REPORT TO

GERMAN INTERNATIONAL SCHOOL SYDNEY

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED ALTERATIONS AND ADDITIONS

AT

33 MYOORA ROAD, TERREY HILLS, NSW

Date: 11 November 2021
Ref: 34428BMrpt

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
34428BMrpt	Final Report	11 November 2021

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed alterations and additions within The German International School Sydney at 33 Myoora Road, Terrey Hills, NSW. The location of the site is shown in Figure 1. The Investigation was commissioned by Mr Robertus Pratikna of TTW on behalf of the German International School Sydney, by letter dated 13 September 2021. The commission was on the basis of our fee proposal (Ref. P54890LM), dated 30 August 2021.

Based on the Geotechnical and Environmental Investigation brief prepared by TTW (Ref No. 211476, dated 24 September 2021), we understand that works are proposed within three areas of the school, comprising the construction of a two-storey science centre, three separate single storey temporary classrooms and a single storey extension to an existing building (reception). At this stage, the design is in a preliminary stage so only general details are available. The locations of the proposed works are shown on Figure 2.

The proposed Science Centre building will have a Ground Floor level at RL180.55m requiring maximum cuts of about 2.7m at the north-eastern end of the building and will be about 1.5m above existing levels at the southern end. However, due to the terraced nature of the site within the central area of the building the floor level will be similar to the existing levels. Column loads of 500kN to 600kN are anticipated.

Floor levels for the temporary classrooms have not yet been nominated but are unlikely to require significant cut or fill. Column loads in the order of 150kN and 200kN are expected.

The single storey extension to the reception building will be at the existing surface level.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on excavation, batters, retention, floor slabs, subgrade preparation, footings and earthquake subsoil classification.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E34428PDrpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on the 14 October 2021 and comprised the following scope of work:

- The auger drilling of six boreholes (BH1 to BH6) using our JK205 track mounted drilling rig to depths ranging from 0.9m (BH2) to 3.5m (BH4) below existing surface levels.
- BH5 was extended below 1.86m depth, by diamond core drilling using NMLC triple tube core barrel with water flush techniques, to a depth of 5.06m.

- Due to the limited access for the drill rig, a seventh borehole, BH7, was drilled using portable hand auger equipment to a depth of 0.75m. A Dynamic Cone Penetration (DCP) test was completed at this borehole location to refusal at a depth of 1.2m.

The investigation locations, as shown on Figure 2, were set out using a tape measure from existing surface features. The approximate surface levels shown on the attached borehole logs and DCP test results were estimated by interpolation between spot levels and surface contour lines indicated on the supplied survey (as presented on the Bettiundknut architectural 'Site Plan Existing' Drawing No. 00, Rev 03, dated 18 August 2021). The supplied survey plan forms the basis of Figure 2. The survey datum is assumed to be the Australian Height Datum (AHD).

The apparent compaction of the fill and the relative density of the natural soils were assessed from the Standard Penetration Test (SPT) and the DCP test results. The purpose of the DCP tests was to assess the compaction of the fill and to probe down to the surface of the underlying sandstone bedrock. Within the rig drilled boreholes, the strength of the upper bedrock profile was assessed by observation of the auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of the recovered rock cuttings. Rock strengths assessed in this way may vary by about one order of rock strength. The strength of the cored bedrock, in BH5 only, was assessed by examination of the recovered rock cores, together with correlation with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results.

The recovered rock core was photographed and Point Load Strength Index testing carried out in our laboratory. The rock core photograph is enclosed with the respective cored borehole log. The Point Load Strength Index test results are plotted on the cored borehole log and are summarised in Table B. The Unconfined Compressive Strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table B.

Further details of the techniques and procedures employed in the investigation are presented in the attached Report Explanation Notes, which also define the logging terms and symbols used.

Groundwater seepage observations were made in the boreholes during and on completion of drilling. No long term ground level monitoring was carried out.

Our geotechnical engineer (Ben Sheppard) was present full time during the fieldwork to set out the investigation locations, nominate the testing and sampling, and prepare the borehole logs and DCP test results sheet.

Selected soil and rock cutting samples were returned to NATA accredited laboratories (Soil Test Services Pty Ltd [STS] and Envirolab Services Pty Ltd) for moisture content, Atterberg Limits, linear shrinkage, soil pH, sulfate content, chloride content and resistivity testing. The test results are summarised in the attached STS Table A and Envirolab Services Certificate of Analysis 280556.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The following should be read with reference to Figures 1 and 2.

The school premises are situated on a broad hill top which has local undulations. The site slopes gently to the south at about 4°. The site is bound to the south-east by Myoora Road.

The school grounds contain several one and two storey structures interspersed with concrete and grassed areas, planter beds, and a covered synthetic sports field in the eastern corner. There is also a single level basement carpark under the south-western portion of the site, accessed by a ramp near the south-western side boundary. The buildings comprise concrete, concrete block and metal shipping containers, all of which are in good external condition based on a cursory inspection. An Asphaltic Concrete (AC) driveway and carpark extends along the Myoora Road boundary and was generally in good condition, with isolated minor cracking. A concrete courtyard connects the classroom structures within the south-western portion of the site, which are predominantly of concrete and concrete block construction. Grassed lawns and gravel pathways connect the north-eastern classroom structures, which predominantly comprise stacked shipping containers supported by steel columns.

