathered sandstone bedrock) engineered fill should be compacted to at least 98% of Standard Maximum Dry Density (SMDD) and reduced to 95% of SMDD in landscaped areas.

Backfill to conventional landscape retaining walls should also comprise engineered fill. The excavated sands or well graded granular materials such as ripped sandstone and demolition rubble would be suitable for this purpose. This granular fill should be free of deleterious substances and should have a maximum particle size of 40mm. Such fill should be compacted in horizontal layers as above using a hand held plate compactor. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

Single sized granular material (or ‘no fines’ gravel) may be used as backfill to retaining walls and this would also act as the drainage behind the wall and would only require nominal compaction (with no compaction testing). The drainage material should be wrapped in a non woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion. Further, retaining wall backfill should be provided with a clay plug at surface level to reduce the likelihood of stormwater surcharging the retaining wall.

4.6 External Paved Areas

Slab-on-grade construction is feasible for the external paved areas provided any areas of exposed soil subgrade are prepared as outlined above. Where bedrock is at, or close to, design subgrade level then a 'bedding' layer of sand will need to be provided.

If concrete paved surfaces are proposed, then 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, we recommend that additional dowels be provided.

4.7 In-Ground Pool

The pool base should be supported on weathered sandstone bedrock in accordance with the advice provided in Section 4.4, above.

The pool should be provided with a one-way or non-return valve at its lowest point so as to prevent buoyancy in the event that external groundwater levels rise above the water level within the pool.

The pool backwash system should be piped and discharged to the main sewer system.

4.8 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Inspection of the test pits exposing existing footings, if appropriate.
- Inspection of exposed bedrock below the soil profile over the upper portion of the rear yard prior to rock excavation commencing and directing rock cut face stabilisation measures, if required.
- Dilapidation reports of adjoining buildings and structures.
- Periodic vibration monitoring during use of rock breakers for bedrock excavation.
- Geotechnical inspection of cut faces and outcrop faces and directing rock cut face stabilisation measures, if required.
- Monitoring of groundwater seepage into bulk excavations.
- Geotechnical inspection of footing bases.
- Proof rolling of exposed sub-grade.
- Density testing of engineered fill.

5 GENERAL COMMENTS

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

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

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

A waste classification 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. 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 JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/1

Client: JOHN AND CHRISTINE KELLEHER

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 12 MONTPELIER PLACE, MANLY, NSW

Job No. 31123ZR **Method:** HAND AUGER **R.L. Surface:** ≈ 42.3m

Date: 8/12/17 **Datum:** AHD

Logged/Checked by: A.C.K./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
AFTER 2 HRS ▼					REFER TO DCP TEST RESULTS	0			FILL: Silty gravelly sand, medium grained, grey brown, fine grained sandstone gravel, trace of root fibres.	M			ARTIFICIAL GRASS COVER
								SP	FILL: Sand, medium grained, brown, trace of fine to medium grained ed sandstone gravel.	M	(L)		APPEARS POORLY COMPACTED
									SAND: medium to coarse grained, grey, light grey and light brown, with medium to coarse grained sandstone gravel.				RESIDUAL ORGANIC ODOUR
									END OF BOREHOLE AT 0.3m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2

Client: JOHN AND CHRISTINE KELLEHER

Project: PROPOSED ALTERATIONS AND ADDITIONS

Location: 12 MONTPELIER PLACE, MANLY, NSW

Job No. 31123ZR **Method:** HAND AUGER **R.L. Surface:** ≈ 44.8m

Date: 8/12/17 **Datum:** AHD

Logged/Checked by: A.C.K./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION AND AFTER 1.5 HRS					REFER TO DCP TEST RESULTS	0			FILL: Silty clay, medium plasticity, dark grey brown, with fine grained sand, trace of root fibres.	MC≈PL			APPEARS POORLY COMPACTED
									FILL: Silty clay, high plasticity, light grey and light brown, trace of fine to medium grained sand.	MC>PL		280 210 215	
						0.5		CL	SILTY CLAYEY SAND: medium grained, grey.	M	(VL)		RESIDUAL
								CL	SANDY CLAY: medium plasticity, grey and light grey brown, fine to medium grained sand.	MC>PL	F-St	125 70 60	
						1			END OF BOREHOLE AT 1.05m			140 120	HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
5
1/1

