UPDATED REPORT

TO
PAYNTER DIXON CONSTRUCTIONS PTY LTD

ON
'DUE DILIGENCE' GEOTECHNICAL INVESTIGATION

FOR
PROPOSED ILU & RACF DEVELOPMENT

AT
BLACKTOWN WORKERS SPORTS CLUB
170 RESERVOIR ROAD, ARNDELL PARK, NSW

9 March 2018
Ref: 28870A6rpt

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

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

PO Box 976, North Ryde BC NSW 1670
Tel: 02 9888 5000 Fax: 02 9888 5003
www.jkgeotechnics.com.au

Jeffery & Katauskas Pty Ltd, trading as JK Geotechnics ABN 17 003 550 801

Date: 9 March 2018
Report No: 28870A6rpt
Revision No: 0

Report prepared by:

Andrew Jackaman
Senior Associate | Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

TABLE OF CONTENTS

STS Table A: **Moisture Content, Atterberg Limits & Linear Shrinkage Test Report**

Borehole Logs 212 to 219

Test Pit Logs 220, 228 & 229

Figure 1: **Site Location Plan**

Figure 2: **Test Location Plan**

Figures 3 to 5: **Graphical Borehole Summaries**

Vibration Emission Design Goals

Report Explanation Notes

1 INTRODUCTION

This updated report presents the results of a 'due diligence' geotechnical investigation for the proposed independent living units (ILU) and residential age care facility (RACF) at Blacktown Workers Sports Club (BWSC), 170 Reservoir Road, Arndell Park NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Geoff Bentley of Paynter Dixon Constructions Pty Ltd (PDC) by Variation Order No. 1 dated 10 January 2018, which referenced Consultancy Services Agreement No. AA10690.

Based on the supplied 'Site Compatibility Certificate' architectural drawings prepared by Allen Jack & Cottier Architects (Job No. 15029, Drawing Nos. SK0001, SK0100, SK0900, SK1000, SK1001, SK1002, SK1003, SK2000, SK3000, SK5000 & SK8000, dated 22 February 2018), we understand that the proposed ILU and RACF development will comprise the construction of thirteen ILU buildings (Buildings A to L) and the RACF, which will ranged in height from four to fourteen stories. All buildings will be underlain by at least one basement level, as detailed below:

- Proposed Buildings A, B & C will overlie a common basement car parking level, which we expect will require excavation to a maximum depth of about 4.5m below existing grade;
- Buildings D & E will overlie a common basement car parking level, which we expect will require excavation to a maximum depth of about 3m below existing grade;
- Buildings F & G will overlie two common basement car parking levels, which we expect will require excavation to a maximum depth of about 6m below existing grade;
- Buildings H & I will overlie two common basement car parking levels, which we expect will require excavation to a maximum depth of about 6m below existing grade;
- Building J will be underlain by a basement car parking level, which we expect will require excavation to a maximum depth of about 3m below existing grade;
- Buildings K & L will overlie a common basement car parking level, which we expect will require excavation to a maximum depth of about 3m below existing grade;
- The RACF will be underlain by a basement car parking level, which we expect will require excavation to a maximum depth of about 4.5m below existing grade.

The footprints of the proposed basements will be set back at least 15m, 6m and 15m from the eastern (Reservoir Road), southern (Penny Place) and western site boundaries, respectively.

The proposed development also includes the construction of a new road network, as well as a pedestrian bridge linking Building C to the existing club building. Structural loads typical of this type of development have been assumed.

The purpose of the 'due diligence' investigation was to carry out a detailed walkover inspection of the site, and to assess the subsurface conditions at eight nominated borehole locations and at three nominated test pit locations. Based on the information obtained, we present our preliminary comments and recommendations on earthworks, excavation conditions, drainage, retention, footings, basement on-grade floor slab, external pavements, and additional investigations.

This report supersedes our previous 'due diligence' geotechnical investigation report, Ref. '28870ZArpt Rev2' dated 23 February 2016, which was prepared for a different proposed development.

Our environmental consulting division, EIS, was commissioned to carry out a Stage 1 environmental site assessment (report Ref. E28870KBrpt-rev3), which was carried out concurrently with the geotechnical investigation. This geotechnical investigation report must be read in conjunction with the EIS report.

2 INVESTIGATION PROCEDURE

2.1 Walkover Inspection

On 26 October 2015, our Senior Geotechnical Engineer (David Schwarzer) carried out a walkover inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. A summary of the observations made during the walkover inspection is presented in Section 3.1.

During this inspection, the nominated borehole and test pit locations were set out. Some of the locations were slightly modified to suit site conditions. A specialist sub-consultant reviewed available 'Dial Before You Dig' information and electro-magnetically scanned the borehole and test pit locations for buried services.

2.2 Borehole and Test Pit Investigation

The fieldwork for the investigation was carried out between 3 & 5 November 2015 and comprised the scope of work outlined below. The test locations are presented on the attached Figure 2.

- Eight boreholes (BH212 to BH219) were auger drilled to depths between 2.7m and 8.3m below existing grade using our truck mounted JK500 drill rig, which is equipped for site investigation purposes. The relative compaction/strength of the subsoil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clay soils recovered in the SPT split-spoon sampler and from the auger, and by tactile examination. The strength of the underlying bedrock was assessed by observation of auger penetration resistance when using a twin-pronged tungsten carbide (TC) bit, together with examination of recovered auger cuttings and correlations with subsequent laboratory moisture content test results. Groundwater observations were also made in the boreholes. On completion, each borehole was backfilled using the drill spoil and surface sealed with a concrete plug.

- Three test pits (TP220, TP228 & TP229) were excavated to depths of either 1.3m or 1.7m below existing grade using a backhoe with a 0.45m wide 'digging' bucket. Groundwater observations were also made in the test pits. On completion, each test pit was backfilled using the excavated spoil and compacted in layers by tamping with the bucket. Excess spoil was mounded above the backfill and compacted by rolling with the backhoe.

The borehole and test pit locations were set out using a combination of tape measurements from existing surface features and a hand held GPS. We expect that the positional accuracy of the test locations is within 4m. The surface RL's indicated on the attached borehole and test pit logs were interpolated between spot level heights and ground contour lines shown on the supplied preliminary survey plans prepared by Landpartners (Plan No. SY073782.000, 4 sheets, dated 29 October 2015), and are therefore only approximate. The survey datum is the Australian Height Datum (AHD).

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

Our engineering geologist was present full-time during the fieldwork to nominate testing and sampling, and to prepare the attached borehole and test pit logs. The Report Explanation Notes define the logging terms and symbols used. The (deep) borehole logs and (shallow) test pit logs have been presented at different vertical scales.

Selected soil and rock cutting samples were returned to a NATA registered laboratory, Soil Test Services Pty Ltd (STS), for moisture content, Atterberg Limits and linear shrinkage testing. The results are summarised in the attached STS Table A.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located in slightly undulating topography at the south-eastern corner of the BWSC grounds. The club building itself is located to the north of the site. Reservoir Road and Penny Place bound the site to the east and south, respectively. An internal asphaltic concrete (AC) surfaced access road ran along the northern site boundary.

