REPORT

TO
AUSTINO PROPERTY GROUP

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

17 October 2017
Ref: 30809YFrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed residential development at 2 Murray Rose Avenue, Sydney Olympic Park, NSW. The investigation was commissioned by Mr Will Wang of Austino Sydney Olympic Park Pty Ltd by signed 'Acceptance of Proposal' form dated 28 August 2017. The commission was in accordance with our proposal, Ref P45452YFrev2, dated 28 August 2017.

At the time of writing this report, no drawings were available, however from our correspondence with Russel Strahle of Austino, we understand it is proposed to construct multi storey residential development with up to two basement levels. The Bulk Excavation Level (BEL) for the lower basement is approximately RL-0.4m resulting in excavation between about 8m and 11m depth.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation conditions, retention systems, footings, basement slabs, hydrogeological considerations and earthquake design parameters.

2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 9 and 10 September 2017, and comprised the auger drilling of four boreholes (BH1 to BH4) to depths between 5.68m and 6.77m below existing surface levels using a Tungsten Carbide ('TC') bit. These boreholes were then extended to depths ranging from 10.75m to 14.34m using an NMLC triple tube barrel fitted with a diamond coring bit.

The strength of the subsurface soils was assessed from Standard Penetration Test (SPT) 'N' values augmented by hand penetrometer tests on the SPT split tube samples. The strength of the shale bedrock was assessed by observation of the auger penetration resistance using a tungsten carbide 'TC' drill bit, together with examination of the recovered rock cuttings and from correlations with subsequent moisture content test results on recovered rock chips. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Selected samples were returned to Soil Tests Services (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, soil pH, sulphate contents, chloride contents and resistivity. The results of the laboratory testing are summarised in STS Table A and Envirolab Services Certificate of Analysis No. 175631.

Where bedrock was diamond cored, the recovered core was returned to our NATA registered laboratory (Soil Test Services (STS)) for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is_{50} results. These results are presented inthe attached Table B. Copies of the colour photographs are provided with the borehole logs.

Groundwater observations were made during and on completion of auger drilling. The use of water for coring limited further groundwater level measurements. Slotted PVC standpipes were installed in BH2 and BH4 on completion of drilling to allow longer term groundwater monitoring. On 19 September 2017, one week after the completion of fieldwork groundwater levels were measured within the wells. No longer term groundwater monitoring was carried out.

The fieldwork was completed in the full-time presence of our geotechnical engineer, who set out the borehole locations, nominated the testing and sampling and prepared the attached borehole logs. The site and borehole locations are shown on the attached Figures 1 and 2, respectively. A graphical borehole summary has been provided as Figure 3. The boreholes were set out by taped measurements from assumed site boundaries and features as shown on survey plans prepared by Craig and Rhodes (Ref: 258-14, Dwg. No. 258 14G T01 [01], Issue 01 dated 30/01/2015). The relative levels shown on the attached logs were interpolated from spot heights shown on the survey plan and are therefore approximate. The height datum used is the Australia Height Datum (AHD). For more details of the investigation procedures and their limitations, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within undulating topography that typically slopes down to the east and the mangroves associated with Powells Creek at about 4°. The surface levels of the site itself appear to have been altered by past cut and fill earthworks.

At the time of investigation, the site was an active construction site with site sheds and gravel access ways predominantly over the western portion of the site and an asphaltic concrete (AC) paved car park over the eastern portion. As a result of the infrastructure, the investigation was limited to the northern half of the site as access was not possible to the southern portion of the site. The site slopes down from the west towards the east along an asphaltic concrete covered driveway and car parking area.

The site is bound to the east by Bennelong Parkway. Running parallel to the site boundary but offset from the boundary by about 10m is a gabion wall. At its northern end this wall swings back to the north-eastern corner of the site, where it has a maximum height of about 5m and runs a short length along the northern site boundary. Between the site boundary and the gabion wall a battered, tree lined slope grades down to the wall at about up to 20°.

To the north is Murray Rose Avenue that follows the natural hillside slope down towards the east. The adjoining western property at the time of the fieldwork was an active construction site which had several meters of basement excavation completed. The adjacent area to the south of the subject site is an open grassed area beyond which is Parkway Drive. The gabion wall running along Bennelong Parkway swings back along Parkway Drive for a short length.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates the site is underlain by Ashfield Shale of the Wianamatta Group. The investigation revealed a generalised subsurface profile comprising sandy and clayey fill over residual silty clay and shale bedrock. The bedrock was generally of extremely low to very low strength when first encountered before improving to high strength at depth. Reference should be made to the attached borehole logs for detailed subsurface conditions at specific locations. A graphical borehole summary is presented in Figure 3 and a summary of the subsurface conditions encountered is presented below:

Fill

Fill was encountered at the surface in all boreholes and extended to depths ranging from 1.2m (BH2 and BH3) to 1.4m (BH1), or between RL6.3m and RL9.2m. The fill comprised silty and sandy clays with varying amounts of fine to coarse grained igneous, sandstone and shale gravel. Based on the SPT results the fill appears to be variably compacted ranging from moderately to well compacted across the site. Asphaltic concrete (AC) pavements varying in thickness between 30mm and 50mm were encountered at BH2, BH3 and BH4.

Residual Soils

Natural residual clays were encountered below the fill and extended to the underlying shale bedrock. The natural silty clays were generally of medium to high plasticity and were assessed to be of very stiff to hard strength. The clays contained varying amounts of fine to medium grained ironstone and shale gravel.

Weathered Shale

Weathered shale bedrock was encountered in all boreholes at depths between 1.7m (BH4) and 2.2m (BH2), or between RL5.7m to RL8.5m. On first contact the bedrock was extremely weathered and of extremely low to very low strength, improving to slightly weathered to fresh and of medium to high strength below 6.75m to 7.69m depth. Please note, occasional very high strength bedrock bands were encountered at depth within BH1 and BH3.

Defects within the shale bedrock comprised bedding partings, extremely weathered seams varying in thickness up to 65mm and jointing inclined at between 20° to 90°. The jointing was generally quite widely spaced, although the highly fractured zones encountered within BH3 and BH4 may reflect joint swarms within the bedrock that has caused the fracturing. The bedrock within these fractured zones were generally of low to medium strength but due to the fractured nature, logging of individual defects was not feasible. The top of a possible joint swarm may also have been encountered near the termination depth of BH4.

In addition to the individual defects observed, zones of core loss were also logged in BH1, BH3 and BH4. Core loss typically represents zones of clay or poorer quality rock that has been washed away during the drilling process. It should be noted that while the core loss and the fractured zone within BH4 may have formed by the coring process which may have washed away the extremely weathered bedrock and clay seams situated between the better quality low to medium strength rock. However, due to the limitation of the coring process we have placed the core loss at the beginning of the core run but the loss may actually be distributed throughout the fractured zone.

Groundwater

Groundwater was not encountered whilst auger drilling the boreholes. The injection of large volumes of water during the coring process precluded further useful measurements of groundwater level on the day of the investigation.

Groundwater monitoring wells were installed in BH2 and BH4 and standing water levels were measured at depths of 7.7m (RL2.3m) in BH2 and 5.9m (RL1.9m) in BH4, on 19 September 2017.

3.3 Laboratory Test Results

The moisture content and point load strength index test results showed reasonably good correlation with our field assessment of rock strength. The estimated Unconfined Compressive Strength (UCS) of the rock core ranged from 6MPa and 86MPa.

The results of the pH, sulphate content, chloride content and resistivity are summarised in the following table:

Sample	pH	Sulphate (mg/kg)	Chloride (mg/kg)	Resistivity (ohm.cm)
BH1 0.5m-0.95m	8.3	500	33	2,800
BH2 1.5m-1.95m	6.8	88	20	14,000
BH3 0.5m-0.95m	6.7	380	35	2,800
BH4 1.5m-1.7m	4.8	450	90	2,800

4 COMMENTS AND RECOMMENDATIONS

4.1 Principal Geotechnical Findings, Issues and Further Work

As discussed in more detail in Section 3.2, the boreholes penetrated fill and residual clays overlying weathered shale bedrock at depths ranging from 1.7m to 2.2m, or about RL5.7m and RL8.5m. The upper portion of the shale bedrock was typically extremely weathered and of extremely low to very low strength before improving to medium and high to very high strength with depth. Standing water was measured on 19 September 2017 in BH1 and BH4 at depths of 5.9m to 7.7m, or about RL1.9m to RL2.3m. It is unclear whether sufficient time had passed following the fieldwork for water levels to equilibrate for the standing water levels measured to reflect actual groundwater levels.

Based on the results of the boreholes and our understanding of the proposed development (refer to Section 1), we have summarised the principal geotechnical findings, issues and recommendations to be considered in the planning, design, and construction of the development.

1. Prior to demolition or excavation, we recommend detailed dilapidation surveys be completed on any neighbouring structures that fall within the zone of influence of the excavation, taken as a horizontal distance extending out from the top of the shoring system and equal to twice the excavation depth.