Client: JOHN AND CHRISTINE KELLEHER
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 12 MONTPELIER PLACE, MANLY, NSW

Job No. 31123ZR **Method:** HAND AUGER **R.L. Surface:** ≈ 42.3m
Date: 8/12/17 **Datum:** AHD
Logged/Checked by: A.C.K./P.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Sand, medium grained, brown, with medium grained sandstone gravel.	M			
									END OF BOREHOLE AT 0.2m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						0.5							
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	JOHN AND CHRISTINE KELLEHER		
Project:	PROPOSED ALTERATIONS AND ADDITIONS		
Location:	12 MONTPELIER PLACE, MANLY, NSW		
Job No.	31123ZR	Hammer Weight & Drop: 9kg/510mm	
Date:	8-12-17	Rod Diameter: 16mm	
Tested By:	A.C.K.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL≈42.3m	RL≈44.8m	RL≈44.0m	RL≈43.6m			
Depth (mm)	**1**	**2**	**3**	**4**			
0 - 100	3	SUNK	3	SUNK			
100 - 200	3	↓	3	2			
200 - 300	20/25mm	↓	1	3			
300 - 400	REFUSAL	2	3	4			
400 - 500		2	6	4			
500 - 600		1	7	4			
600 - 700		1	4/20mm	10/60mm			
700 - 800		1	REFUSAL	REFUSAL			
800 - 900		2					
900 - 1000		2					
1000 - 1100		8/5mm					
1100 - 1200		REFUSAL					
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

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

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

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

Title:	SITE LOCATION PLAN
Location:	12 MONTPELIER PLACE MANLY, NSW
Report No: 31123ZR	Figure No: 1

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PLOT DATE: 4/01/2018 12:28:24 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31123ZR MANLY\CAD\31123ZR.DWG

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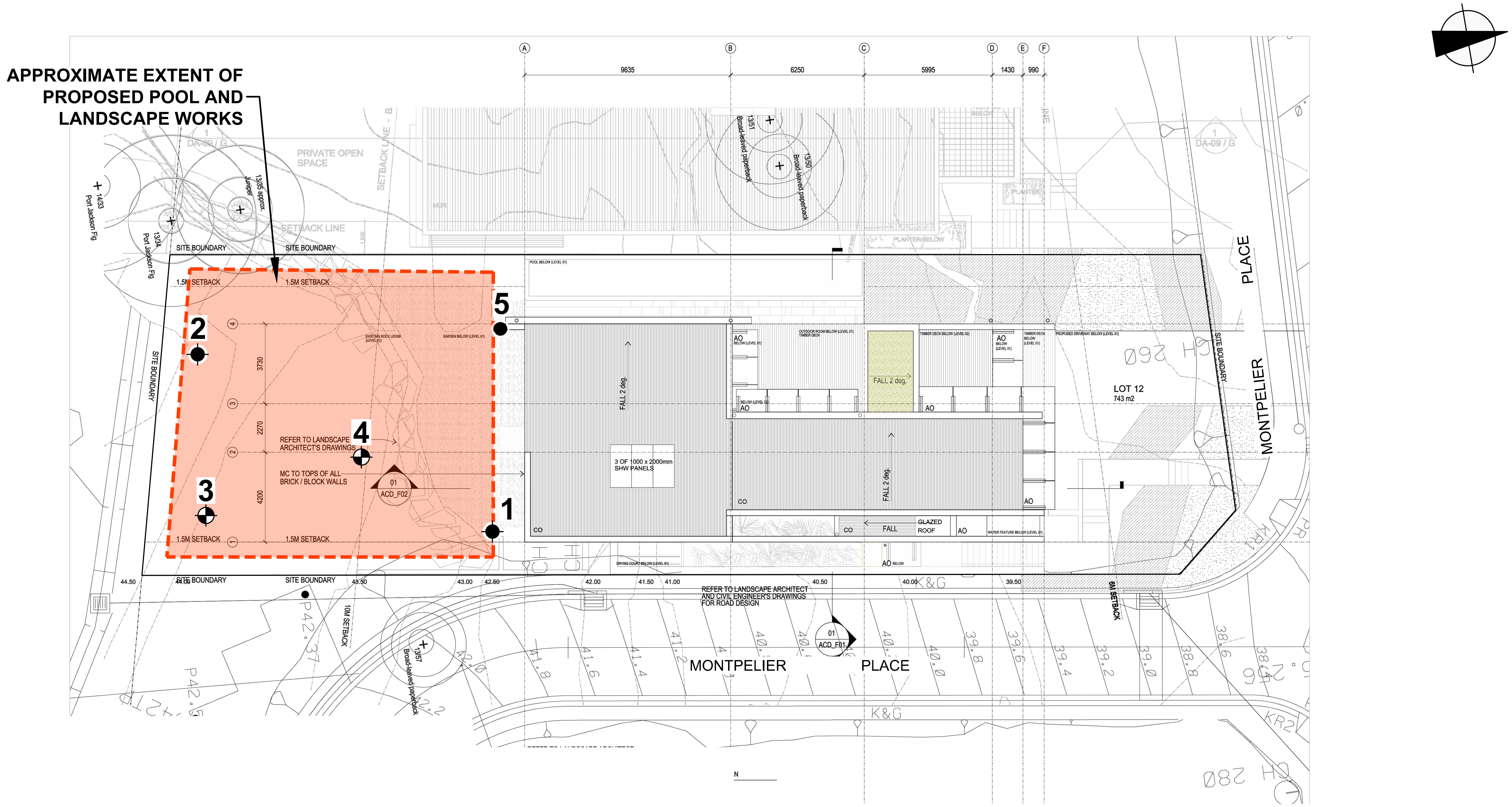