At the time of our inspection, the site was occupied by two grass covered, terraced playing fields which had been formed by cut and fill earthworks, but predominantly by filling. The higher lying eastern field and the lower lying western field were separated by an approximately 1.5m high grass covered batter slope, which graded at about 27°. A stormwater inlet pit was located centrally along the eastern side of the upper field. Scattered small to large size trees lined the perimeter of the site, as well as internally along the crest of the above mentioned batter slope.

Along the northern two-thirds of the eastern site boundary was a sandstone block retaining wall, which supported the Reservoir Road footpath to a maximum height of 0.8m. The sandstone block wall appeared to be in good condition based on a cursory inspection. Beyond the southern end of the wall, the south-eastern corner of the site graded at a maximum of 9° between the road boundaries and the lower playing field surface.

For the remainder of the southern boundary (ie. western three-quarters), the playing field surfaces were higher in elevation than the Penny Place boundary. Here, the southern fill slope was up to 2m high and graded at about 27° down to the south. Along the western side of the playing fields, the fill batter slope was also up to 2m high and graded at about 32° down to the west.

Along the northern side, the fill batter slope was up to 1.5m high and graded to a maximum of about 20° down to the north. Beyond the toe of the northern fill batter slope was the AC surfaced internal access road, discussed above. Towards the western end of the northern fill batter slope was an AC surfaced car park located behind the crest of the slope, at a similar level to the adjacent playing field surface.

To the west of the site were two precast concrete panel warehouse buildings, which abutted the western site boundary. The neighbouring buildings appeared to be in good condition when viewed from within the subject site. The neighbouring surface levels were similar to the toe level of the western fill batter slope.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates the site to be underlain by Bringelly Shale of the Wianamatta Group. Generally, the boreholes and test pits encountered fill to variable depths, overlying residual silty clay, then shale and/or sandstone bedrock at generally shallow to moderate depths. Reference should be made to the attached borehole and test pit logs for specific details at each location. Graphical borehole summaries are presented as Figures 3 to 5. A summary of the subsurface characteristics encountered in the current boreholes and test pits is provided below:

Fill

Clayey fill was encountered in all eight boreholes and three test pits to depths between 0.4m (BH219) and 3.0m (BH212 & BH215). Inclusions of shale, sandstone and igneous gravel, shale cobbles, ash, brick, slag, roots and root fibres were found in the fill. The fill at all boreholes and test pits was grass covered. Based on the SPT results and the limited hand penetrometer readings, the fill was generally assessed to be moderately to well compacted. TP220, TP228 and TP229 were terminated within the fill profile at depths of either 1.3m or 1.7m.

Residual Silty Clay

Residual silty clay of predominantly high plasticity and of stiff to hard strength was encountered below the fill in all boreholes.

Bedrock

Shale or sandstone bedrock was encountered in all boreholes at the depths and RL's tabulated below:

Borehole	Depth to Bedrock (m)	Approximate RL of Bedrock Surface (mAHD)
BH212	4.5	54.7
BH213	3.6	56.2
BH214	3.0	60.0
BH215	5.2	54.0
BH216	2.3	57.5
BH217	2.4	59.9
BH218	3.5	58.5
BH219	1.8	61.6

The bedrock surface levels generally deepened in a westerly to north-westerly direction. In BH213, BH216 and BH217, the bedrock comprised sandstone. The sandstone was distinctly weathered and of low, medium and high strength. In all three boreholes, auger refusal occurred in high strength sandstone bedrock. The sandstone was only proven for a penetration length between 0.4m (BH216) and 1.6m (BH217).

In the remaining boreholes, the bedrock comprised shale. The shale was generally distinctly to slightly weathered and of low, medium and high strength. The upper bedrock profile in BH212, BH214 and BH215 was generally extremely to distinctly weathered and of extremely low and very low strength. This 'weak' profile was 0.5m thick in BH212, 0.4m thick in BH214, and 1.8m thick in BH215. In BH212, BH214, BH215, BH218 and BH219, auger refusal occurred in high strength shale bedrock.

Groundwater

On completion of drilling, groundwater was encountered in BH212 at 6.3m depth. All remaining boreholes were 'dry' during and on completion of drilling. Groundwater seepage was encountered in TP229 at 1.0m depth; presumably 'perched' groundwater within the fill profile. TP220 and TP228 were 'dry' during and on completion of excavation. We note that the groundwater levels may not have stabilised within the limited observation period. No long-term groundwater level monitoring was carried out.

3.3 <u>Laboratory Test Results</u>

The moisture content and Atterberg Limits test results confirmed our field classification of the site soils. The Atterberg Limits and linear shrinkage test results generally indicated the sampled residual silty clay of medium or high plasticity to have a moderate to high potential for shrink-swell reactivity with changes in moisture content.

The results of the moisture content tests carried out on recovered rock chip samples generally correlated poorly with our field assessment of bedrock strength. As such, our assessment of rock strength was based on observation of auger penetration resistance and examination of recovered auger cuttings. We note that there are limitations in assessing rock strength based on a combination of auger penetration resistance and tactile examination of recovered auger cuttings, and in some instances the assessed strength may vary from the actual strength by one order of rock strength.

4 COMMENTS AND RECOMMENDATIONS

The comments and recommendations provided below are generalised and are of a preliminary nature, and will need to be reviewed and most likely supplemented once the architectural and civil designs have been finalised.

4.1 Geotechnical Issues

We consider the following to be the primary geotechnical issues for the proposed ILU & RACF development:

- Potential groundwater seepage into the basement excavations.
- Presence of medium and high strength shale and sandstone bedrock for basement excavation.
- Presence of clay soils with a moderate to high potential for shrink-swell movements with changes in moisture content.
- Expected low CBR values for the clay subgrade.

The effects of the above geotechnical issues on design and construction are detailed in the sections which follow.

4.2 Site Preparation

4.2.1 Dilapidation Surveys

Prior to the commencement of any site works, we recommend that detailed dilapidation surveys be compiled on the neighbouring buildings to the west. Dilapidation reports provide a record of existing conditions and are used as a benchmark against which to set vibration limits during rock excavation, and for assessing possible future claims for damage arising from the works. The respective owners of the neighbouring properties should be asked to confirm in writing that the dilapidation reports present a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc).

The dilapidation reports should be reviewed by JK Geotechnics and the structural engineer prior to commencement of the works.

4.2.2 Vibration Monitoring

We recommend that quantitative vibration monitoring be carried out on the neighbouring warehouse buildings to the west at the commencement, and then periodically during rock excavation as a safeguard against possible vibration induced damage. The vibration monitoring locations should be assessed following review of the dilapidation survey reports, and should be jointly nominated by JK Geotechnics and the acoustic consultant.

The vibrations on the neighbouring warehouse buildings should be tentatively limited to a peak particle velocity of 20mm/s, subject to review of the dilapidation survey reports. If higher vibrations are recorded, then they should be measured against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the associated vibration frequency. Reference should be made to Section 4.4 if it is assessed that transmitted vibrations are excessive during rock excavation.