2. Excavation for the proposed basement will be through pavements, fill, natural clays and then predominantly shale/laminite bedrock of medium and high strength, possibly with some bands of very high strength. Excavation of the shale will require the use of "hard rock" excavation equipment for effective excavation, which may transmit vibrations through the rock mass that may adversely affect adjoining movement sensitive structures.

3. The retention systems may comprise a full depth soldier pile wall with shotcrete infill panels socketed not less than 0.5m below the Bulk Excavation Level. However, alternative retention systems, such as secant pile walls, may need to be considered to address hydrogeological issues. Temporary lateral support for the piles will be required and is anticipated to comprise temporary ground anchors and/or internal bracing and propping. Permanent support is anticipated to be provided by the completed structure of the building.

4. Considering the proposed bulk excavation level of RL-0.4m and the proximity of the site to Homebush Bay it is likely that excavation will extend below the existing groundwater table. However, as discussed above, the standing water levels measured in BH1 and BH4 some weeks after the completion of the investigation may not represent groundwater levels across the site. Given the relatively low permeability of the soils and bedrock, we expect that during construction seepage rates will be manageable using conventional sump and pump methods. However, seepage rates may be greater than allowed by the relevant authorities and the basement may, in the long term, need to be designed as a tanked structure. The retention system should be reviewed following further analysis of expected groundwater seepage into the excavation and the proposed basement construction techniques.

5. Pad footings founded within the shale bedrock at the base of the excavation can be used to support the proposed development. Suitable geotechnical inspections and testing of the footing excavations will have to be scheduled.

Further comments on these issues and geotechnical design parameters are provided in the subsequent sections of this report.

4.1.1 Further Work

As part of the detailed design stages of the proposed development, we consider that the following additional geotechnical investigation and input will be required:

- Review of this report once architectural and structural drawings are available.
- Determine the details and extent of the existing basement within the adjacent western property (currently under construction) and assess what impact, if any, this will have on the proposed basement.
- Additional cored boreholes if higher footing bearing pressures are required.
- Further groundwater monitoring and pump out tests to assess the permeability and expected groundwater inflows into the excavation. Additional computer analysis using specialist geotechnical software such as SeepW may also be required. Pump out testing can be completed from within the current monitoring wells installed as part of this investigation.
- Dilapidation surveys for the neighbouring structures and infrastructure, especially as percussive excavation techniques (ie. rock hammers) will almost certainly be used.
- Analysis of potential retention system deflections depending on Council requirements.
- At least initial quantitative vibration monitoring during percussive (i.e. hydraulic rock impact hammers) excavation.
- Inspections during piling for the retention system to confirm founding conditions.
- Progressive inspection of excavated cut faces to confirm if additional support or treatment is required.
- Witnessing installation and proof testing of anchors.
- Footing inspections and testing

Given no drawings have been issued at the time of writing this report, we recommend a review by a geotechnical engineer after the initial structural design has been completed to confirm that our recommendations have been correctly interpreted and that we have understood the proposed scope of work. It is possible that further advice/input will be required during the structural design stage to address issues that may not have been addressed in this report. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

4.2 Excavation Conditions

All excavation recommendations should be complemented by reference to the latest edition of Safe Work Australia's 'Excavation Work Code of Practice'.

4.2.1 Dilapidation Surveys

Prior to the commencement of excavation, we recommend that dilapidation surveys be completed on any neighbouring buildings or infrastructure within the zone of influence of the excavation. The zone of influence is taken as a horizontal distance extending out from the top of the shoring system of at least twice the excavation depth. As the excavation ranges from about 8m to 11m deep, this zone of influence extends about 16m to 22m. Apart from the neighbouring western property, which is currently under construction, the remaining nearest structures are at least 15m distance and so a reduction in the detail of the dilapidation reports for these structures may be considered. We consider that it would also be prudent to carry out dilapidation surveys on the adjoining footpaths and roadways surrounding the site.

The dilapidation surveys should include internal and external inspection of the buildings, where all defects including defect location, type, length and width are described and photographed. The respective owners of the buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works.

4.2.2 Excavation Methods

The proposed basement will have a BEL of RL-0.4m resulting in up to about 11m of excavation. The excavations will encounter pavements, fill, natural clays and shale bedrock of up to high strength, with possibly some very high strength bands. Excavation of the soils and shale bedrock varying up to low strength will be achievable using conventional excavation techniques, such as large hydraulic excavators (i.e. 25 tonnes or larger) equipped with buckets and possibly requiring some light ripping from a dozer or ripping hook fitted to the excavator. Excavation of shale bedrock of greater than low strength or higher will represent 'hard rock' excavation conditions and will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping hooks and rock saws.

The excavator contractor should be made aware of this by being supplied with all geotechnical information, particularly the borehole logs and point load strength test results. Low productivity and increased equipment wear should be expected due to the rock strength. We recommend that a copy of this report be provided to the excavation contractor so that they can make their own assessment of excavation conditions.

4.2.3 Vibration Monitoring

Subject to review of the dilapidation reports, vibrations, measured as Peak Particle Velocity (PPV), should be limited to no higher than 5mm/sec whenever hydraulic rock impact hammers are used. If required and to provide some reduction in the transmission of vibrations a perimeter vertical saw cut may be provided through the shale bedrock and the base of the slot maintained at a lower level than the adjoining rock excavation at all times. However, it should be noted thatthis generally only has a marginal impact on the magnitude of transmitted vibrations and the effectiveness of this technique should be confirmed on-site.

Rock excavation using hydraulic rock hammers will need to be strictly controlled as there is likely to be direct transmission of ground vibrations to nearby structures and buried services. We recommend that, as a minimum initial quantitative vibration monitoring be carried out when using hydraulic rock hammers to determine if transmitted vibrations fall within an acceptable limit for the nearby structures and services. Whether periodic or continuous vibration monitoring is required depends on the level of assurance required by the builder, the proximity of nearby movement sensitive structures and the condition of these structures. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations. Where the transmitted vibrations are excessive, alternative excavation methods will be required and may include reducing the size of rock hammers or the use of non-percussive excavation techniques such as rotary grinders, ripping tynes, rock saws etc. Where non-percussive excavation techniques are adopted quantitative vibration monitoring is not required.

Where percussive excavation techniques are adopted the following procedures are recommended to reduce the magnitude of transmitted vibrations:

- Maintain rock hammer orientation towards the face and enlarge the excavation by breaking small wedges out of the face.
- Operate the rock hammer in short bursts only, to reduce amplification of vibrations.
- Maintain a sharp moil.

Alternatively, rock excavations using low vibration emitting equipment, such as rock saws and rock grinders fitted to a hydraulic excavator may be used. If rock saws or rock grinders are used, the resulting dust should be suppressed with water. Use of this low vibration emitting equipment would reduce the likelihood of vibration induced damage to the neighbouring structures and services. With the use of the low vibration equipment we do not consider that it will be necessary to carry out any quantitative vibration monitoring, although we recommend that at least an initial site visit at the commencement of rock excavation be carried out by a geotechnical engineer to inspect the excavation methods and procedures being adopted.

The use of excavation contractors with appropriate experience and with a competent supervisor who is aware of vibration damage risks is also recommended. The contractor should have all appropriate statutory and public liability insurances.

4.3 Retention Systems

Excavation through the soils and shale bedrock of less than medium strength will not be self-supporting and some form of retention will need to be installed prior to the start of excavation. The medium to high and high strength shale is generally self-supporting, provided it is free of adverse defects. However, all boreholes encountered a number of joints that may be adversely inclined. In addition, it is known that shales are susceptible to the presence of large continuous inclined joints which can adversely affect the stability of excavations. Therefore we do not recommend vertical unsupported excavations within the shales. Inclined joints within the shale may not become apparent until bulk excavation level is reached and at that time it may be too late to install the necessary lateral support to retain the rock wedges isolated by the inclined joints. Any such instability would undermine the existing piles and potentially lead to major failure of the exposed rock and potentially the piles and shoring system above.

Consequently, we recommend that a full height soldier pile wall founded a minimum embedment of at least 0.5m below bulk excavation level be adopted. A greater embedment may be necessary to satisfy overall stability and founding considerations. Furthermore, we recommend the shale cut faces between the soldier piles be progressively inspected by a geotechnical engineer at no more than 1.5m depth increments to check for the presence of any potentially unstable rock wedges and, in particular, large scale features isolated by large continuous joints. Where present the need for further stabilisation works can be assessed, e.g. additional anchors, rock bolts, dowels, etc. A provision should be made in the contract documents (budget and program) for the above inspections and potential stabilisation measures.

During the excavation, reinforced shotcrete panels should be progressively installed as the excavation deepens to support the weathered shale between the piles such that there is no more than 1.5m of vertical face exposed below the base the shotcrete panel above at any one time. It will be necessary to install strip drains behind each shotcrete panel to dissipate the pore pressures from immediately behind the shotcrete facing and in this regard strip drains should be installed at spacings of no greater than 1.5m.