The area of the proposed new science centre is terraced and steps down to the south-west. Where the building is proposed comprised two relatively level 'gardens', split by an irregular sandstone boulder retaining wall. The retaining wall is about 0.6m in height and is generally in good condition, with no signs of boulder displacement or tension cracks above. An open channel extends along the base of the retaining wall, flowing towards Myoora Road. Along the south-western edge of the lower garden area is another sandstone boulder retaining wall, 1.6m in height, with a level area on the low side to the south-west. This wall also appears to be in good condition. About 1m to the south-west of this retaining wall, are stairs leading down to the basement car park.

The area of the proposed extension to the reception building comprises a concrete surfaced courtyard with a centrally located medium sized tree. Adjacent to the existing concrete building is a separate narrow garden bed. From inspection inside the existing basement, it appears that the proposed extension is located just outside the footprint of the existing basement, but the proposed south-eastern and south-western walls of the extension may be over or very close to the existing basement walls.

The area of the proposed temporary classrooms contains gravel pathways and unmaintained sloping gardens, with some medium to large sized trees. This area slopes down to the south at about 3° to 6°.

The neighbouring properties to the north-west and north-east of the school comprise undeveloped bushland. The neighbouring property to the south-west contains a single storey residential dwelling close to the Myoora Road frontage, with gardens to the rear of the house. The house is located within about 1m of the common boundary. Ground surface levels across the common boundaries were similar to those within the school site.

3.2 Subsurface Conditions

Reference to the Sydney 1:100,000 geological map, Geological Series Sheet 9130$^{\text{Edition 1}}$ 1983, indicates that the site is underlain by Hawkesbury Sandstone, which comprises medium to coarse grained quartz sandstone with minor shale and laminite lenses.

In summary, the boreholes encountered fill over residual sandy soils, at some locations, and sandstone bedrock at relatively shallow depths. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered at each location.

Fill

Fill was encountered in all boreholes, to depths ranging from 0.2m (BH2) to 1.3m (BH4). The fill material comprised silty sand, clayey sand, sandy gravel and gravelly silty sand. Inclusions within the fill comprised sandstone and igneous gravel, timber, tile and terracotta fragments. Based on the SPT 'N' vales, the fill was generally assessed to be of poorly to well compacted.

In BH7, the hand auger refused at a depth of 0.75m within fill. It is possible the fill extends deeper, perhaps even beyond the refusal depth of the DCP test, at a depth of 1.2m, that was carried out at that location.

Residual soils

Thin layers of residual sand, silty sand and clayey sand were encountered in BH2, BH3 and BH4, respectively. Since these soil layers were assessed to be about 0.3m thick, assessment of relative density was limited but is inferred to be of loose relative density.

Weathered Sandstone

Sandstone bedrock was encountered in all of the rig drilled boreholes at depths ranging from 0.4m (BH1) to 1.65 (BH4). The sandstone was generally assessed to be of very low to low strength on first contact, improving to medium to high strength relative quickly with depth. However, the sandstone in BH2 and BH3 was high strength on first contact, while BH4 had a deeper weathered profile extending to a depth of 3m, including a layer of extremely weathered siltstone bedrock. In BH4, medium to high strength sandstone was encountered below a depth of 3m. BH1 to BH3 refused within the sandstone at depths ranging from 0.9m to 2.3m.

Within the cored portion of BH5, the sandstone was assessed to be moderately weathered and of low to medium strength. The core was almost free of defects with only one extremely weathered seams of 12mm thickness and a 'No Core' zone of 60mm. The no core zone is likely to represent a weak seam washed out by the drill flush water.

In DCP7 refusal to further penetration occurred at a depth of 1.2m. What caused refusal of the test could not be confirmed as such tests do not allow for material recovery. Assuming that the basement does not extend to this location, which appears the case from our on site measurements, we infer is likely refusal may have been caused by the presence of sandstone bedrock. However, refusal could also have occurred on an obstruction in fill or a hard layer in the natural soil profile.

Groundwater Seepage

All the boreholes were dry during and upon completion of drilling except for BH3 and BH4 where seepage was encountered during drilling at depths of 1.4m and 3.0m, respectively. These depths are similar to the respective top of competent rock levels in those boreholes. In BH3 standing water was measured on completion at 1.8m depth.

3.3 Laboratory Test Results

Based on the test results for Atterberg limits (N/A and 23%) and linear shrinkage (both 3%) as shown on STS Table A, the samples of the clayey sand fill from BH4 had negligible to low plasticity and was assessed to have a slight potential for shrink/swell movements with changes in moisture content.

The results of the point load strength index tests showed reasonably good correlation with our field assessment of rock strength. The estimate unconfined compressive strength (UCS), which is based on correlation with the point load strength test (Is_{50}) results, ranged from 6MPa to 14MPa, as shown on Table B.

Aggression testing was carried out on samples of residual silty sand, silty sandy fill and sandstone bedrock. As shown the Envirolab Certificate of Analysis, the results returned pH values varying from 5.7 and 8.7, while the chloride content ranged from less than 10mg/kg to 65mg/kg, the sulphate content ranged from less than 10mg/kg to 20mg/kg and the resistivity values ranged from 10,000 ohm.cm to 74,000 ohm.cm. Based on these results, the samples tested would have an exposure classification of ‘non-aggressive’ for concrete piles and steel piles in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009 ‘Piling – Design and Installation’.

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Issues

Three different structures are proposed each with different geotechnical issues as discussed below.

The new science centre will require excavation to a maximum depth of about 2.5m at the north-eastern end. This excavation will extend close to the synthetic surfaced sports field and it is unlikely that sufficient space will be available to form temporary batter within the soils (encountered to a depth of 1.65m in BH4) and the weathered rock. Shoring will be required to support the excavations and will need to be installed prior to the start of excavation.