4.2.3 Stripping

Site preparation will include demolition of the AC surfaced car park, and stripping of all grass, topsoil, root affected soils and any deleterious fill or contaminated soil. Based on the results of the investigation, root affected soil should be stripped to a nominal depth of about 0.1m. We note that it is difficult to accurately assess the depth of topsoil and root affected soils in 100mm diameter boreholes and a limited number of test pits. If considered to be an important contractual issue, we recommend that a number of shallow test pits be excavated across the site to more accurately confirm the root affected soil stripping depth or alternatively a geotechnical inspection could be carried out after initial stripping to confirm the depth. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes, subject to approval by EIS. Reference should be made to the EIS report for guidance on the offsite disposal of soil.

Care must be taken not to undermine or remove support from the site boundaries during stripping and subsequent bulk excavation works.

4.3 Excavation Retention

4.3.1 Design Approach

Given the size of the proposed basement excavations, groundwater seepage should be expected. Discharge from the drainage system could be significant and therefore a dewatering license may need to be obtained from the relevant authorities such as Council and WaterNSW to allow temporary dewatering and discharge. These authorities impose limits on the amount of discharge allowed and analysis of the likely discharge may be required as part of the approval process. This would require the installation of standpipes to monitor groundwater levels, and testing to assess the permeability of the soil and rock profiles, as well as groundwater quality testing. Based on the groundwater level monitoring results and insitu permeability test results, inflows into the basement excavations can be estimated. Depending on the groundwater inflow rates a tanked basement may be required, such that the basement walls and possibly the lower basement floor slabs are designed to resist the hydrostatic pressures.

Based on the results of the investigation, it is considered unlikely at this stage that the shallower basement excavations (ie. the eastern 75% of the site) will need to be tanked. Nonetheless for confirmation, we strongly recommend that a groundwater investigation and seepage analysis be carried out as soon as possible. We could carry out this work, if commissioned to do so.

The comments and recommendations provided below are tentative and assume that a drained basement will be feasible. Following the results of the groundwater investigation and seepage analysis, the advice provided below will need to be reviewed and updated as appropriate.

Furthermore, we also recommend that all buried services located immediately outside the proposed basement walls be accurately located in both alignment and depth. This information should be plotted on the survey drawings for ease of reference. The locations of these services will need to be taken into account in the design of the basement walls.

4.3.2 Temporary Batter Slopes

Where space permits, the sides of the excavations can be temporarily battered back on condition that surcharge loads are kept well away from their crests of the batter slopes. For the soil profile and extremely low, very low and low strength bedrock profile, the cut faces should be temporarily battered or benched back at an overall grade of no steeper than 1 Vertical (V) on 1 Horizontal (H) for stability considerations. Any underlying medium and high strength bedrock can be tentatively cut vertically, on condition that the cut faces are progressively inspected by an experienced geotechnical engineer. Due to the likely presence of seams and steeply inclined joints, which could initiate rock wedge failures, we strongly recommend that all vertically cut rock faces be progressively inspected at no more than 1.5m depth increments to assess the need for temporary support (eg. rockbolts, etc.).

Free-standing cantilever walls can then be constructed at bulk excavation level, and then backfilled once lateral restraint is provided by the proposed buildings.

Where batter slopes cannot be accommodated or are not preferred, then further advice should be sought from JK Geotechnics.

4.3.3 Retention Design Parameters

Where 'weathered bedrock' is referred to below, it means all extremely low and very low strength shale and sandstone bedrock.

Free-standing cantilever basement walls, incorporated into and supported by the proposed new buildings, can be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient (K_0) of 0.55 for the soil and weathered bedrock profiles, assuming a horizontal backfill surface. An average bulk unit weight of 21kN/m^3 should be adopted for the soil and weathered bedrock profiles.

Assuming all vertical cut faces through underlying medium and high strength bedrock are inspected and stabilised, as appropriate, this lower more competent profile can be taken to be self-supporting.

Any surcharge affecting the basement walls (eg. traffic loading, inclined backfill, compaction stresses during backfilling, etc.) should be allowed in the design using the K_0 value provided above. The retaining walls should be designed as fully drained with measures undertaken to induce complete and permanent drainage of the ground behind the walls. Subsurface drains should incorporate (1) an appropriately sized 'ag' pipe with filter sock, surrounded by (2) free draining, single size, durable aggregate, such as 'Blue Metal' gravel or recycled concrete aggregate, and encapsulated within (3) a non-woven geotextile filter fabric such as Bidim A34 to control subsoil erosion. All drainage water should be piped to the stormwater system.

4.3.4 Backfilling Behind Basement Walls

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 ('Guidelines on Earthworks for Commercial and Residential Developments').

Backfilling behind free-standing basement walls must be carried out using engineered fill in order to reduce post-construction settlements. The excavated clay soils are suitable for reuse as engineered fill on condition that they are free of organic matter and contain a maximum particle size of 50mm. The excavated weathered shale bedrock will likely be too coarse for reuse as backfill, and should therefore be appropriately disposed off site. Alternatively, it could potentially be suitable for reuse as engineered fill where surface levels between the proposed buildings need to be raised.

Engineered fill comprising the excavated clay soils should be compacted in maximum 150mm thick loose layers using a hand operated vertical rammer compactor (also known as a 'Wacker Packer'), at least for the lower layers, trench roller and/or pad foot roller attachment fitted to an excavator to a minimum density ratio of 98% of Standard Maximum Dry Density (SMDD) and at a moisture content within 2% of Standard Optimum Moisture Content (SOMC). Where the backfill is located in proposed landscape areas, then the above specification can be relaxed to a minimum density ratio of 95% of SMDD.

Density tests should be regularly carried out on the engineered backfill to confirm the above specifications are achieved. The frequency of density testing for basement wall backfill should be at least one test per two layers per $50m^2$ (assumes maximum 150mm thick loose layers). Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by PDC, and not by the earthworks contractor or sub-contractors.

Compaction of engineered fill behind basement walls is very difficult. The use of a single sized durable aggregate, such as 'Blue Metal' gravel or crushed concrete aggregate (free of fines), which do not require significant compactive effort is often preferred if good performance is a priority; at least in the lower layers. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion. Provided the aggregate backfill is placed as recommended above, density testing would not be required. The geotextile should then be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of clay engineered fill.

4.4 Excavation Conditions

Prior to any excavation commencing, reference should be made to the Safe Work Australia 'Excavation Work Code of Practice' dated July 2015.

Excavation of the soils and extremely low, very low and low strength bedrock can be completed with a 'digging bucket' fitted to a large hydraulic excavator (say, at least 30 tonnes), using a ripping tyne where necessary, and/or by using a dozer. Notwithstanding, for such a large excavation, we expect that dozers will be mostly used. Hard ripping or 'hard rock' excavation conditions should be expected for the medium and high strength bedrock. Ripping may only just be possible with a Caterpillar D10 dozer and a very generous allowance would need to be made for hydraulic rock hammer assistance to the ripping. Notwithstanding, rock hammers will need to be used for detailed footing, lift pit and trench excavations.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to neighbouring buildings. As discussed in Section 4.2.2, we recommend that quantitative vibration monitoring be carried out on the neighbouring warehouse buildings at the commencement, and then periodically during rock excavation as a safeguard against possible vibration induced damage. If the vibration monitoring confirms that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a smaller rock hammer.