Due to the presence of medium, high and very high strength bedrock of up to 86MPa, only high torque drilling rigs suitable for these conditions and equipped with rock augers should be used on this site. We strongly recommend that a full copy of this report be provided to prospective piling contractors.

The details and extent of the basement within the neighbouring western property must be determined. From our site observations we know the site has basement levels but do not know the extent of these basements. Depending on their location it may not be feasible to install ground anchors which will have a significant impact on the design of temporary lateral support for the shoring wall. Alternatively, if the neighbouring basement extends to the common boundary and the proposed basement similarly extends to the boundary retention along this boundary will not be required. However, if this is the case and the subject site excavation extends below the neighbouring basement underpinning of the adjoining retention system and footings may be required. We recommend further advice be obtained once details of the neighbouring basement are known.

4.3.1 Retaining Wall Design Parameters

Propped or anchored retaining walls may be designed using a trapezoidal earth pressure distribution of 6H kPa or 8H kPa, where H is the retained height of soils and weathered shale of less than medium strength. A pressure of 8H kPa should be used adjacent to movement sensitive buildings and services, while a pressure of 6H kPa may be used where some movement of the shoring system can be tolerated. The trapezoidal pressure distribution should comprise a pressure of either 6H or 8H kPa (depending on the amount of deflection permissible, as discussed above) over the middle 50% that then tapers off to zero over the upper and lower 25% of the pressure distribution.

Where the retention system supports the medium and high strength shale the design philosophy must allow for the presence of potentially adversely orientated defects whilst at the same time allowing the most efficient system to be constructed should adverse defects not be present. In this regard we recommend that, as a minimum, the retention system be designed for a uniform pressure of 10kPa to support small potentially unstable localised wedges of rock. However, as discussed above, weathered shales have the potential for large continuous defects. Therefore the retention system must also be designed to have appropriate capacity to support a large sliding wedge of rock inclined at about 45° to the horizontal, daylighting just above BEL and with an effective friction angle of 25°. Whilst from a design perspective it is envisaged that while the piles would be designed to have sufficient capacity to restrain such a large scale feature, the additional rows of anchors would not be installed unless such a feature were present. In this regard it is very important that regular inspections by a geotechnical engineer be completed every 1.5m of vertical cut as the excavation deepens so that if such a feature is present it is identified appropriate retention measures may be adopted. . .

Appropriate surcharge loads (such as adjoining buildings, traffic, sloping backfill, footing loads etc.) are additional to the above earth pressures and should be allowed for in the design. The additional earth pressures from surcharge loads may be calculated using an 'at rest' earth pressure coefficient (K_o) of 0.5.

Hydrostatic pressures should also be accounted for in the design, since the strip drains will only be effective in reducing pore pressures immediately behind the shotcrete facing and may not reduce the pore pressures from behind potentially larger adverse failure wedges.

Passive toe resistance of the retention system below the base of the bulk excavation, where piles extend below the base of the excavation, may be estimated based on a maximum allowable lateral resistance of 400kPa for shale of medium or higher strength. The upper 0.5m of socket (taken from below the base of the excavation, including footing, lift pit and service excavations) should be ignored when calculating the passive resistance, due to the potential for fracturing of the upper shale during bulk excavation.

Anchors should have their bond length formed within shale of at least medium strength and may be provisionally designed based on an allowable bond stress of 350kPa. The anchor bond should be formed below a line drawn up at 45° from the bulk excavation level, with a minimum free length of 4m and a minimum bond length of 3m. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally anchors are installed on a design and construct contract so that optimisation of bond stresses does not become a contractual issue in the event of an anchor failing the test load. We have assumed that the final lateral support will be provided by the floor slabs for the proposed structure.

If temporary anchors extend below neighbouring properties, permission from the adjoining owners must be obtained prior to installation. We recommend that requests for permission commence early in the construction process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping, although this is unlikely to be the most economic means of support.

Specific shoring wall analysis should be undertaken, including an assessment of the likely ground movements beyond the shoring walls. The shoring wall design engineers should then be requested to provide comment on whether such movements will be problematic to any adjoining structures or services.

4.4 Footings

Based on the borehole results and the indicative rock classifications in the table below, it can be seen that following bulk excavation for the proposed basement, at least Class II and probably Class I shale bedrock will be exposed across the basement footprint. Therefore pad/strip footings founded on at least Class II shale would be feasible. For such footings, we consider that an allowable bearing pressure of 6000kPa may be adopted. In order for such a bearing pressure to be adopted, all pad/strip footing excavations must be inspected by a geotechnical engineer and at least 50% of all pad/strip footings must be spoon tested. Spoon testing involves drilling a 50mm diameter hole through the base of the footing excavation to a depth of at least 1.5 times the minimum footing width. The side of the hole is then probed by the geotechnical engineers to check for adverse defects in the rock portion immediately below the footing base. Even higher bearing pressures may be feasible, say up to 8,000kPa, however given the current number of boreholes, we have limited the recommended allowable bearing pressure to 6000kPa. Nevertheless if higher bearing pressures are being considered then additional cored boreholes at specific footing locations would be required.

The reduced level for the top of each rock class for each borehole at this site are provided in the following table. We note the classification is dependent upon pile diameter and pad footing width, and this classification has been based upon representative lengths of core and some judgement within the overlying augered portions of each borehole and should be treated as approximate only. Further, within each rock class given below, there may be some subsections of rock which may be say one class higher or lower than the overall class of that band. Therefore, further confirmation of the classification must be obtained when further details of the pile diameter/socket length and shallow footing details are known.

Borehole	Approx. Surface RL (mAHD)	Indicative Depths (m) to Top of Bedrock Unit (Reduced Level mAHD)				
		Class V	Class IV	Class III	Class II	Class I
BH1	10.6	2.10* (RL8.50)	-	6.75 (RL3.85)	-	7.66 (RL2.94)
BH2	10.0	2.20* (RL7.80)	5.20* (RL4.80)	-	-	7.41 (RL2.59)
BH3	7.5	1.80* (RL5.70)	3.50* (RL4.00)	-	7.69 (RL-0.19)	9.00 (RL-1.50)
BH4	7.8	1.70* (RL6.10)	3.00* (RL4.00)	-	-	7.53 (RL0.27)

***** indicates partially or wholly assessed from augered portion of the borehole and should be treated as approximate only

NOTE: Rock Classification in accordance with Foundations on Sandstone and Shale in the Sydney Region, Pells, Mostyn and Walker, Australian Geomechanics, Dec 1998

The 6000kPa is a serviceability bearing pressure in which settlements are predicted to be kept to below 1% of the minimum footing width. If limit state design is to be adopted, then ultimate end bearing values with an appropriate geotechnical reduction factor calculated in accordance with the methodology presented in AS2159-2009 may also be used. We consider that an ultimate bearing pressure of 60MPa may be adopted for Class II shale. As discussed above this is also somewhat conservative but reflects the limited number of boreholes. We note that if ultimate values are to be adopted, then more rigorous analysis will be required to predict likely footing settlements. The use of ultimate values can result in settlements in excess of 5% of the minimum footing width.

Piles socketed into the Class III or better quality shale may also be designed for an allowable skin friction of 350kPa for that portion of the socket within the Class III shale. The sides of piles designed for skin friction must be appropriately roughened to at least Roughness Class R2 in accordance with Pells et al 1998.

Assuming some seepage will occur into the basement excavation, the rock exposed in the base of the footing excavations will likely weather if left exposed and inundated with water. Therefore we recommend that footing excavations be inspected, tested and poured with minimal delay, preferably on the same day as excavation. If a delay in pouring the footing is expected, we recommend that a thick blinding layer of concrete be placed in the base of the footing for protection.

4.5 Hydrogeological Considerations

A standing water level was measured in the two installed wells at between RL1.9m and RL2.3m. As discussed above, these levels may not represent the actual groundwater table. However, due to the location of the site, it is likely that BEL will extend below the groundwater table. As such, we expect groundwater flow into the basement excavation during construction and over the life of the building. Given the assumed relatively low permeability of the natural clays and shale bedrock, we expect that during construction seepage will be able to be controlled using conventional sump and pump techniques.

Given the basement excavations will likely extend below the natural groundwater table, we recommend pump out tests be completed to assess the expected permeability of the shales. Once the basement dimensions are known this should be followed by some seepage analysis to assess the likely annual groundwater inflow rates. The relevant authorities (such as WaterNSW) have specific requirements on the quantity and quality of water that can be pumped from the site during construction and over the life of the building. It is likely a dewatering licence will need to be obtained for temporary dewatering during construction but this will be dependent on the size of the proposed basement. In the long term we suspect that the volume of water will probably exceed WaterNSW's limits for a drained basement. Where a drained basement is not permitted the basement will need to be tanked and designed to resist appropriate hydrostatic uplift pressures.

Further groundwater monitoring is recommended during the detailed design stages of the project to allow appropriate hydrostatic uplift pressures to be nominated for design purposes. It would be advisable to install some groundwater data loggers in boreholes so that changes in water level with rainfall can be monitored.