At the south-western end of the proposed science centre building the floor slab will be about 1.5m above existing levels. Placement of fill to build up levels is not advisable since the fill would surcharge the walls of the existing basement and stairs, which we do not expect would have been designed for the additional surcharge pressures. Therefore, we recommend that the floor slab at this end of the building be designed as a fully suspended slab supported on piers, with a void left below the slab. For the remainder of the floor slab we expect that it would be underlain mostly by fill with weathered rock at the very north-eastern end. Therefore, if a slab on grade construction is proposed the existing fill would need to be fully excavated and replaced and allowance made for variable movement of the slab between each end supported on the rock and the central portion supported on soils. We consider that the simplest design would be to design the entire floor slab as a fully suspended slab supported on footings founded within the sandstone to reduce the risk of differential movements. The footings for the building must be founded on rock since rock will be encountered at the north-eastern end. At the south-western end of the building these footings must also be below the zone of influence of the basement walls, with the zone of influence taken as a line of 1V (Vertical) : 1H (Horizontal) drawn up from the toe of the basement.

Given sandstone bedrock is the only suitable bearing strata present across the footprint of all the buildings, all footings should be designed to be uniformly founded on sandstone bedrock.

The proposed reception extension appears to be located outside of the existing basement footprint based on our measurements, but this should be confirmed by accurate survey measurements. Along the south-eastern and south-western sides of the extension it could be supported directly on the basement walls if they align with the extension and the walls are structurally adequate to support the loads. Where new footings are proposed these should also be founded within rock below a line of 1V:1H drawn up from the toe of the basement.

We expect the temporary classrooms will be lightly loaded and are expected to only require minor cut and/or fill given the gentle to moderate hill slope where these building are proposed. Rock was encountered at shallow depths in BH1 and BH2 and slightly deeper in BH3, which is towards the toe of the slope, at a depth of 1.1m. The rock in BH2 was high strength so if cuts are required, hard rock excavation techniques will be required.

4.2 Excavation

Excavation will be mostly limited to the north-eastern end of the proposed science centre, where excavation will be required to a depth of about 2.7m, and will encounter fill, soils and weathered sandstone bedrock. The strength of the upper sandstone bedrock in BH4 ranged from residual soil strength to low to medium strength, although the higher strength material was of limited thickness of 0.4m. However, sandstone of medium to high strength may be encountered within the base of the excavation and away from BH4.

Excavation of the fill, residual soils, and the extremely weathered bedrock is expected to be readily achievable using conventional techniques, such as the buckets of medium to large sized hydraulic excavators. We also expect that excavation of the remaining very low to low strength sandstone will be achievable using large excavators with a combination of ripping tynes and toothed buckets where higher strength bands are encountered.

Some assistance with rock hammers may be required for excavation of bands of higher strength material. Care must be taken when using rock hammers due to the risk of unacceptable vibrations being transmitted to the existing structures. If hydraulic rock hammers are used the vibrations transmitted to existing structures may need to be monitored to confirm that they are less than the acceptable limits given in the attached Vibration Emission Design Goals sheet. The extent of the monitoring required will depend on the size of the hammer used and the proximity to existing structures and should be specifically assessed at the time of excavation by a geotechnical engineer.

Some seepage may occur into the excavation, which would tend to occur along the soil/rock interface or through joints and bedding partings within the rock. Any such seepage is expected to be minor and should be able to be controlled using gravity drainage and conventional sump and pump techniques. In the long term, drainage should be provided behind all retaining walls to collect and divert any seepage into the stormwater system.

4.3 Batters, Retaining Walls and Shoring

4.3.1 Batters and Landscaped Retaining Walls

For limited excavations, of no more than about 3m depth, where space permits, temporary batters within the soils and extremely weathered sandstone of 1 Vertical (V) to 1 Horizontal (H) are recommended in the short term, provided that no surcharge loads, including construction loads and existing footing loads, are placed at the top of the batters.

Permanent batters, if required, of no more than about 3m in height should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation. Permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, to reduce erosion. All stormwater runoff should also be directed away from all temporary and permanent batters to also reduce erosion.

Long term landscaping retaining walls constructed in front of temporary batters may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.3 and a bulk unit weight of $20kN/m^3$. This assumes that some resulting ground movements are tolerable. This coefficient assumes horizontal backfill behind the wall and if inclined backfill is proposed the coefficient would need to be increased or the inclined backfill taken as a surcharge load.

All surcharges must be allowed for in design, including hydrostatic pressures unless full and effective drainage is provided for the design life of the structure.

Backfilling between temporary batters and permanent walls will need to be carried out with care to reduce the future settlement of the backfill. We recommend the use of hard and durable gravel as this is readily compactable. Only light compaction equipment should be used so that excessive lateral pressures are not placed on the walls, and therefore, the backfill will need to be placed in thin layers, say 100mm loose thickness.

The compaction specification for backfill will depend if paving will be supported on the backfill. If the fill is used to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular materials. For landscaped areas, a lower compaction specification of at least 95% of SMDD may be appropriate, provided the risk of future settlement and maintenance can be accepted.

4.3.2 Shoring at North-Eastern End of Science Centre

Shoring is likely to be required at the north-eastern end of the proposed science centre building as insufficient space will be available for temporary batters, unless the batters can extend into the synthetic sports field and then repairs to the field carried out once the permanent retaining walls are backfilled.