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

- Maintain rock hammer oriented towards the face and enlarge excavation by breaking small wedges off face.
- Operate hammer in short burst only, to reduce amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances and should be provided with a full copy of this report.

Once the architectural design has been finalised, additional geotechnical investigations comprising cored boreholes should be completed so that a more detailed assessment of rock excavation (particularly below the auger refusal depths) can be made. We can complete the cored boreholes and provide the advice on rock excavation, if commissioned to do so.

4.5 Drainage

Groundwater inflows into the excavations are expected to occur as local seepage flows from the fill profile, at the fill/residual silty clay interface, through gravel bands or relic joints/fissures within the residual silty clay, at the soil/rock interface, and through joints and bedding partings within the bedrock profile, particularly after heavy rain. Seepage volumes into the excavation are expected to be controllable by conventional sump and pump discharge systems. Piped discharge from the drainage system into the stormwater system can only be completed once the approvals have been obtained. The excavations should be monitored as they progress by PDC, JK Geotechnics and the hydraulic engineer to confirm the drainage requirements.

4.6 Footings

Based on the results of the 'due diligence' investigation, we expect that bedrock will be encountered either within, or a short distance below, the proposed basement excavation levels. Where bedrock is exposed, pad and strip footings founded in low strength or stronger shale and sandstone bedrock may be tentatively designed for a maximum allowable bearing pressure of 1000kPa. Elsewhere within the basement excavations, and for the proposed pedestrian bridge, conventional bored piles should be socketed at least 0.3m into low strength or stronger shale and sandstone bedrock and tentatively designed for a maximum allowable end bearing pressure of 1000kPa.

For bored piles, rock sockets formed below the 0.3m length requirement may be tentatively designed for maximum allowable shaft adhesion values of 100kPa in compression, and 50kPa in tension, on condition that the pile shafts are suitably roughened using a grooving tool fitted to the side of the auger. The upper 0.3m length should be ignored in shaft adhesion design.

The provided design pressures are based upon serviceability criteria of deflections at the footing base of less than 1% of the minimum footing dimension/pile diameter. We note that these footing settlements will be of an elastic nature and are expected to occur as construction proceeds.

For limit state design, an ultimate bearing capacity of 3000kPa and ultimate pile shaft adhesion value of 150kPa in compression could be tentatively adopted for the low strength or stronger bedrock. Settlement limitations to the structures will still need to be satisfied and can be estimated using an Elastic Modulus value of 200MPa for low strength or stronger bedrock. It should be noted that the ultimate bearing pressures must be used in conjunction with an appropriate '*Geotechnical Strength Reduction Factor*' (ϕ_g), as defined in Section 4.3 of AS2159-2009 'Piling Design and Installation'.

The medium and high strength bedrock is more than likely suitable for a higher bearing pressure, most likely in the order of 3500kPa (serviceability) and 15-30MPa (ultimate limit state), but is dependent on the amount of rock proving. In conjunction with the additional investigation recommended in Section 4.4 to further assess the excavability of the bedrock, the cored boreholes could also be used to attempt to optimise the bearing pressures for footing design.

All pad and strip footings should be cleaned out, inspected by a geotechnical engineer (prior to the installation of reinforcement cages) and poured on the same day as excavation. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering.

Conventional bored piles should be cleaned out, inspected and poured on the same day as drilling. All pile holes should be cleaned out using a cleaning bucket (for all pile diameters) for effective removal of sludge and loose material. Due to the expected groundwater seepage, the piles should only be cleaned out when concrete is ready to be tremie poured. For a design bearing pressure of 1000kPa, we recommend that the bored pile drilling be inspected by a geotechnical engineer during the initial stages and then periodically during the works.

4.7 Basement On-Grade Floor Slab

The advice provided below is tentative, assuming that drained basements will be permitted.

4.7.1 Bedrock Subgrade

The surface of the bedrock at bulk excavation level will need to be graded and/or trenched to provide good and effective drainage both during construction and in the long-term. The principal aim of the drainage is to promote run-off towards designated sumps by cross-falls and to reduce ponding. Any softened material must be scraped off prior to the placement of the sub-floor drainage layer.

4.7.2 Soil Subgrade

At the western end of the site, a soil subgrade should be expected based on the boreholes. The clay subgrade should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas. If subgrade heaving during rolling is encountered, then further geotechnical advice from JK Geotechnics should be sought with respect to subgrade improvement.

4.7.3 General

The proposed basement floor slabs should be separated from all walls, footings, etc. (ie. designed as 'floating') to permit relative movement. Slab joints should be capable of resisting shear forces but not bending moments by providing dowels or keys. Where basement floor slabs are supported on both soil and bedrock subgrades, they should be provided with joints at, or close to, the change in founding conditions. If this is not possible, then additional reinforcement should be provided to the slab to cater for the differential settlement.

The basement floor slabs should be provided with at least a 100mm thick sub-base of good quality, durable, single size, crushed rock (free of fines) such as 'Blue Metal' gravel or crushed concrete aggregate, which will also act as underfloor drainage.

The underfloor drainage should include a sump and pump dewatering system. The basement wall drains should be connected into the underfloor drainage system. Groundwater seepage monitoring should be carried out during basement excavation prior to finalising the design of the pump out facility. In order to avoid flooding, appropriately sized sumps each with an automatic level control pump will be required to intermittently discharge the seepage water to the stormwater system. Outlets into the stormwater system will require Council approval.

4.8 Internal Road Network

4.8.1 Earthworks

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

Site Drainage

The clay subgrade at the site is expected to undergo substantial loss in strength when wet. Furthermore, the clay subgrade is expected to have a moderate to high shrink-swell reactive potential. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principle aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may become untrafficable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

Subgrade Preparation

Following stripping, as discussed in Section 4.2.3, and excavation down to design subgrade level (if required), the exposed subgrade should be proof rolled with at least six passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

Subgrade heaving during proof-rolling may occur in areas where the clays have become 'saturated' and/or where deep under-compacted existing fill exists. If subgrade heaving during rolling is encountered, then further geotechnical advice from JK Geotechnics should be sought with respect to subgrade improvement.

If soil softening occurs after rainfall periods, then the clay subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface must be moistened with a water cart and rolled until the shrinkage cracks are no longer evident. Care must be taken not to over-water the subgrade as this will result in softening.

Engineered fill must be reused to raise site levels.

Engineered Fill

From a geotechnical perspective, the excavated clay fill, residual silty clay and weathered bedrock are considered suitable for reuse as engineered fill on condition that they are ‘clean’, free of organic matter and contain a maximum particle size of 100mm. Excavated low, medium and high strength bedrock, and any boulders and over-size fragments removed from the existing fill profile, will most likely need to be crushed in order to meet the maximum particle size specification. It is common place for earthworks contractors to attempt to break down over-size particles with numerous passes of large pad-foot rollers. However, this only results in over-compaction and potential failure of the compaction specification provided below.

Engineered fill comprising the excavated above mentioned material should be compacted in maximum 300mm thick loose layers using a large static pad-foot roller (say, at least 15 tonnes deadweight) to a minimum density ratio of 98% of SMDD and at a moisture content within 2% of SOMC. For such an earthworks project, moisture conditioning (ie. ‘drying out’ or ‘wetting up’) of the clay soils should be expected. Our preference is for static (non-vibratory) rolling for fill compaction so as limit the potential for ground borne vibration damage to nearby buildings.