If a tanked basement is required, consideration must be given to the type and depth of any shoring system, including drainage behind the shoring walls and whether a tanked basement is constructed within the temporary shoring system. It should be noted that if a secant pile wall or similar is adopted this will prohibit geotechnical inspections at 1.5m intervals as the basement excavation deepens. Where this is the case the design of the retention system must be based on the assumption that a continuous 45° joint extends upwards from just above BEL forming a large sliding wedge.

4.6 Subgrade Preparation and Engineered Fill

Earthworks recommendations in this report should be read in conjunction with AS3798-2007: '*Guidelines on Earthworks for Commercial and Residential Developments*' which should also be adopted.

We currently are not aware whether any subgrade preparation will be required at this stage and so the following has been provided for information only. These comments and recommendations must be reviewed by this office once architectural and structural drawings are available.

Where engineered fill is to be placed over the exposed subgrade, we recommend that the following subgrade preparation be followed:

- Strip the subgrade of all existing pavements, vegetation, root affected soils and other deleterious materials.
- Following stripping, proof roll the subgrade with a minimum of 6 passes using a smooth drum non-vibratory roller of no less than 8 tonnes static weight. All proof-rolling should be completed in the presence of an experienced geotechnical engineer or geotechnician.
- The purpose of proof rolling is to improve the near surface density of the soils and identify any soft or unstable areas. Any soft or unstable areas identified should be excavated down to a sound base and reinstated with engineered fill as described below.

Engineered fill should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 70mm. We expect that, if required the excavated soils will be used as engineered fill and the recommendations provided below are based on this premise. Engineered fill should be placed in layers of maximum 200mm loose thickness and compacted to between 98% and 102% of standard maximum dry density (SMDD) where structures are proposed to be supported. In areas of soft landscaping this may be reduced to between 95% and 102% of SMDD. Engineered fill should also be compacted to ±2% of Standard Optimum Moisture Content (SOMC).

Density tests should be carried out at a frequency of one test per layer per 500m^2 or three tests per visit, whichever requires the most tests, to confirm the above specification has been achieved. For backfilling of localised excavations, such as service trenches or localised soft spots, testing should consist of one test per two layers per 50m^2. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

4.7 Basement Floor Slabs

Based on the investigation results the exposed subgrade below the basement slab will comprise good quality shale bedrock. We recommend the basement slab be underlain by a layer of durable igneous granular material such as DGB20 or other approved material which will act as a separation layer between the rock and the basement slab.

If a drained basement is permitted then drainage should be provided around the basement perimeter and below the lowest basement slab to direct seepage into sumps with permanent and fail safe automatic pumps to maintain the basement in a dry state. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage is sufficient for the actual seepage flows. A full drainage blanket may be necessary. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel.

4.8 Exposure Classification

Based on Envirolab Services test results, for concrete piles an exposure classification of ''Mild' applies in accordance with Table 6.4.2(C) of AS2159-2009. For steel piles a 'Non-Aggressive' exposure classification applies in accordance with Table 6.5.2(C) of AS2159-2009.

4.9 Earthquake Design Parameters

Based upon AS1170.4-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia", the following design parameters may be adopted:

- Hazard Factor (Z) = 0.08;
- Class Be - Rock Site

5 GENERAL COMMENTS

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

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

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

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

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT TEST REPORT

Client:	JK Geotechnics	**Ref No:**	30809YF
Project:	Proposed Residential Development	**Report:**	A
Location:	2 Murray Rose Avenue, Sydney Olympic Park, NSW	**Report Date:**	18/09/2017
		Page 1 of 1	

AS 1289	**TEST METHOD**	**2.1.1**
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**
1	2.50-3.00	6.0
1	4.00-4.50	8.9
2	2.80-3.00	7.8
2	5.50-6.00	6.1
3	1.80-1.85	12.6
3	3.50-4.00	6.1
4	1.70-1.80	12.5
4	3.50-4.00	7.9

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Residential Development
Location: 2 Murray Rose Avenue,
Sydney Olympic Park, NSW

Ref No: 30809YF
Report: B
Report Date: 13/09/2017
Page 1 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	6.80-6.83	0.6	12
	7.26-7.29	1.3	26
	7.74-7.77	1.0	20
	8.25-8.28	0.8	16
	8.69-8.72	0.7	14
	9.30-9.33	0.6	12
	9.70-9.74	0.7	14
	10.23-10.25	0.6	12
	10.74-10.77	0.7	14
	11.30-11.33	1.0	20
	11.77-11.81	0.9	18
	12.23-12.26	1.2	24
	12.77-12.81	2.3	46
	13.27-13.31	2.6	52
	13.80-13.84	2.5	50
	14.18-14.23	3.2	64
2	6.79-6.82	0.4	8
	7.23-7.26	1.0	20
	7.78-7.81	0.7	14
	8.28-8.30	0.7	14
	8.79-8.82	0.6	12
	9.24-9.27	0.9	18
	9.77-9.79	0.7	14
	10.30-10.33	0.7	14
	10.74-10.77	0.7	14
	11.21-11.24	1.5	30
	11.67-11.70	2.2	44
	12.35-12.39	2.2	44
	12.71-12.74	2.7	54
	13.29-13.32	2.4	48

NOTES: See Page 2 of 2

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Residential Development
Location: 2 Murray Rose Avenue,
Sydney Olympic Park, NSW

Ref No: 30809YF
Report: B
Report Date: 13/09/2017
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
2	13.78-13.81	2.8	56
	14.12-14.15	2.2	44
3	6.24-6.28	0.4	8
	6.91-6.94	0.6	12
	7.18-7.21	0.3	6
	7.75-7.77	0.6	12
	8.12-8.15	0.6	12
	8.66-8.69	0.6	12
	9.39-9.42	1.1	22
	9.81-9.84	2.0	40
	10.15-10.19	3.1	62
	10.56-10.58	1.5	30
4	7.57-7.60	0.7	14
	7.90-7.93	0.7	14
	8.35-8.38	1.1	22
	8.81-8.84	0.9	18
	9.15-9.18	2.8	56
	9.69-9.73	3.0	60
	10.27-10.29	4.3	86
	10.68-10.71	3.0	60
	11.06-11.09	2.9	58

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\,(50)}$

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CERTIFICATE OF ANALYSIS 175631

Client Details

Client	JK Geotechnics
Attention	A Kourtesis
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**30809YF, Sydney Olympic Park**
Number of Samples	4 soils
Date samples received	14/09/2017
Date completed instructions received	14/09/2017

Analysis Details

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

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

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

Report Details

Date results requested by	21/09/2017
Date of Issue	19/09/2017

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

David Springer

David Springer, General Manager

Client Reference: 30809YF, Sydney Olympic Park

Misc Inorg - Soil					
Our Reference		175631-1	175631-2	175631-3	175631-4
Your Reference	UNITS	BH3	BH4	BH1	BH2
Depth		0.5-0.95	1.5-1.7	0.5-0.95	1.5-1.95
Date Sampled		09/09/2017	09/09/2017	10/09/2007	10/09/2017
Type of sample		Soil	Soil	Soil	Soil
Date prepared	-	15/09/2017	15/09/2017	15/09/2017	15/09/2017
Date analysed	-	18/09/2017	18/09/2017	18/09/2017	18/09/2017
pH 1:5 soil:water	pH Units	6.7	4.8	8.3	6.8
Chloride, Cl 1:5 soil:water	mg/kg	35	90	33	20
Sulphate, SO4 1:5 soil:water	mg/kg	380	450	500	88
Resistivity in soil*	ohm m	28	28	28	140

Client Reference: 30809YF, Sydney Olympic Park

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

Client Reference: 30809YF, Sydney Olympic Park

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			18/09/2017	1	15/09/2017	15/09/2017		18/09/2017	[NT]
Date analysed	-			18/09/2017	1	18/09/2017	18/09/2017		18/09/2017	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	6.7	7.6	13	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	35	42	18	92	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	380	380	0	107	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	28	27	4	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
1
1 / 3

BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF **Method:** SPIRAL AUGER **R.L. Surface:** ~10.6 m
Date: 10/9/17 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.S./O.F.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■ ■			■	N = 12 3,6,6	10	1	▨		FILL: Silty sandy clay, medium plasticity, orange brown, fine to coarse grained sand, with fine to coarse grained igneous, sandstone and shale gravel.	MC<PL			APPEARS MODERATELY TO WELL COMPACTED
	■			■	N = 7 3,4,3	9	2	▨	CH	SILTY CLAY: high plastiicty, grey mottled orange brown, with fine to coarse grained ironstone and shale gravel.	MC<PL	H	410 450 520	RESIDUAL
								▤		SHALE: grey.	XW	EL - VL		VERY LOW 'TC' BIT RESISTANCE
				■ ■		8 7 6	3 4 5	▤		as above, but dark grey.	DW	L		LOW RESISTANCE
				■		5		▤				L - M		LOW TO MODERATE RESISTANCE
						4	6			REFER TO CORED BOREHOLE LOG				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
1

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Core Size: NMLC
R.L. Surface: ~10.6 m
Date: 10/9/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: K.S./O.F.