To support the soils and poor quality rock beneath the sports field, a shoring system should be installed prior to the start of bulk excavation. A cantilevered soldier pile wall with infill panels would be suitable, given that the surcharge load from the sport court are low. The suitability of a soldier pile wall will depend on the ability of the sandy fill and residual sandy soils to stand unsupported to allow placement of the infill panels and the risk to the sports field that is acceptable to the site owner. This wall could likely comprise steel I-beams embedded in concrete piers with precast concrete planks used as infill panels progressively slotted between the I beams. However, difficulties could be experienced during drilling of the piers due to the sandy soils and we would expect that temporary liners may be required. Similarly, collapse of the sandy soils may occur between the piles and to reduce such risks a close spacing of the piles may need to be adopted.

If the sandy soils will not stand unsupported a contiguous pile wall may need to be adopted, potentially using CFA piles, but mobilisation of such equipment for a small wall is unlikely to be economical. Such a contiguous pile wall should be adopted if the risk of movement of the adjacent synthetic sports field is unacceptable to the site owner. It would be advisable to excavate trial pits were the wall is proposed to assess how well the soils stand unsupported and assess the feasibility of a soldier pile wall.

The retaining wall may be designed as a cantilevered wall using a triangular earth pressure distribution and a coefficient of active earth pressure, k_a, of 0.3 and a bulk unit weight of $20kN/m^3$, provided the risk of movement behind the wall is acceptable. If movement are to be reduced, then an ‘at rest’ earth pressure coefficient, K_0, of 0.55 should be used.

The above coefficients assume horizontal backfill behind the wall and if inclined backfill is proposed the coefficients would need to be increased or the inclined backfill taken as a surcharge load. All surcharge loads and appropriate hydrostatic pressures should be added to the above pressures unless complete and permanent behind wall drainage is adopted in the design.

Where the piles extend below the base of the proposed excavation, including local excavations for footings and services, a lateral passive resistance of 200kPa may be adopted.

4.4 Subgrade Preparation for On Grade Floor Slab(s)

As discussed in Section 4.1 above, the south-western end of the proposed science centre building should be fully suspended so that additional surcharge loads are not placed on the existing basement walls. For the remainder of this building slab on grade construction could be adopted, but where existing fill is present below the slab it would need to be removed and replaced with controlled, engineered fill. In addition, movement joints would need to be provided within the slab to allow for differential movement. Due to this it may be more practical to adopt a fully suspended slab for the entire building.

For the proposed reception building extension, again a fully suspended floor slab should be adopted so that surcharge loads are not placed on the existing basement walls.

For the proposed temporary classrooms, to allow slab on ground construction all existing fill would need to be fully excavated and replaced by controlled, engineered fill. We are unaware of any records of placement or compaction control of the existing fill so it must be considered 'uncontrolled' and is not suitable to support floor slabs.

The following recommendations should be followed for placement of fill and where slabs are to be cast on the existing subgrade. However, where fully suspended floor slabs are adopted no particular subgrade preparation would be required other than stripping of root affected soils.

All vegetation and any root affected soils should be fully stripped prior to proof rolling. If excavation and replacement of the uncontrolled fill is to be carried out then all existing fill should be fully stripped and the sides of the excavation battered.

The exposed soil subgrade should be proof rolled with at least 7 passes of a minimum 8 tonne deadweight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and in the presence of a geotechnical engineer to detect any weak subgrade areas. Vibration should only be used in conjunction with vibration monitoring given the close proximity to existing structures. We recommend vibrations are limited to a maximum 5mm/s peak particle velocity. Reference should also be given to the attached Vibration Emission Design Goals. If vibration becomes untenable, rolling should be carried out without vibration and the number of static roller passes increased. Any weak areas detected should be locally excavated to a sound base and the excavated material replaced with engineered fill, or as directed by the geotechnical engineer during the proof rolling inspection.

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations, such as service trenches, or where small rollers need to be used, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated fill and residual soils and weathered rock my be reused as engineered fill, provide they are free of deleterious materials and particles of greater than 75mm in size. Crushing of the excavated rock may be required to reduce the maximum particle size.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. Where the fill is to support building loads it should be placed under Level 1 control, as defined by AS3798. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.5 Site Classification and Footings

Due to the uncontrolled fill encountered within the boreholes, the site would be classed as ‘Class P’ in accordance with AS2870-2011 Residential Slabs and Footings.

The only suitable strata for footings that was present at each borehole is the sandstone bedrock. All footings should be founded within sandstone bedrock of at least very low strength and may be designed based on an allowable bearing pressure of 1,000kPa.

Strip and pad footings will be suitable where rock is at shallow depth, but bored piers may be required where rock is deeper than about 1m. However, difficulties with collapse of the sandy fill and sand may be experienced and the use of sacrificial pier liners may be required. For pad por strip footings, temporary shoring may be required or the sides of the excavation temporarily battered.

Screw piles are not recommended as they are unlikely to uniformly bear upon the bedrock where the surface of the rock is of low or higher strength. The pilot of the screw piles is likely to refuse so that the load plate is not embedded into the sandstone and this may lead to excessive settlement. Screw piles may also meet premature refusal on obstructions in fill.

Seepage occurred in BH3. We expect this is due to ponding on a localised depression in the rock surface and should be easily removed. Where seepage does occur a larger footing excavation may be required with a sump and pump to remove the water. Should the water not be readily removable or should large volumes of water be encountered we should be contacted immediately for further advice.

All footings must be founded below a line of 1V: 1H drawn up from the toe of any excavations, buried service trenches or basements. Piling rigs with sufficient power to penetrate medium and high strength rock should be selected, given the strength of rock encountered within the boreholes.