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of the compaction plant that can be placed in trenches, backfilling should be carried out in maximum 150mm thick loose layers and compacted using a trench roller, a pad foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a ‘Wacker Packer’). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also reduce to 50mm. The compaction specification provided above is applicable.

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved, as outlined below:

- The frequency of density testing for engineered fill should be at least one test per layer per 1000m^2 or one test per 200m^3 distributed reasonably evenly throughout the full depth and area, or 3 tests per visit, whichever requires the most tests (assumes maximum 300mm thick loose layers).
- The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 150mm thick loose layers).

Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. Due to a potential conflict of interest, the GTA should be directly engaged by PDC, and not by the earthworks contractor or sub-contractors.

Warning

The long-term successful performance of the road 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. This should be detailed in the tender documents.

4.8.2 Permanent Batter Slopes

Where permanent batter slopes of soil cuts or of fill embankments are proposed, we recommend that they be graded at no steeper than 1V on 2H. Surface erosion protection, for example, quick establishing grass or proprietary systems (such as those provided by Geofabrics Australasia or Global Synthetics) should be provided to the permanent batter slopes. Dish drains should also be provided along the crest of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system.

4.8.3 Pavement Design

Based on an established correlation between plasticity index, linear shrinkage and CBR, and our experience elsewhere within the BWSC grounds, we recommend that the proposed new external pavements be tentatively designed for a CBR value of 2% or a short-term Young’s modulus of 16MPa for the compacted clay subgrade.

The additional investigation should include Standard compaction and soaked CBR tests on representative subgrade materials in order to confirm the design CBR value.

4.9 Additional Geotechnical Input

We summarise below the previously recommended additional work that needs to be carried out:

1. Groundwater investigations and seepage analyses for the proposed basement excavations.
2. Additional geotechnical investigations once the architectural and civil designs are finalised.
3. Test pit investigation, or geotechnical inspection during initial stripping, to confirm topsoil depths.
4. Review of dilapidation survey reports.
5. Vibration monitoring during rock excavation.
6. Progressive rock face inspections as the excavation proceeds.
7. Proof-rolling inspections of the soil subgrade.
8. Footing inspections.
9. Groundwater monitoring of seepage volumes in the basement excavations.

5 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes and test pits 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.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Sports Facilities , Residential Age Care Facility and Childcare Centre
Location: Reservoir Road, Arndell Park, NSW

Ref No: 28870AD
Report: A
Report Date: 18/11/2015

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 %
201	0.50-0.95	27.6	61	20	41	15.0
201	2.50-3.00	9.4				
201	3.50-4.00	12.6				
202	0.50-0.95	24.6	67	25	42	16.0
202	2.50-3.00	7.2				
202	3.00-3.60	13.9				
203	0.50-0.95	14.3	48	18	30	13.5
203	2.50-3.00	7.3				
203	4.00-4.50	10.4				
204	0.50-0.95	23.2	54	19	35	15.0
204	3.60-4.00	13.3				
204	5.50-6.00	15.7				
204	7.00-7.50	8.1				
205	0.50-0.95	22.1	41	16	25	11.0
205	4.10-4.50	14.4				
205	6.70-7.10	6.1				
206	4.00-4.50	7.4				
206	7.00-7.50	8.3				
207	10.00-10.50	14.7				
208	6.00-6.45	21.8	74	25	49	16.5
208	7.00-7.50	5.3				
208	8.50-9.00	7.9				
209	7.00-7.50	8.6				
209	7.60-7.80	10.5				

Notes: See Page 2 of 2

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Sports Facilities , Residential Age Care Facility and Childcare Centre
Location: Reservoir Road, Arndell Park, NSW

Ref No: 28870AD
Report: A
Report Date: 18/11/2015

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 %
210	0.50-0.95	11.8	33	16	17	7.0
210	1.60-2.00	5.7				
210	3.00-3.40	8.6				
210	3.40-3.60	11.8				
211	0.50-0.95	12.7	32	15	17	7.0
211	1.70-1.80	9.8				
212	3.00-3.45	17.9	36	14	22	7.5
212	4.50-4.60	10.8				
212	5.50-6.00	6.4				
212	7.20-7.70	6.9				
213	0.50-0.95	11.8	42	18	24	10.0
213	4.00-4.30	6.9				
214	1.50-1.95	20.3	64	23	41	16.0
214	4.00-4.30	9.1				
215	5.50-6.00	10.4				
215	7.00-7.50	7.8				
215	8.00-8.30	6.0				
216	1.50-1.95	24.1	60	22	38	15.5
217	2.80-3.00	4.9				
217	3.60-4.00	7.9				
219	0.50-0.95	15.9	54	20	34	14.5
219	2.50-3.00	7.0				
219	4.00-4.30	9.9				

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: 11/11/2015

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
212
1/2

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD | **Method:** SPIRAL AUGER JK500 | **R.L. Surface:** ≈ 59.2m
Date: 3-11-15 | **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, medium palsticity, dark brown, with roots and root fibres, trace of fine to medium grained sand and ash.	MC<PL			GRASS COVER
	N = 8 3,3,5			FILL: Silty clay, medium plasticity, red brown, with fine to coarse grained sandstone and igneous gravel, trace of ash.	MC>PL		550 500 500	APPEARS WELL COMPACTED
	N = 15 2,8,7	1–2					350 380 400	
	N = 15 5,7,8	3	CL	SILTY CLAY: medium plasticity, red brown and brown, trace of coarse grained ironstone gravel.	MC>PL	H	500 450 480	
	N = SPT 16/100mm REFUSAL	4.5	-	SHALE: brown	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
		5		SHALE: brown and grey.	DW	M		MODERATE RESISTANCE
ON COMPLETION & AFTER 30 MINS		6–7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
212
2/2

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 59.2m
Date: 3-11-15
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SHALE: grey.	SW	H		MODERATE TO HIGH RESISTANCE
									END OF BOREHOLE AT 7.4m				'TC' BIT REFUSAL
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
213
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 59.8m
Date: 3-11-15
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, medium plasticity, light brown, with roots and root fibres.	MC<PL			GRASS COVER
		N = 14 5,7,7				FILL: Silty clay, medium plasticity, light brown, with shale gravel, trace of root fibres and ash.			>600 >600 >600	APPEARS WELL COMPACTED
			1							
		N = 11 2,5,6				FILL; Silty clay, high plasticity, brown.	MC>PL		150 150 150	APPEARS MODERATELY COMPACTED
			2							
					CH	SILTY CLAY: high plasticity, red brown, with fine to coarse grained ironstone gravel.	MC>PL	VSt		
		N = 16 4,8,8	3						220 250 250	
					-	SANDSTONE: fine to coarse grained, brown.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
			4					H		MODERATE TO HIGH RESISTANCE
						END OF BOREHOLE AT 4.3m				'TC' BIT REFUSAL
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **214**
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 63.0m
Date: 4-11-15
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, low plasticity, dark brown, with root fibres.	MC<PL			GRASS COVER
	N = 8 4,4,4			FILL: Silty clay, medium plasticity, dark brown, with fine to medium grained shale gravel, trace of ash and root fibres.				APPEARS MODERATELY COMPACTED
	N = 13 4,5,8	1.5	CH	SILTY CLAY: high plasticity, red and orange brown, trace of root fibres.	MC<PL	H	450 450 480	
	N = SPT 12/150mm REFUSAL	3	-	SHALE: brown.	XW-DW	EL-VL		VERY LOW 'TC' BIT RESISTANCE
					DW	M		LOW TO MODERATE RESISTANCE
		4.3		END OF BOREHOLE AT 4.3m				'TC' BIT REFUSAL