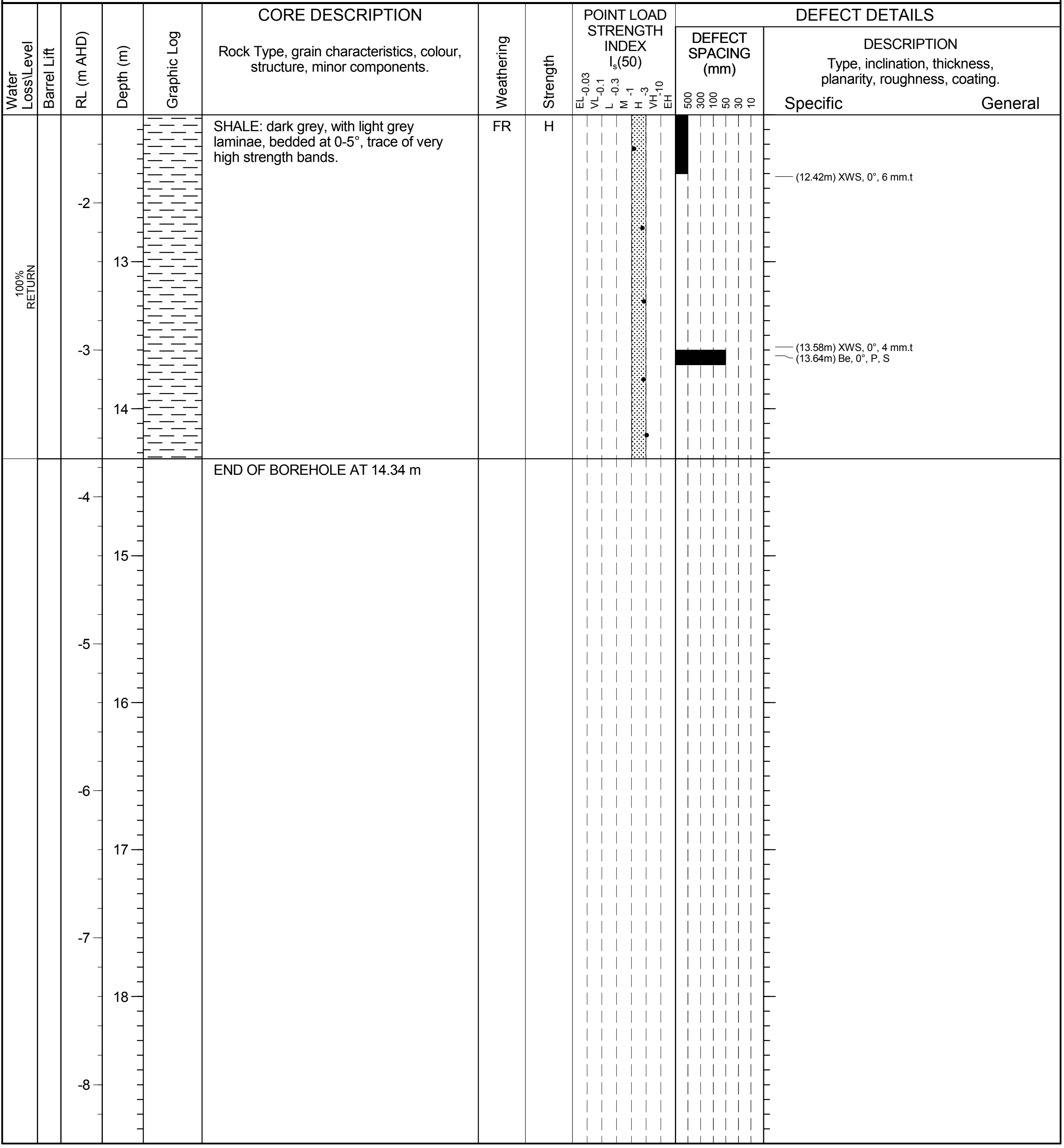
Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
		5			START CORING AT 5.89m					
100% RETURN		4	6		CORE LOSS 0.86m					
		3 2 1 0 -1	7 8 9 10 11		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	SW	M - H			(6.87m) Be, 0°, P, S (7.20m) Be, 0°, P, S (7.29m) Jx 2, 90°, P, S (7.47m) Be, 0°, P, S (7.50m) Be, 0°, P, S (7.66m) Be, 0°, P, S (8.62m) XWS, 0°, 22 mm.t (9.37m) J, 70°, P, S (9.95m) J, 90°, P, S (9.98m) Be, 0°, P, S (10.06m) J, 70 - 90°, Un, S (10.66m) Be, 0°, P, S (11.05m) Be, 0°, P, S (11.68m) Be, 0°, P, S

Borehole No.
1

CORED BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Core Size: NMLC
R.L. Surface: ~10.6 m
Date: 10/9/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: K.S./O.F.



Water Loss\Level	Barrel Lift	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		SHALE: dark grey, with light grey laminae, bedded at 0-5°, trace of very high strength bands.	FR	H	(12.42m) XWS, 0°, 6 mm.t (13.58m) XWS, 0°, 4 mm.t (13.64m) Be, 0°, P, S	
		END OF BOREHOLE AT 14.34 m				


JK Geotechnics
30809YF
BH1
START CORING AT 5.89m
6
CORE LOSS 0.86m
7
8
9
10
11
12
13
14
END OF BH1 AT 14.34m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
1 / 3

BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF **Method:** SPIRAL AUGER **R.L. Surface:** ~10.0 m
Date: 10/9/17 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** K.S./O.F.

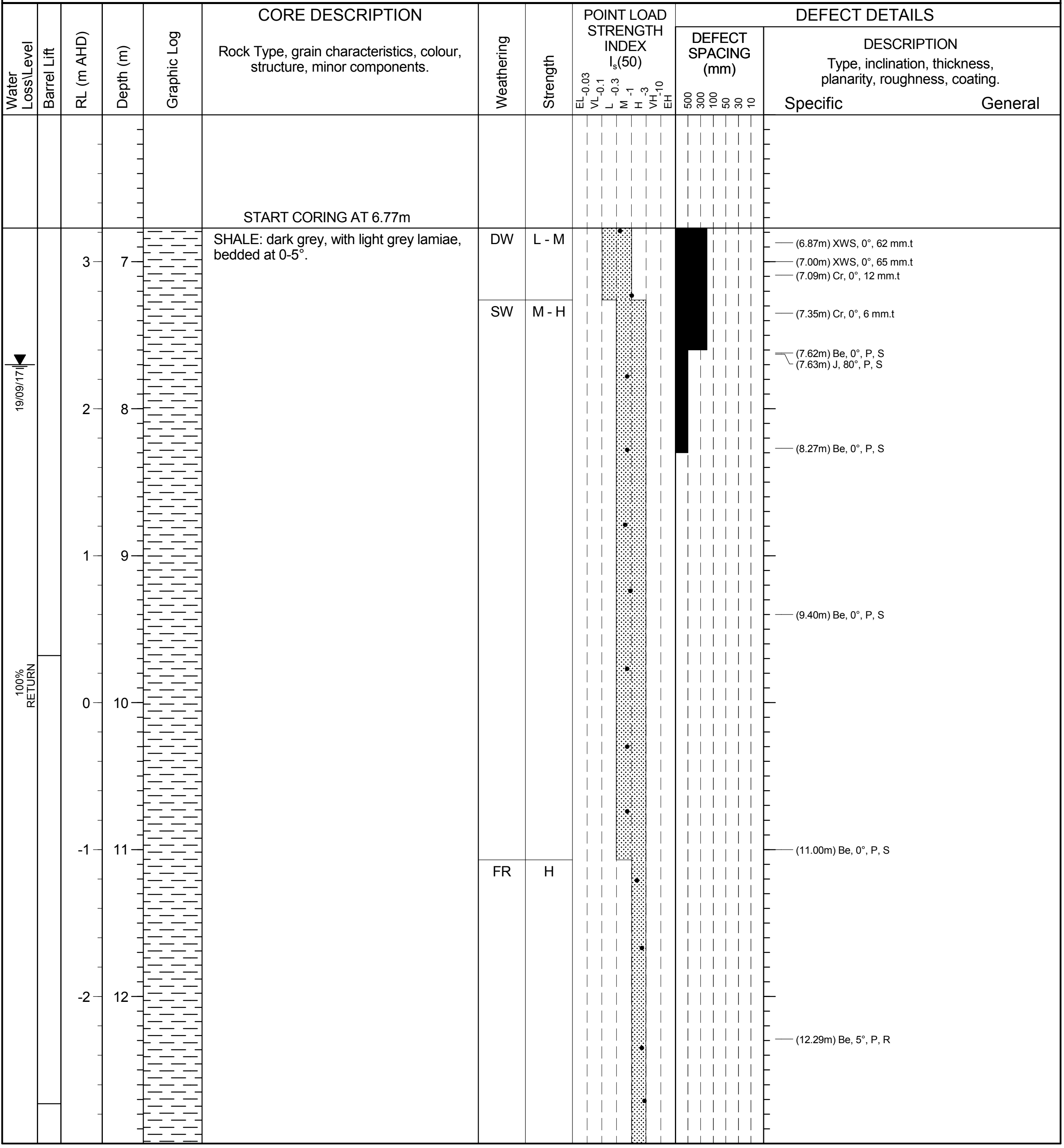
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■									ASPHALTIC CONCRETE : 30mm.t	M			APPEARS MODERATELY TO WELL COMPACTED
										FILL: Gravelly sand, fine to coarse grained, dark grey, fine to coarse grained igneous gravel.	MC~PL			
	■			■	N = 7 3,4,3	9	1			FILL: Silty clay, low to medium plasticity, dark grey and brown, with fine to coarse grained igneous and shale gravel.				
	■			■	N = 7 4,3,4	8	2		CH	SILTY CLAY: high plasticity, light grey mottled red brown, with fine to coarse grained ironstone gravel.	MC<PL	VSt - H	450 390 470	RESIDUAL
										SHALE: grey, with iron indurated bands.	XW	EL - VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
				■ ■		7 6	3 4			as above, but with XW bands.	DW	L		LOW TO MODERATE RESISTANCE
						5	5				XW	EL		VERY LOW RESISTANCE
				■						SHALE: dark grey.	DW	L - M		LOW RESISTANCE
				■		4	6							LOW TO MODERATE RESISTANCE
										REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
2
2 / 3