All foundation material should be inspected by a geotechnical engineer.

4.6 Earthquake Subsoil Classification

The site sub-soil class is a ‘Class B_e – Rock’ in accordance with AS1170-2007 with Amdt 1 and 2.

4.7 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Excavation of a series of test pits to check if the soils will stand unsupported if a soldier pile walls is adopted at the north-eastern end of the science centre building.
- Inspection of proof rolling of subgrade prior to placement of fill.
- Density testing of any fill placed.
- Inspection of footing excavations.

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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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

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

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.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Alterations Additions
Location: 33 Myoora Road, Terrey Hills, NSW

Report No.: 34428LM - A
Report Date: 28/10/2021
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AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
4	0.50 - 0.95	19.1	N/A	N/A	NP	3.0
6	0.50 - 0.95	10.8	23	16	7	3.0

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/10/2021.
- Sampled and supplied by client. Samples tested as received.
- NP denotes Non-Plastic
- For Liquid Limit: N/A denotes Slippage in the cup
- For Plastic Limit: N/A denotes Not Obtainable

NATA Accredited Laboratory Number:1327

28/10/2021
Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	German International School Sydney C/- TTW	**Ref No:**	34428LM
Project:	Proposed Alterations and Additions	**Report:**	B
Location:	33 Myoora Road, TERREY HILLS, NSW	**Report Date:**	15/10/21

Page 1 of 1

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH5	1.90 - 1.94	0.2	4	A
	2.16 - 2.19	0.3	6	A
	2.75 - 2.78	0.6	12	A
	3.26 - 3.30	0.2	4	A
	3.85 - 3.90	0.3	6	A
	4.23 - 4.27	0.4	8	A
	4.72 - 4.76	0.7	14	A
	5.01 - 5.05	0.6	12	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the IS(50) has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 IS(50).

Envirolab Services Pty Ltd
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www.envirolab.com.au

CERTIFICATE OF ANALYSIS 280556

Client Details

Client	JK Geotechnics
Attention	Ben Sheppard
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34428LM, 33 Myoora Road, Terrey Hills, NSW**
Number of Samples	3 Soil
Date samples received	18/10/2021
Date completed instructions received	18/10/2021

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	25/10/2021
Date of Issue	21/10/2021

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		280556-1	280556-2	280556-3
Your Reference	UNITS	BH4	BH5	BH6
Depth		1.5-1.65	0.5-0.95	1.67-1.95
Type of sample		Soil	Soil	Soil
Date Sampled		14/10/2021	14/10/2021	14/10/2021
Date prepared	-	19/10/2021	19/10/2021	19/10/2021
Date analysed	-	19/10/2021	19/10/2021	19/10/2021
pH 1:5 soil:water	pH Units	6.0	8.7	5.7
Sulphate, SO4 1:5 soil:water	mg/kg	<10	65	10
Chloride, Cl 1:5 soil:water	mg/kg	<10	20	<10
Resistivity in soil*	ohm m	450	100	740

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			19/10/2021	1	19/10/2021	19/10/2021		19/10/2021	[NT]
Date analysed	-			19/10/2021	1	19/10/2021	19/10/2021		19/10/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	6.0	6.0	0	99	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	88	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	89	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	450	440	2	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

JKGeotechnics

BOREHOLE LOG

Borehole No.
1
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM
Method: SPIRAL AUGER
R.L. Surface: ~184.8 m
Date: 14/10/21
Datum: AHD
Plant Type: JK205
Logged/Checked By: B.S./M.P.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION										FILL: Silty sand, fine to medium grained, dark brown, trace of fine grained sandstone gravel, timber fragments and root fibres.	M			MULCH COVER
				■	N=SPT 7/ 50mm REFUSAL	184	1		-	SANDSTONE: medium to coarse grained, yellow brown and grey.	DW	VL - L		HAWKESBURY SANDSTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
				■		183						M - H		MODERATE TO HIGH RESISTANCE
							2			as above, but high strength iron indurated bands				
										END OF BOREHOLE AT 2.00 m				'TC' BIT REFUSAL
						182	3							
						181	4							
						180	5							
						179	6							
						178								

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BOREHOLE LOG

Borehole No.
2
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM **Method:** SPIRAL AUGER **R.L. Surface:** ~184.3 m
Date: 14/10/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./M.P.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION								▩		FILL: Silty sand, fine to medium grained, brown, trace of timber fragments and root fibres.	M			GRAVEL COVER
				■		184		⠿	SP	SAND: fine to coarse grained, grey, trace of fine grained quartz gravel and silt.	M	(L)		RESIDUAL
				■	N=SPT 4/ 0mm REFUSAL			⠶	-	SANDSTONE: medium to coarse grained, orange brown and grey.	DW	H		HAWKESBURY SANDSTONE HIGH 'TC' BIT RESISTANCE
							1			END OF BOREHOLE AT 0.90 m				'TC' BIT REFUSAL
						183								
							2							
						182								
							3							
						181								
							4							
						180								
							5							
						179								
							6							
						178								