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
215
1/2

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 59.2m
Date: 3-11-15 **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION	N = 9 4,4,5	0		FILL: Silty clay, medium plasticity, light brown, with roots, root fibres and fine grained sand, trace of fine to coarse grained igneous gravel and ash.	MC<PL			GRASS COVER APPEARS MODERATELY COMPACTED
	N = 6 2,3,3	1		FILL: Silty clay, medium plasticity, red brown and dark brown, trace of ash.	MC>PL		250 250 250	
		2		FILL: Silty clay, high plasticity, red brown and dark brown, trace of ash.				
	N = 8 3,4,4 N = 18 3,6,12	3	CH	SILTY CLAY: high plasticity, orange brown mottled grey.	MC>PL	VSt	250 220 220 400 250 350	
		5	-	SHALE: brown and grey.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
		6						
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
215

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 59.2m
Date: 3-11-15 **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SHALE: grey.	DW-SW	L-M		LOW TO MODERATE RESISTANCE
						8				SW	M		
									END OF BOREHOLE AT 8.3m				'TC' BIT REFUSAL
						9							
						10							
						11							
						12							
						13							
						14							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **216**
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** SPIRAL AUGER JK500 **R.L. Surface:** ≈ 59.8m
Date: 3-11-15 **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low plasticity, light brown, with roots and root fibres.	MC<PL			GRASS COVER
		N = 9 4,4,5				FILL: Silty clay, high plasticity, brown, with fine to coarse grained ironstone gravel, trace of ash, roots and root fibres.	MC>PL		350 350 300	APPEARS WELL COMPACTED
			1							
		N = 9 2,4,5			CH	SILTY CLAY: high plasticity, red brown, trace of root fibres.	MC>PL	VSt	300 320 300	
			2							
					-	SANDSTONE: fine to coarse grained, light brown.	DW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE
						END OF BOREHOLE AT 2.7m				'TC' BIT REFUSAL
			3							
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
217
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Date: 4-11-15
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 62.3m
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium plasticity, dark brown, with roots and root fibres.	MC<PL			GRASS COVER
	N = 8 5,5,3			FILL: Silty clay, medium plasticity, dark red brown, with fine to coarse grained sandstone gravel, trace of ash.	MC≈PL		600 580 570	APPEARS WELL COMPACTED
	N = 6 2,3,3	1	CH	SILTY CLAY: high plasticity, orange brown mottled grey, trace of root fibres.	MC>PL	St	200 150 180	
		2	-	SANDSTONE: fine to medium grained, brown.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
		4		END OF BOREHOLE AT 4.0m				'TC' BIT REFUSAL
		5						
		6						
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
218
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Date: 4-11-15
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 62.0m
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium plasticity, dark brown, with root fibres.	MC<PL			GRASS COVER
	N = 8 3,4,4			FILL: Silty clay, medium plasticity, dark brown, with fine to medium grained ironstone gravel, trace of ash.	MC>PL		350 350 350	APPEARS MODERATELY COMPACTED
	N = 11 5,4,7			FILL: Silty clay, high plasticity, red brown, trace of ash.			>600 >600 >600	APPEARS WELL COMPACTED
	N = 18 5,7,11		CL	SILTY CLAY: medium plasticity, red brown mottled grey, trace of fine to medium grained ironstone gravel.	MC>PL	VSt -H	450 450 400	
			-	SHALE: grey, with iron indurated bands.	DW	L-M		LOW 'TC' BIT RESISTANCE
		4		END OF BOREHOLE AT 4.0m				'TC' BIT REFUSAL

Samples: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
219
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD
Method: SPIRAL AUGER JK500
R.L. Surface: ≈ 63.4m
Date: 4-11-15
Datum: AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium plasticity, dark brown, with roots and root fibres.	MC<PL			GRASS COVER
	N = 12 3,5,7 N > 8 5,8/100mm REFUSAL		CH	SILTY CLAY: high plasticity, orange brown.	MC<PL	H	500 520 480 450 550 550	
		2	-	SHALE: brown and grey.	DW	L		LOW 'TC' BIT RESISTANCE
				SHALE: grey.	SW	M		LOW TO MODERATE RESISTANCE
				END OF BOREHOLE AT 4.3m				'TC' BIT REFUSAL

Samples: ES, U50, DB, DS

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

TEST PIT LOG

Test Pit No. **220**
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** BACKHOE **R.L. Surface:** ≈ 63.4m
Date: 5-11-15 **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low plasticity, dark brown, with roots and root fibres, trace of ash.	MC<PL			GRASS COVER
						FILL: Silty clay, low plasticity, light brown, with fine to medium grained shale gravel, trace of root fibres.				
			0.5			FILL: Silty clay, high plasticity, light grey and orange brown, with root fibres, trace of ash.	MC>PL			
						FILL: Silty clay, high plasticity, dark orange brown, trace of fine to medium grained ironstone gravel and ash.				
			1							
			1.5							
						END OF TEST PIT AT 1.7m				
			2							
			2.5							
			3							
			3.5							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

TEST PIT LOG

Test Pit No. **228**

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD

Project: PROPOSED DEVELOPMENT

Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** BACKHOE **R.L. Surface:** ≈ 59.1m

Date: 5-11-15 **Datum:** AHD

Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Silty clay, low plasticity, dark brown, with roots and root fibres, trace of ash.	MC<PL			GRASS COVER
									FILL: Silty clay, low plasticity, light brown, with fine to medium grained shale gravel, trace of ash, slag and brick fragments.				
						0.5							
						1			FILL: Silty clay, high plasticity, light grey and red.	MC>PL			
									END OF TEST PIT AT 1.3m				
						1.5							
						2							
						2.5							
						3							
						3.5							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

TEST PIT LOG

Test Pit No. **229**
1/1

Client: PAYNTER DIXON CONSTRUCTIONS PTY LTD
Project: PROPOSED DEVELOPMENT
Location: RESERVOIR ROAD, ARNDELL PARK, NSW

Job No. 28870AD **Method:** BACKHOE **R.L. Surface:** ≈ 59.8m
Date: 5-11-15 **Datum:** AHD
Logged/Checked by: L.M./D.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						0			FILL: Silty clay, low plasticity, dark brown, with fine grained sand, roots and root fibres, trace of ash.	MC<PL			GRASS COVER
									FILL: Silty clay, high plasticity, light brown, trace of ash.	MC>PL			
						0.5			FILL: Silty clay, high plasticity, light grey and orange brown, with fine to medium grained shale gravel and cobbles, trace of ash.				
▶						1			FILL: Silty clay, medium plasticity, dark brown, trace of ash and organic matter.				
						1.5			END OF TEST PIT AT 1.3m				
						2							
						2.5							
						3							
						3.5							