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF	**Core Size:** NMLC	**R.L. Surface:** ~10.0 m
Date: 10/9/17	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK300	**Bearing:** N/A	**Logged/Checked By:** K.S./O.F.



START CORING AT 6.77m

SHALE: dark grey, with light grey lamiae, bedded at 0-5°.

(6.87m) XWS, 0°, 62 mm.t
(7.00m) XWS, 0°, 65 mm.t
(7.09m) Cr, 0°, 12 mm.t
(7.35m) Cr, 0°, 6 mm.t
(7.62m) Be, 0°, P, S
(7.63m) J, 80°, P, S
(8.27m) Be, 0°, P, S
(9.40m) Be, 0°, P, S
(11.00m) Be, 0°, P, S
(12.29m) Be, 5°, P, R

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
2
3 / 3

CORED BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Core Size: NMLC
R.L. Surface: ~10.0 m
Date: 10/9/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: K.S./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL -0.03 VL -0.1 L -0.3 M -1 H -3 VH -10 EH	DEFECT DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
100% RETURN		-4	14		SHALE: dark grey, with light grey lamiae, bedded at 0-5°. *(continued)*	FR	H			(14.12m) Be, 0°, P, S	
		-5 -6 -7 -8 -9	15 16 17 18 19		END OF BOREHOLE AT 14.29 m					GOUNDWATER MONITORING WELL INSTALLED TO 11.5m, CLASS 18 MACHINE SLOTTED 50mm PVC STANDPIPE 8.5m TO 11.5m, CASING 0m TO 8.5m, 2mm SAND FILTER PACK 6.0m TO 1.2m, BENTONITE SEAL FROM 0.1m TO 6.0m, BACKFILLED WITH SAND TO SURFACE AND COMPLETED WITH A CONCRETED GATIC COVER	


JK Geotechnics
30809YF BH2 START CORING AT 6.77m
6
7
8
9
10
11
12
13
14
END OF BH2 AT 14.29m

Borehole No.
3
1 / 2

BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Method: SPIRAL AUGER
R.L. Surface: ~7.5 m
Date: 9/9/17
Datum: AHD
Plant Type: JK300
Logged/Checked By: A.C.K./O.F.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			ASPHALTIC CONCRETE: 50mm.t				
	N = 21 12,12,9	-	FILL: Silty gravelly clay, low plasticity, dark grey brown, light grey and orange brown, medium grained ironstone and shale gravel, trace of fine to coarse grained sand.	MC<PL			APPEARS WELL COMPACTED TOO FRIABLE FOR HP TESTING
	N > 16 4,9,7/ 100mm REFUSAL	CH	SILTY CLAY: high plasticity, light grey brown mottled orange brown, with medium to coarse grained ironstone and shale gravel.	MC<PL	(H)		RESIDUAL
		-	SHALE: grey brown, with iron indurated bands.	XW	EL - VL		VERY LOW TO LOW 'TC' BIT RESISTANCE LOW RESISTANCE
ON COMPLETION OF CORING		-	SHALE: dark grey, with VL strength bands.	DW	L - M		LOW RESISTANCE WITH MODERATE BANDS
			REFER TO CORED BOREHOLE LOG				

Column headings also printed: SAMPLES (ES, U50, DB, DS), RL (m AHD), Depth (m), Graphic Log.

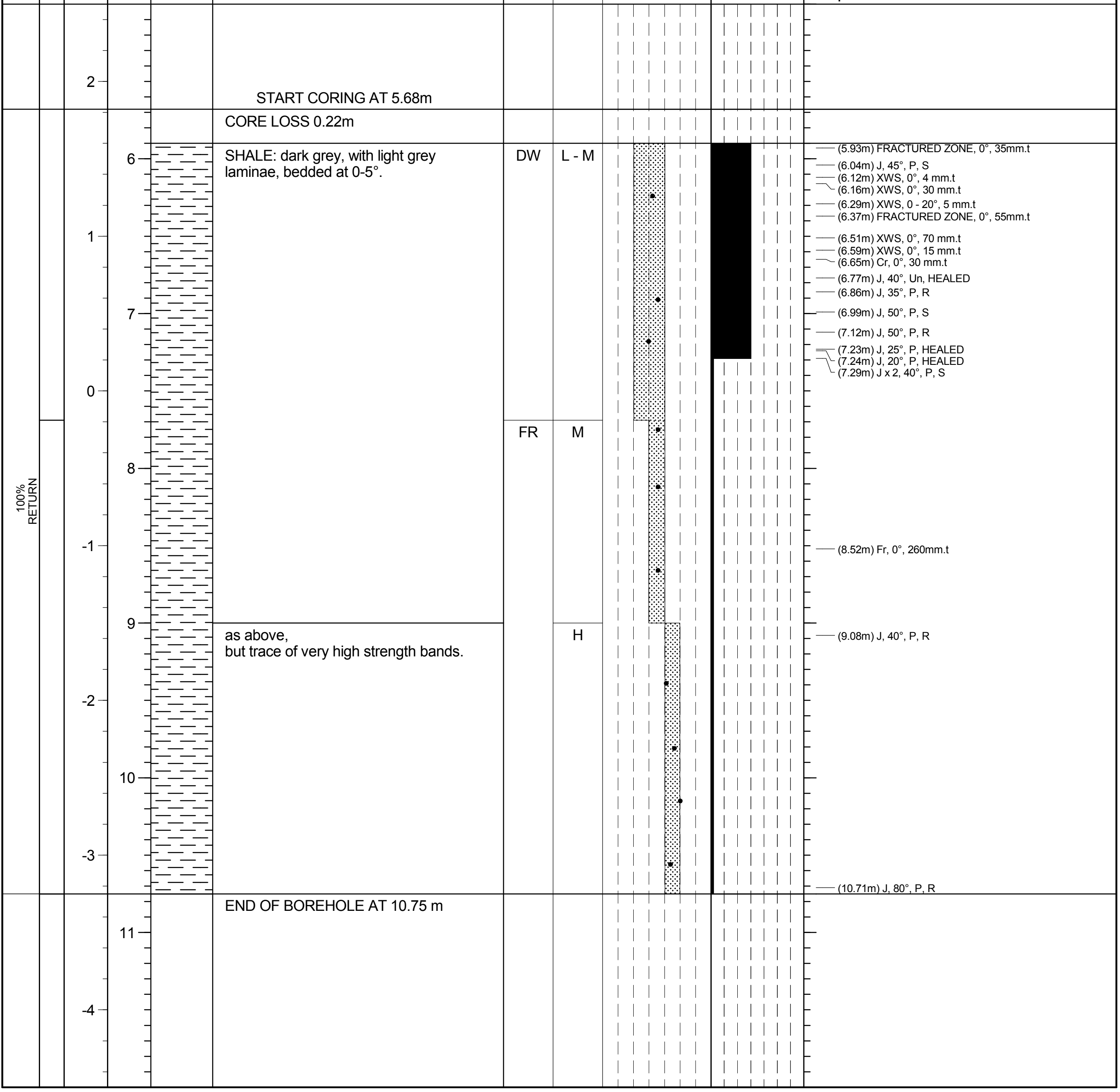
RL (m AHD): 7, 6, 5, 4, 3, 2, 1
Depth (m): 1, 2, 3, 4, 5, 6