BOREHOLE LOG

Borehole No.
3
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM **Method:** SPIRAL AUGER **R.L. Surface:** ~183.6 m
Date: 14/10/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./M.P.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
				■ ■	N = 3 1,2,1	183	1	▩		FILL: Silty sand, fine to medium grained, dark brown, trace of fine grained igneous gravel and timber fragments.	M			GRASS COVER APPEARS POORLY COMPACTED
								▨	SC	Clayey SAND: fine to medium grained, orange brown, trace of fine grained sand and clay lenses.	M	(L)		RESIDUAL
▶ ▽						182	2	░	-	SANDSTONE: medium to coarse grained, red brown.	DW	H		HAWKESBURY SANDSTONE MODERATE TO HIGH 'TC' BIT RESISTANCE
						181 180 179 178 177	3 4 5 6			END OF BOREHOLE AT 2.30 m				'TC' BIT REFUSAL

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34428LM TERREYHILLS.GPJ <<DrawingFile>> 08/11/2021 09:51 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

BOREHOLE LOG

Borehole No.
4
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM **Method:** SPIRAL AUGER **R.L. Surface:** ~182.3 m
Date: 14/10/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./M.P.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
			182				FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained igneous gravel, clay fines, terracota fragments and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
		N = 8 3,3,5		1			FILL: Clayey sand, fine to medium grained, dark brown, trace of fine to medium grained igneous and sandstone gravel, silt and organic sand lenses.				
			181			SM	Silty SAND: medium to coarse grained, grey.	M	L		RESIDUAL
		N > 12 4,12/ 60mm REFUSAL	180	2		-	SANDSTONE: fine to coarse grained, grey, orange brown and red brown.	HW	VL L - M		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE MODERATE RESISTANCE
						-	Extremely Weathered siltstone: silty CLAY, grey and red brown.	XW	(Hd)		VERY LOW RESISTANCE
▶			179	3		-	SANDSTONE: medium to coarse grained, red brown and orange brown.	MW	M - H		MODERATE TO HIGH RESISTANCE
			178 177 176	4 5 6			END OF BOREHOLE AT 3.50 m				

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BOREHOLE LOG

Borehole No.
5
1 / 2

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM
Method: SPIRAL AUGER
R.L. Surface: ~180.6 m
Date: 14/10/21
Datum: AHD
Plant Type: JK205
Logged/Checked By: B.S./M.P.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					N = 26 14,12,14	180				FILL: Silty sand, fine to medium grained, grey brown, with fine to coarse grained igneous and sandstone gravel, trace of sandy clay lenses terracota and tile fragments.	M			GRASS COVER APPEARS WELL COMPACTED
							1			FILL: Silty sand, fine to coarse grained, dark brown and brown, trace of fine to medium grained igneous and sandstone gravel.				
						179			-	SANDSTONE: fine to medium grained, red brown, orange brown and grey, with extremely weathered bands.	HW	VL		HAWKESBURY SANDSTONE VERY LOW TO LOW 'TC' BIT RESISTANCE
											MW	L		LOW TO MODERATE RESISTANCE WITH VERY LOW BANDS
							2			REFER TO CORED BOREHOLE LOG				
						178	3							
						177	4							
						176	5							
						175	6							
						174								

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CORED BOREHOLE LOG

Borehole No.
5
2 / 2

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM	**Core Size:** NMLC	**R.L. Surface:** ~180.6 m
Date: 14/10/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK205	**Bearing:** N/A	**Logged/Checked By:** B.S./M.P.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		179			START CORING AT 1.86m							
100% RETURN ON COMPLETION OF CORING		178 177	2 3 4		SANDSTONE: fine to medium grained, orange brown and grey, distinctly bedded at 0-10°, with cross bedding up to 25° and medium to coarse grained bands.	MW	L L - M	0.20 0.30 0.60 0.20 0.30		(3.00m) XWS, 0°, 12 mm.t		Hawkesbury Sandstone
					NO CORE 0.06m							
		176	5		SANDSTONE: fine to medium grained, grey and purple brown, distinctly bedded at 0-10°, with cross bedding up to 25°.	MW	M	0.40 0.70 0.60				Hawkesbury Sandstone
		175 174 173	6 7		END OF BOREHOLE AT 5.06 m							

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 34428LM TERREYHILLS.GPJ <<DrawingFile>> 08/11/2021 09:51 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Job No: 34428LM
Borehole No: BH5
Depth: 1.86m - 5.06m

BOREHOLE LOG

Borehole No.
6
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM **Method:** SPIRAL AUGER **R.L. Surface:** ~180.4 m
Date: 14/10/21 **Datum:** AHD
Plant Type: JK205 **Logged/Checked By:** B.S./M.P.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■					180				FILL: Gravelly silty sand, fine to medium grained, brown and dark brown, fine to coarse grained igneous gravel, trace of tile fragments, clay nodules and root fibres.	M			GRASS COVER APPEARS MODERATELY COMPACTED
	■			■	N = 14 5,6,8		1			FILL: Clayey sand, fine to medium grained, grey brown and red brown, trace of medium grained sandstone gravel, silt, sand lenses and roots,				
				■	N > 6 9,6/ 20mm REFUSAL	179			-	Extremely Weathered sandstone: silty SAND, fine grained, grey and orange brown, trace of clay fines and very low strength bands.	XW	D		HAWKESBURY SANDSTONE VERY LOW 'TC' BIT RESISTANCE
				■			2			SANDSTONE: fine to coarse grained, grey, red brown and orange brown.	MW	L M		LOW TO MODERATE RESISTANCE MODERATE RESISTANCE
				■		178				as above, but medium to coarse grained, brown and red brown.		M - H		MODERATE TO HIGH RESISTANCE
						177 176 175 174	3 4 5 6			END OF BOREHOLE AT 3.00 m				