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

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

Title: **SITE LOCATION PLAN**

Project: BLACKTOWN WORKERS SPORTS CLUB
170 RESERVOIR ROAD, ARNDELL PARK, NSW

Report No: 28870A6

Figure No: 1

JK **Geotechnics**

PLOT DATE: 8/03/2018 3:42:51 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\28000'S\28870AD ARNDELL PARK\CAD\28870A6.DWG

LEGEND

- BOREHOLE
- TEST PIT

0 12 24 36 48 60
SCALE 1:1200 @A3 METRES

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

Title: **TEST LOCATION PLAN**

Location: BLACKTOWN WORKERS SPORTS CLUB
170 RESERVOIR ROAD, ARNDELL PARK, NSW

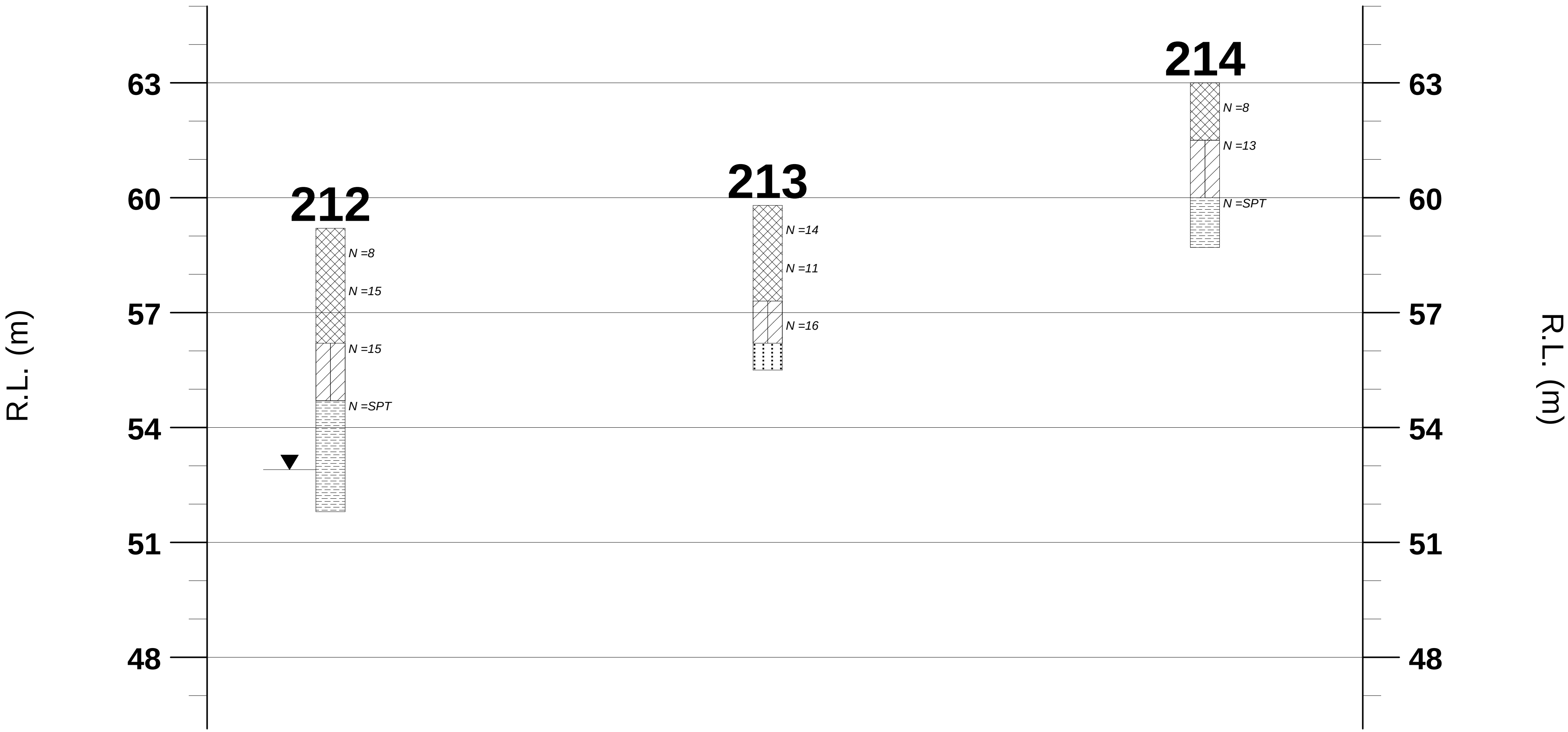
Report No: 28870A6

Figure No: 2

JK **Geotechnics**

PLOT DATE: 8/03/2018 4:23:04 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\28000'S\28870AD ARNDELL PARK\CAD\28870A6.DWG

GRAPHICAL BOREHOLE SUMMARY



Fill
Silty Clay
Shale
Sandstone/ Greywacke
Observed water level
N SPT "N" VALUE
Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

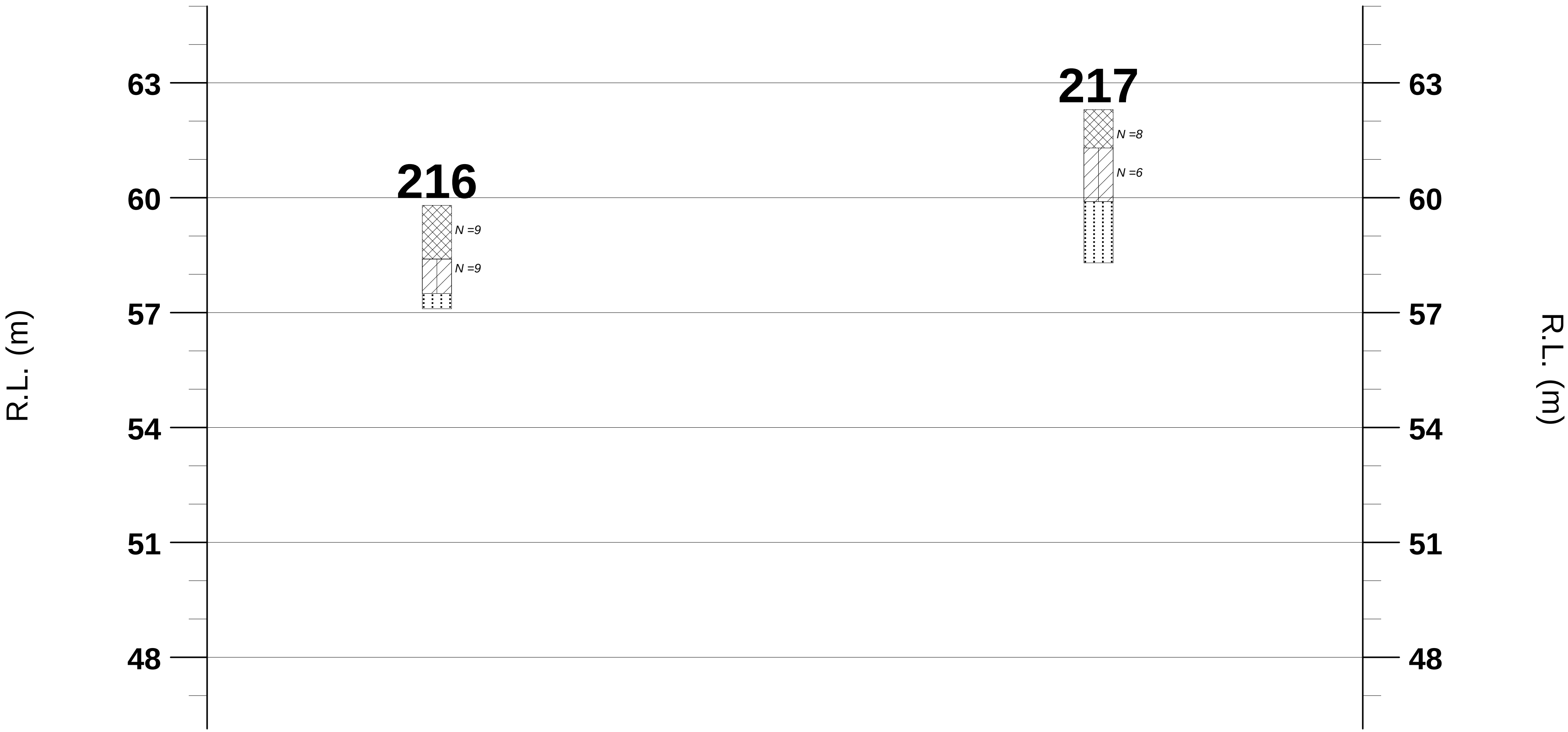
Scale: 1 : 150 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 28870A6