Borehole No.
3

CORED BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Core Size: NMLC
R.L. Surface: ~7.5 m
Date: 9/9/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: A.C.K./O.F.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
		2			START CORING AT 5.68m					
					CORE LOSS 0.22m					
100% RETURN		1 0	6 7		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	DW	L - M			(5.93m) FRACTURED ZONE, 0°, 35mm.t (6.04m) J, 45°, P, S (6.12m) XWS, 0°, 4 mm.t (6.16m) XWS, 0°, 30 mm.t (6.29m) XWS, 0 - 20°, 5 mm.t (6.37m) FRACTURED ZONE, 0°, 55mm.t (6.51m) XWS, 0°, 70 mm.t (6.59m) XWS, 0°, 15 mm.t (6.65m) Cr, 0°, 30 mm.t (6.77m) J, 40°, Un, HEALED (6.86m) J, 35°, P, R (6.99m) J, 50°, P, S (7.12m) J, 50°, P, R (7.23m) J, 25°, P, HEALED (7.24m) J, 20°, P, HEALED (7.29m) J x 2, 40°, P, S
		-1	8			FR	M			(8.52m) Fr, 0°, 260mm.t
		-2 -3	9 10		as above, but trace of very high strength bands.		H			(9.08m) J, 40°, P, R (10.71m) J, 80°, P, R
		-4	11		END OF BOREHOLE AT 10.75 m					

JK_LIB_CURRENT - V8.00.GLB Log J & K CORED BOREHOLE - MASTER 30809YF SYDNEY OLYMPIC PARK.GPJ <<DrawingFile>> 19/10/2017 14:07 Produced by gINT Professional, Developed by Datgel


JK Geotechnics
30809YF BH3 START CORING AT 5.68m
5
CORE LOSS 0.22m
6
7
8
9
10
11
END OF BOREHOLE AT 10.75m

Borehole No.
4
1 / 2

BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF **Method:** SPIRAL AUGER **R.L. Surface:** ~7.8 m
Date: 9/9/17 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** A.C.K./O.F.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■			■				-	ASPHALTIC CONCRETE: 50mm.t				
									FILL: Silty sandy gravel, medium to coarse grained, igneous, grey brown, fine grained sand.	D			
	■			■	N=SPT 5/ 150mm REFUSAL	7	1		FILL: Silty clay, medium to high plasticity, brown mottled light grey, dark grey and orange brown, trace of medium grained ironstone gravel.	MC>PL		220 220	
	■			■	N > 16 8,16/ 150mm REFUSAL			CH	SILTY CLAY: high plasticity, light grey and orange brown, trace of medium grained ironstone gravel.	MC>PL	VSt - H	380 500	RESIDUAL
						6 5	2	-	SHALE: grey brown, with iron indurated bands.	XW	EL - VL		VERY LOW TO LOW 'TC' BIT RESISTANCE
				■		4	3 4		SHALE: dark grey, with iron indurated bands.	DW	L		LOW RESISTANCE LOW RESISTANCE WITH MODERATE BANDS
ON COMPLETION OF CORING				■		3	5		SHALE: dark grey.		L - M		
						2 1	6		REFER TO CORED BOREHOLE LOG				

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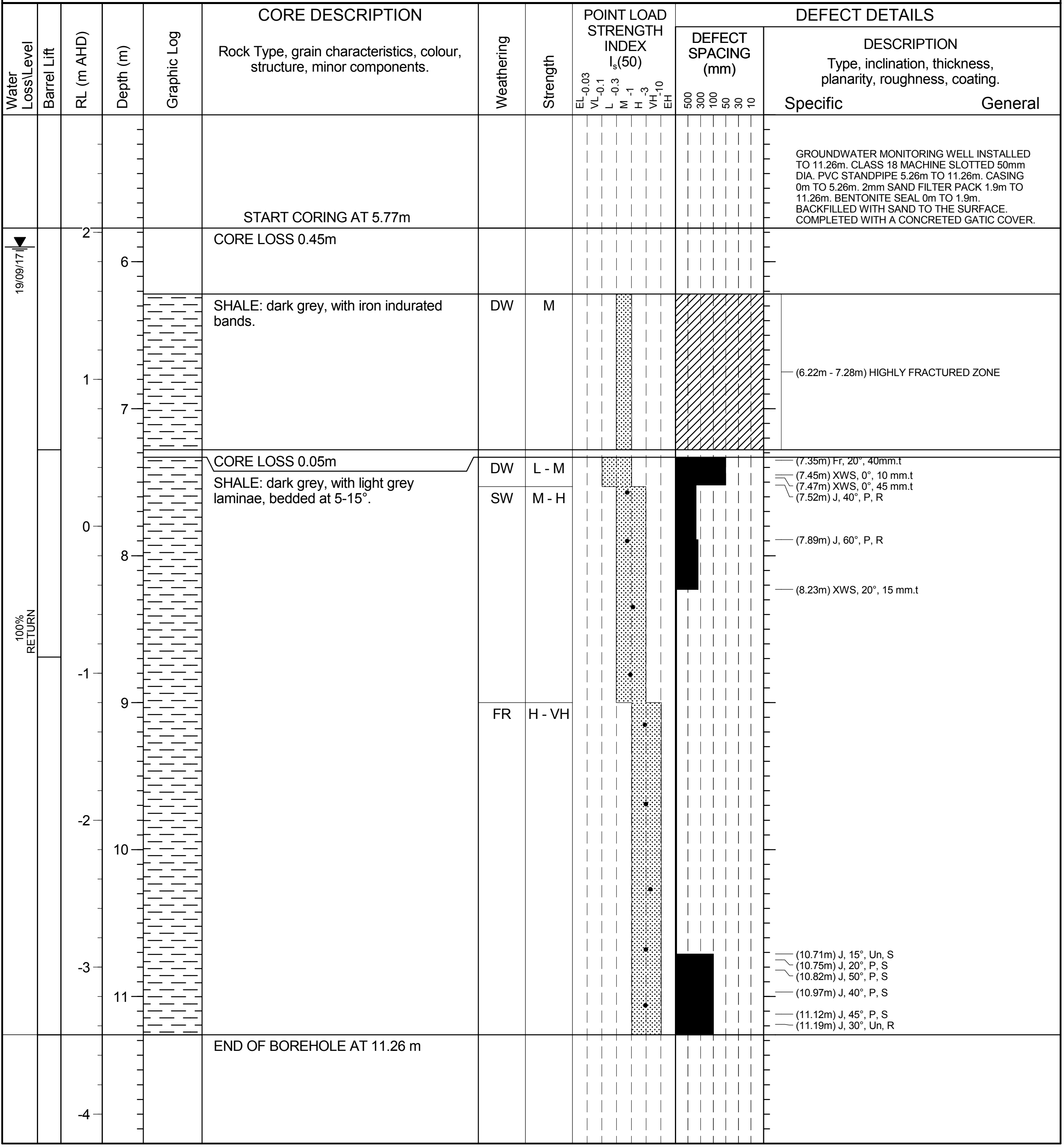
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
2 / 2

CORED BOREHOLE LOG

Client: AUSTINO PROPERTY GROUP
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 2 MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK, NSW

Job No.: 30809YF
Core Size: NMLC
R.L. Surface: ~7.8 m
Date: 9/9/17
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: A.C.K./O.F.



Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL 0.03, VL 0.1, L 0.3, M 1, H 3, VH 10, EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific / General
					START CORING AT 5.77m					GROUNDWATER MONITORING WELL INSTALLED TO 11.26m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 5.26m TO 11.26m. CASING 0m TO 5.26m. 2mm SAND FILTER PACK 1.9m TO 11.26m. BENTONITE SEAL 0m TO 1.9m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
19/09/17		2	6		CORE LOSS 0.45m					
		1	7		SHALE: dark grey, with iron indurated bands.	DW	M			(6.22m - 7.28m) HIGHLY FRACTURED ZONE
					CORE LOSS 0.05m					
100% RETURN		0	8		SHALE: dark grey, with light grey laminae, bedded at 5-15°.	DW	L - M			(7.35m) Fr, 20°, 40mm.t (7.45m) XWS, 0°, 10 mm.t (7.47m) XWS, 0°, 45 mm.t (7.52m) J, 40°, P, R
		-1				SW	M - H			(7.89m) J, 60°, P, R (8.23m) XWS, 20°, 15 mm.t
		-2 -3	9 10 11			FR	H - VH			(10.71m) J, 15°, Un, S (10.75m) J, 20°, P, S (10.82m) J, 50°, P, S (10.97m) J, 40°, P, S (11.12m) J, 45°, P, S (11.19m) J, 30°, Un, R
		-4			END OF BOREHOLE AT 11.26 m					


JK Geotechnics
30809YF BH4 START CORING AT 5.77m
5
CORE LOSS 0.45m
6
7
CL 0.05
8
9
10
11
END OF BOREHOLE AT 11.26m

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

PLOT DATE: 26/09/2017 5:52:57 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30809YF SYDNEY OLYMPIC PARK\CAD\30809YF.DWG