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BOREHOLE LOG

Borehole No.
7
1 / 1

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No.: 34428LM
Method: HAND AUGER
R.L. Surface: ~178.9 m
Date: 14/10/21
Datum: AHD
Plant Type: -
Logged/Checked By: B.S./M.P.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION				■	REFER TO DCP TEST RESULTS			▨		FILL: Gravelly silty sand, medium grained, grey brown, fine to medium grained igneous gravel.	M			MULCH COVER APPEARS POORLY COMPACTED
				■				▨		FILL: Sandy gravel, coarse grained, red brown, brown and grey, igneous and ironstone gravel, fine to coarse grained sand, and tile fragments.	M			
						178	1			END OF BOREHOLE AT 0.75 m				NOTE: 3 ATTEMPTS WERE MADE TO ADVANCE LAYER, HOWEVER REFUSAL OCCURRED AT OR PRIOR TO 0.75m DEPTH
						177	2							
						176	3							
						175	4							
						174	5							
						173	6							
						172								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34428LM TERREYHILLS.GPJ <<DrawingFile>> 08/11/2021 09:52 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

DYNAMIC CONE PENETRATION TEST RESULTS

Client: GERMAN INTERNATIONAL SCHOOL SYDNEY C/-TTW
Project: PROPOSED ALTERATIONS AND ADDITIONS
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Job No. 34428BM
Date: 14-10-21
Tested By: B.S.

Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Test Location	7						
Surface RL	≈178.9m						
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	1/20mm						
100 - 200	5						
200 - 300	9						
300 - 400	8						
400 - 500	5						
500 - 600	7						
600 - 700	4						
700 - 800	5						
800 - 900	4						
900 - 1000	5						
1000 - 1100	7						
1100 - 1200	12						
1200 - 1300	10/0mm						
1300 - 1400	REFUSAL						
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

Ref: JK Geotechnics DCP 0-3m Rev5 Feb19

SOURCE: http://www.whereis.com/

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:
SITE LOCATION PLAN

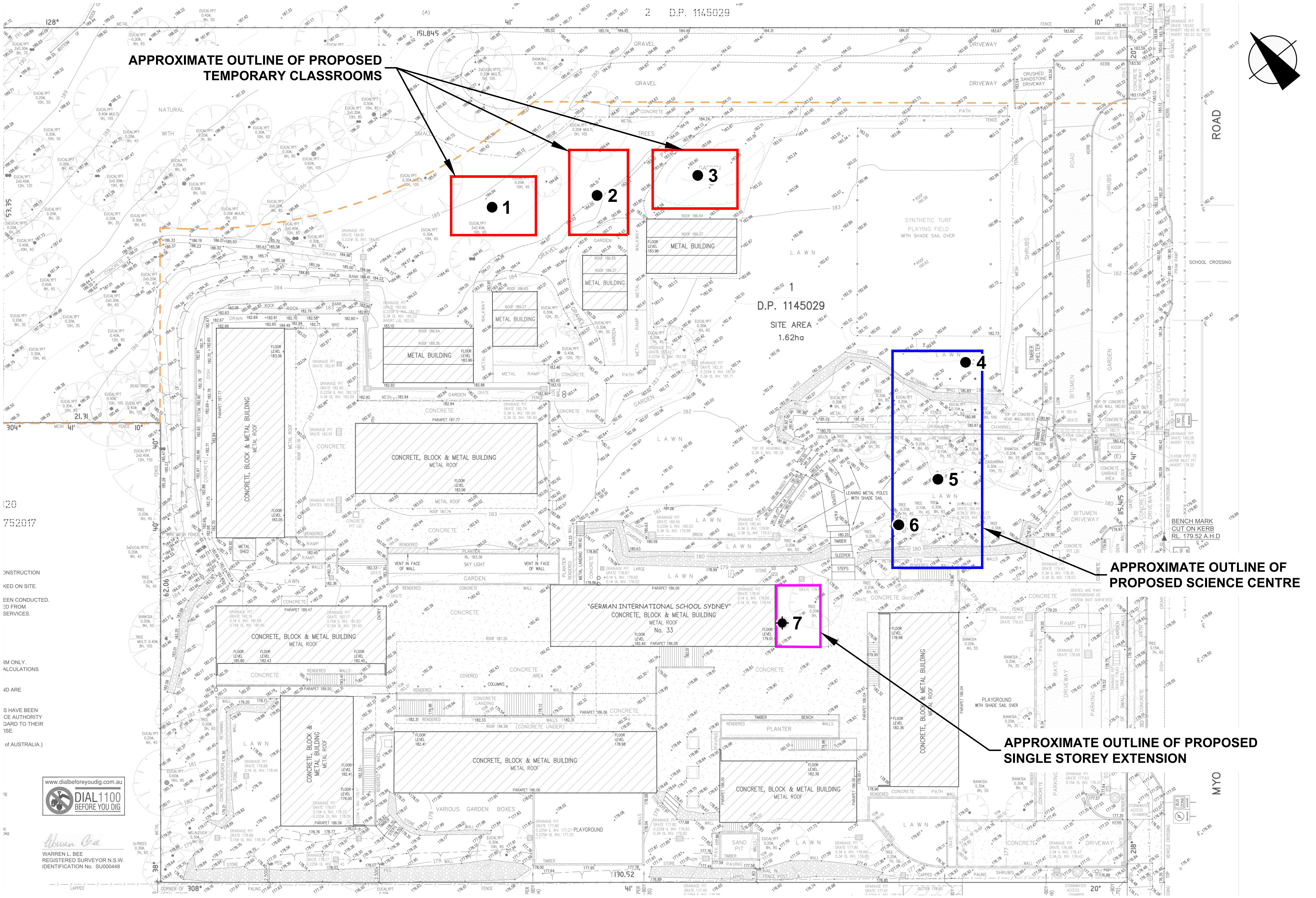
Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Report No: 34428BM