Figure No.: 3

GRAPHICAL BOREHOLE SUMMARY



Fill		N	SPT "N" VALUE
Silty Clay		Nc	SOLID CONE BLOW COUNTS PER 150mm
Sandstone/ Greywacke			

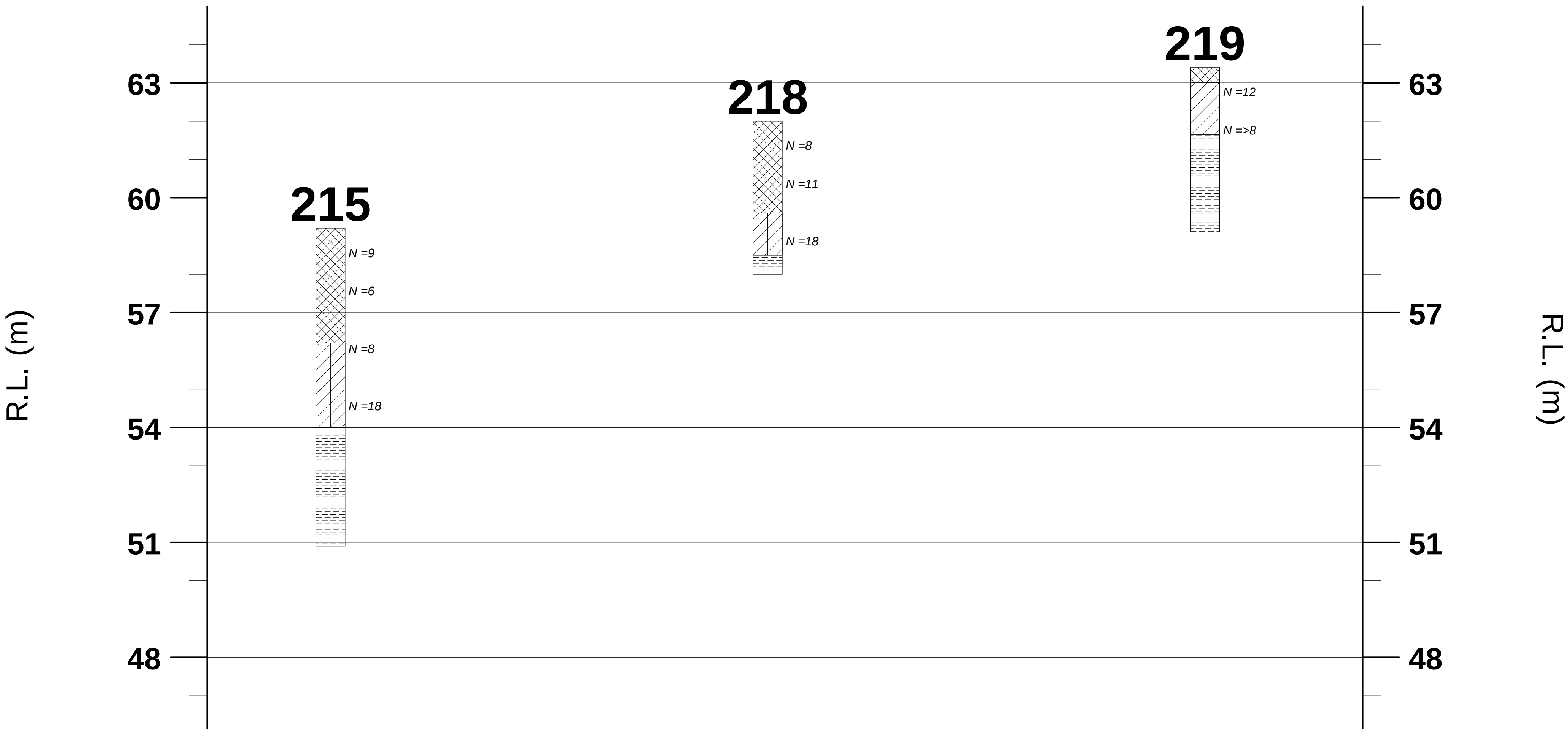
NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 150 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 28870A6 **Figure No.:** 4

GRAPHICAL BOREHOLE SUMMARY



Fill		N	SPT "N" VALUE
Silty Clay		Nc	SOLID CONE BLOW COUNTS PER 150mm
Shale			

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 150 (vert) ; NTS (horiz)

JK Geotechnics

Job No.: 28870A6 **Figure No.:** 5

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

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

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

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

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

The attached explanatory notes define the terms and symbols used in preparation of the logs.

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

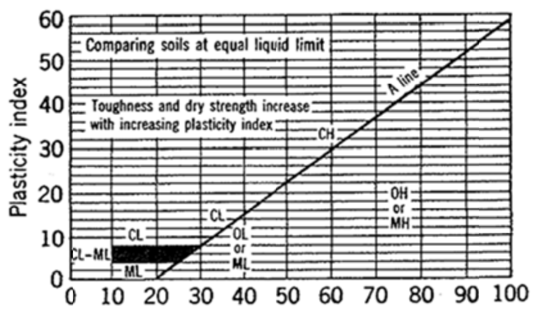
Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)				Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 μm sieve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

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

Use grain size curve in identifying the fractions as given under field identification

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols

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



Plasticity chart for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	C	Extent of borehole collapse shortly after drilling.
	►	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY – Runs freely through fingers.
	M	MOIST – Does not run freely but no free water visible on soil surface.
	W	WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – Unconfined compressive strength less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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