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

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

Title:
SITE LOCATION PLAN

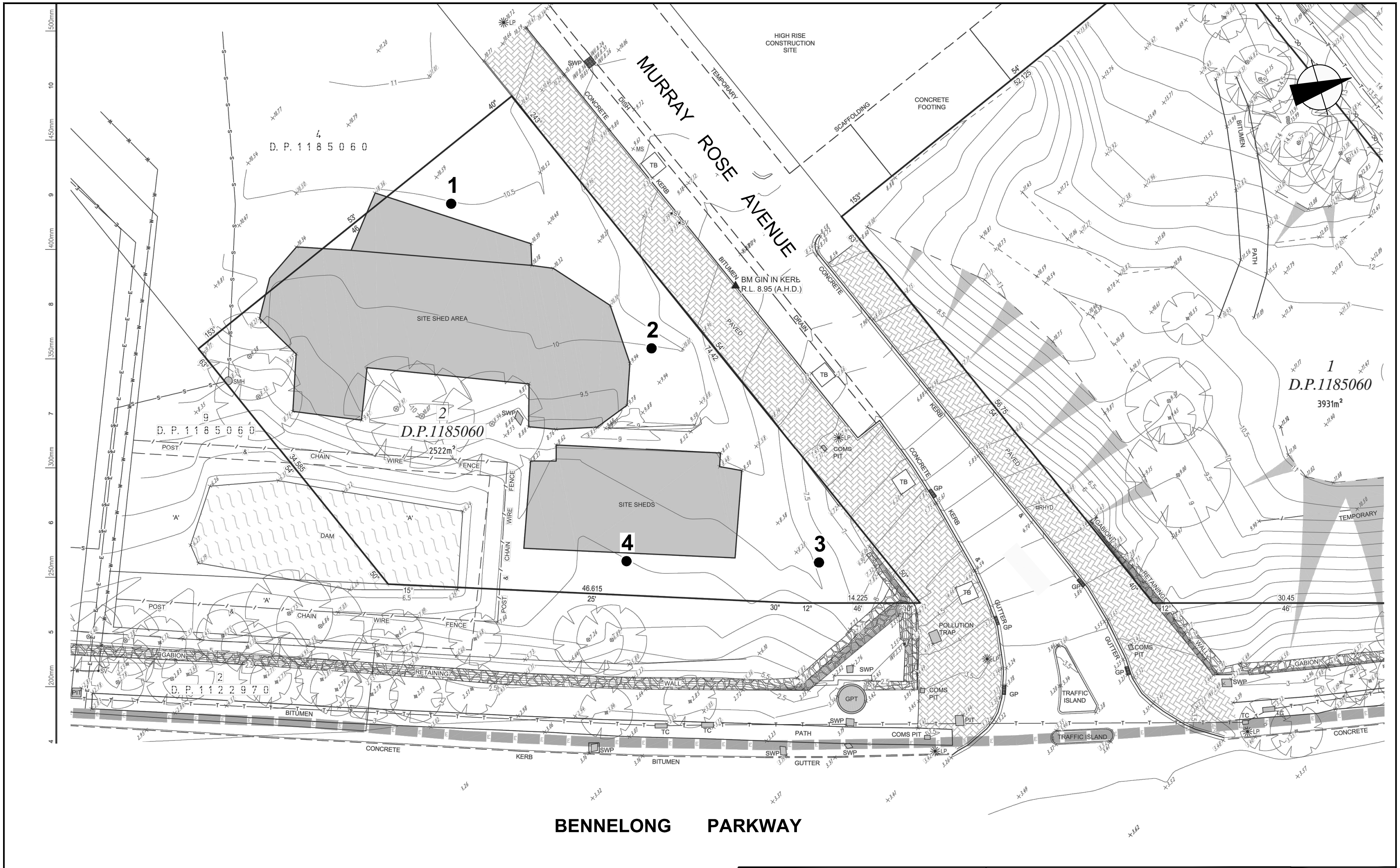
Location: 2 MURRAY ROSE AVENUE
SYDNEY OLYMPIC PARK, NSW

Report No: 30809YF

Figure No: 1

JK Geotechnics

PLOT DATE: 13/10/2017 2:41:44 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30809YF SYDNEY OLYMPIC PARK\CAD\30809YF.DWG



	0 4 8 12 16 20 SCALE 1:400 @A3 METRES
	This plan should be read in conjunction with the JK Geotechnics report.

Title: **BOREHOLE LOCATION PLAN**

Location: 2 MURRAY ROSE AVENUE
SYDNEY OLYMPIC PARK, NSW

Report No: 30809YF
Figure No: 2

J K **Geotechnics**

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.

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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:

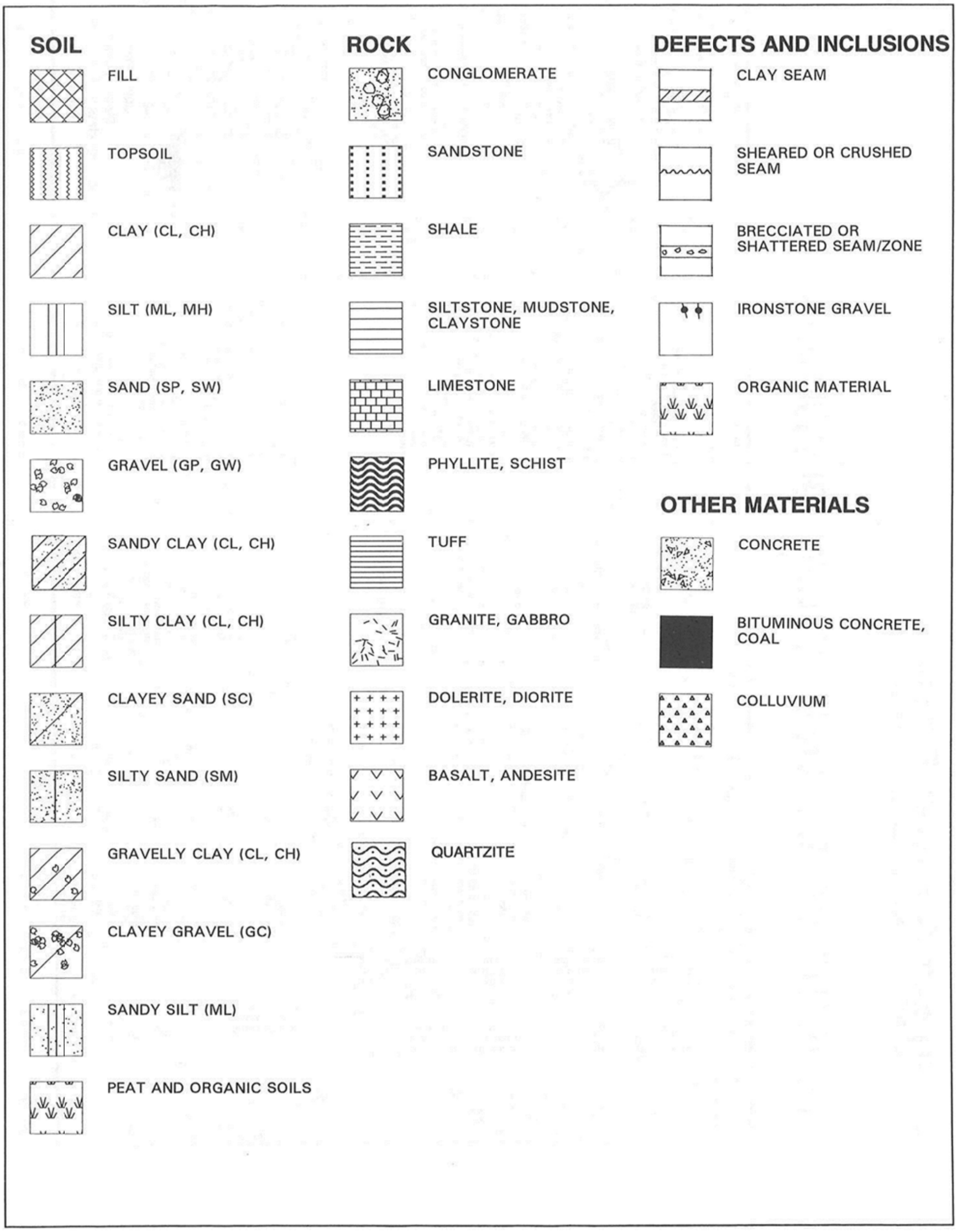
i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

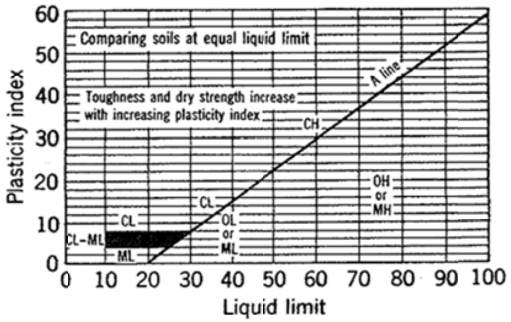
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)						Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirements for *GW*	
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures				Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures				Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines				$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirements for *SW*	
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line with *PI* greater than 7	
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size											
			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		 Plasticity chart for laboratory classification of fine grained soils		
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays					
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity					
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts					
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays					
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity					
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils					

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	