Figure No: 1

JKGeotechnics

PLOT DATE: 19/10/2021 9:52:43 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34428LM TERREY HILLS\CAD\34428LM.DWG



LEGEND

● BOREHOLE

⊕ BOREHOLE AND DCP TEST

0 5 10 15 20 25

SCALE 1:500 @A3 METRES

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

Title:

INVESTIGATION LOCATION PLAN

Location: 33 MYOORA ROAD, TERREY HILLS, NSW

Report No: 34428BM

Figure No: 2

JKGeotechnics

PLOT DATE: 19/10/2021 9:52:54 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34428BM TERREY HILLS\CAD\34428BLM.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

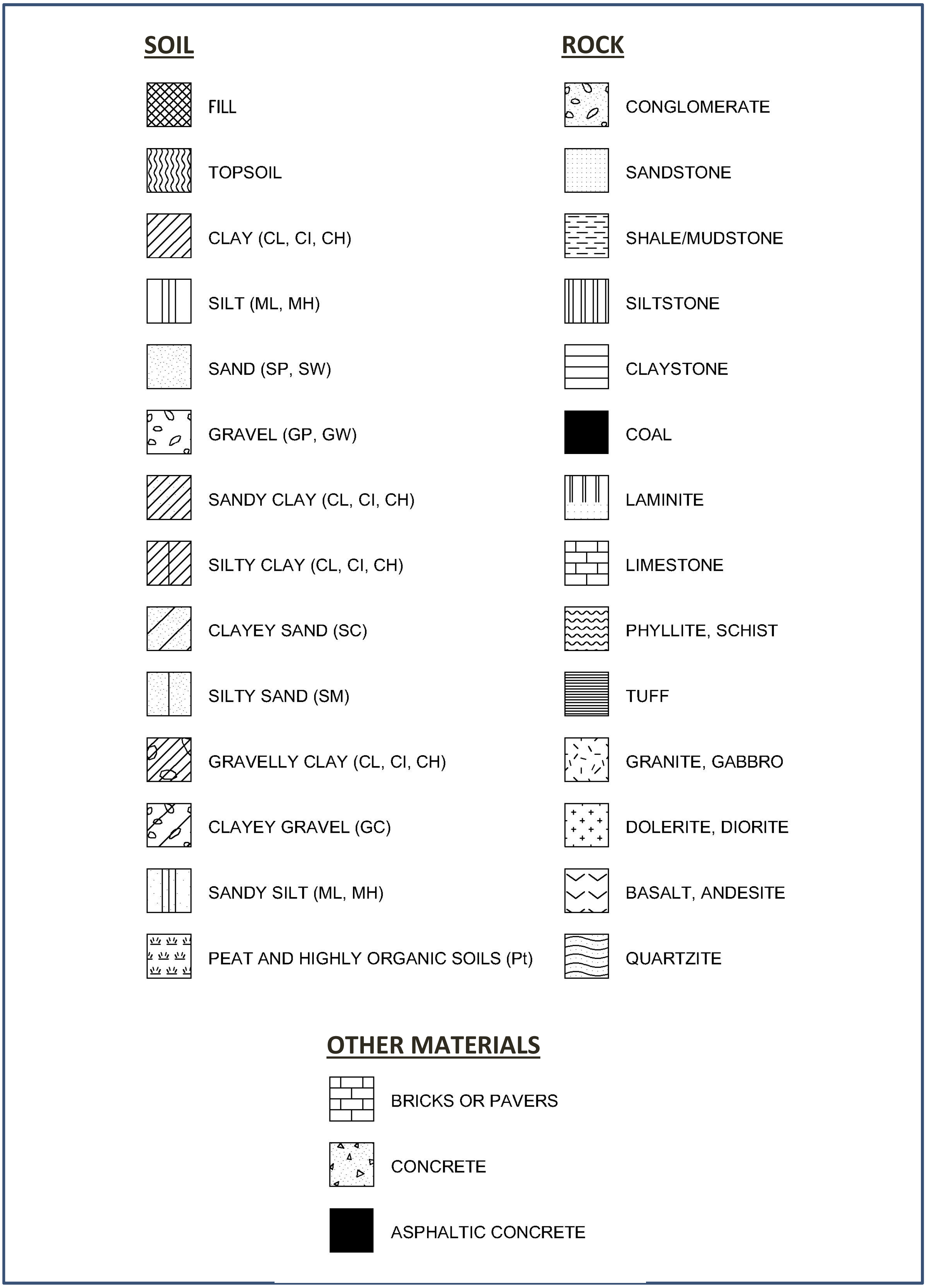
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS



SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ■ ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

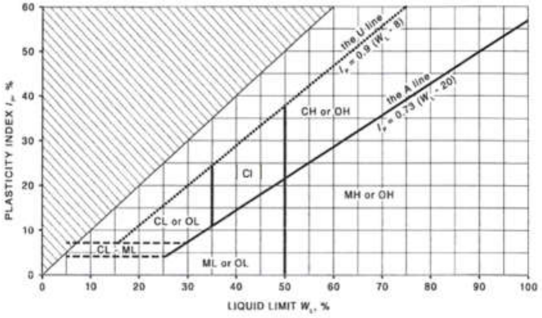
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



LOG SYMBOLS

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

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

Classification of Material Weathering

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

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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