LEGEND

- BOREHOLE
- BOREHOLE AND DCP TEST
- DCP TEST

0 2 4 6 8 10

SCALE 1:200 @A3 METRES

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

Title:	TEST LOCATION PLAN
Location:	12 MONTPELIER PLACE MANLY, NSW
Report No:	31123ZR
Figure No:	2

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PLOT DATE: 5/01/2018 10:45:43 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\31000'S\31123ZR MANLY\CAD\31123ZR.DWG

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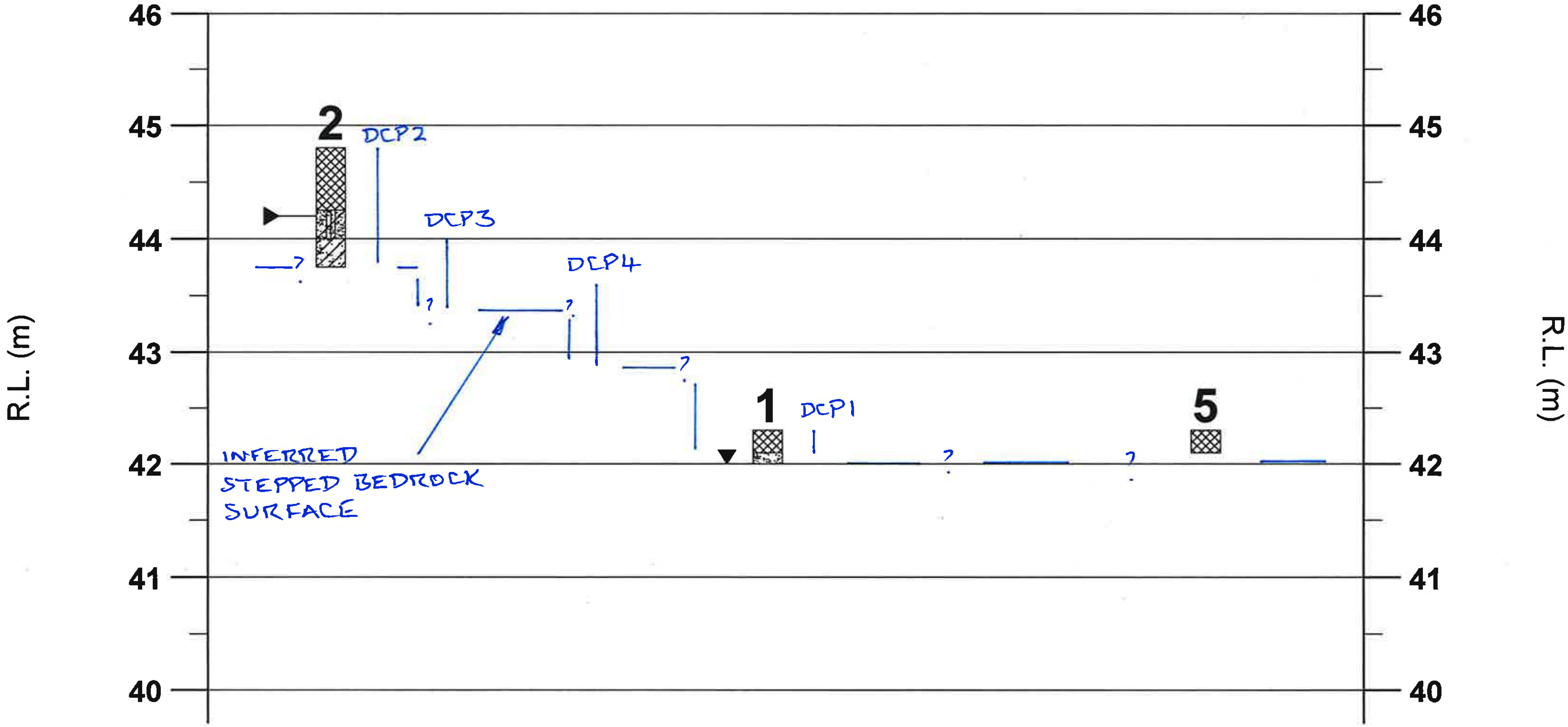
GRAPHICAL BOREHOLE SUMMARY



Fill

Description not given for: "Z#1"

Sandy Clay

Sand

Groundwater seepage level

Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale:	1 : 50 (vert) ; NTS (horiz)
JK Geotechnics	
Job No.: 31123ZR	Figure No.: 3

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, 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 a Cone Penetrometer Test (CPT). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

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

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

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

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.

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

Major divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b]	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. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. 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 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		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		$C_U = \frac{D_{60}}{D_{10}}$ 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

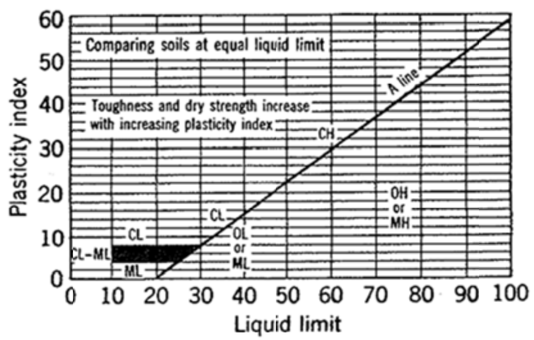
(The 75 μm sieve size is about the smallest particle visible to naked eye)

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

Identification Procedures on Fraction Smaller than 380 μm Sieve Size

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



Plasticity chart
for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. 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.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶——	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Soil sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos screening. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of Undrained Shear Strength. Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils) (Cohesionless Soils)	MC>PL MC≈PL MC<PL D M W	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. DRY – Runs freely through fingers. MOIST – Does not run freely but no free water visible on soil surface. WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt H ( )	VERY SOFT – Unconfined compressive strength less than 25kPa SOFT – Unconfined compressive strength 25-50kPa FIRM – Unconfined compressive strength 50-100kPa STIFF – Unconfined compressive strength 100-200kPa VERY STIFF – Unconfined compressive strength 200-400kPa 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)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)** Very Loose <15 0-4 Loose 15-35 4-10 Medium Dense 35-65 10-30 Dense 65-85 30-50 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 'TC' bit T_{60}	Hardened steel 'V' shaped bit. Tungsten carbide wing bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

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	