REVISED REPORT

TO
WOOLACOTTS CONSULTING ENGINEERS

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED ORAN PARK HIGH SCHOOL

AT
SOUTH CIRCUIT, ORAN PARK, NSW

2 February 2017
Ref: 29593Srptrev1 Oran Park

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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of additional geotechnical investigations for the proposed new Oran Park High School, at South Circuit, Oran Park, NSW, together with the previous investigation carried out at the site. The investigation was commissioned by Mr Kevin Christesen, Principal Engineer, of Woolacotts Consulting Engineers Pty Ltd by signed Acceptance of Proposal Form Ref: 29593Sprop dated 15 December 2016.

JK Geotechnics completed a previous geotechnical investigation at the site for the proposed development and the results were presented in our report, Ref 29593Vrpt, dated 2 September 2016. This report supersedes our previous report and presents our updated recommendations on geotechnical issues relevant to the proposed development.

A summary of the principal geotechnical issues, based on the findings of this investigation, is provided on Section 4.1.

The development was at concept stage and drawings were unavailable at the time of the investigation. However, from discussions with Mr Kevin Christesen, we understand that the development is to comprise new high school, which is to be constructed on the currently vacant site next to the existing Oran Park Primary School. It is understood that the proposed school buildings may be up to four levels in some parts of the site, with column loads of as much as 2500kN. Final details of the development, such as exact locations of the proposed facilities and structures, development levels, proposed earthworks and structural loads were unavailable or had not been supplied. This supplementary investigation has included a borehole within the adjacent primary school site where additions are planned. Comments in this report relate also to the primary school site.

The report essentially provides recommendations on geotechnical parameters and issues associated with the proposed development, based on the original 21 geotechnical test locations, the supplementary 4 test locations and laboratory test data. In summary, the report presents the findings and recommendations:

- Detailed logs of the boreholes with penetration test results and groundwater observations;
- Interpretation of Subsurface Profile;
- AS2870 Site classification;
- Suitable foundation strata and footing options;

- Allowable Bearing Pressures;
- Allowable Shaft Adhesions;
- Earthworks: excavation and filling;
- Groundwater;
- Subgrade preparation for slabs/pavements;
- Engineered Fill and compaction specification;
- Retaining walls and geotechnical design parameters;
- CBR value for pavement thickness design;
- Further Work at later project stage.

A preliminary environmental site screening of the in-situ soil and stockpiled materials has been carried out by our specialist environmental consulting division, Environmental Investigation Services (EIS), based on samples obtained from the geotechnical test boreholes. EIS has submitted separate reports with the results, Ref. E29593K, which should be read in conjunction with this geotechnical report.

2 INVESTIGATION PROCEDURE

The fieldwork for the original investigation comprised drilling eighteen test boreholes, referenced BHs 1 to 18 and excavating three test pits, referenced TPs 19 to 21, during the period 5 to 11 August 2016. The fieldwork for the additional investigation comprised drilling four test boreholes, referenced BHs 101 to 104 between 16 and 18 January 2017. The borehole locations are shown on the attached Figure 2, and these were set out, as close as practically possible to nominated locations, by taped measurements from assumed site boundaries as shown on the Survey Drawing by Rygate & Company Limited Ltd Ref 77497 dated 3 May 2016.

Prior to commencement of the fieldwork the investigation locations were electromagnetically scanned by a specialist subcontractor so that all borehole locations could be located clear of buried services. Reference to ‘Dial Before You Dig’ drawings was also completed. Safe work measures and procedures were implemented during the course of the fieldwork and in accordance to the requirements of working in and around schools.

BH1 to BH18 and BH101 to BH104 were drilled to depths between 2.8m and 7.5m using spiral augering techniques with our track mounted JK300 and JK308 drill rigs. Nine boreholes (BH1, BH2, BH4, BH7, BH10, BH101, BH102, BH103 and BH104) were subsequently extended to depths between 5.80m to 11.0m using rotary diamond coring techniques with water flush.

The strength of the natural soils was assessed from the results of Standard Penetration Tests (SPTs) completed in the boreholes, augmented by hand penetrometer tests on recovered soil samples from the SPT split spoon sampler. The strength of the augered bedrock was assessed from observation of the drilling resistance of a twin pronged Tungsten Carbide (TC) drill bit attached to the augers, tactile examination of recovered rock chips and correlation with the results of subsequent laboratory moisture content tests. In the case of the augered bedrock, strength assessments on this basis are approximate and variances of one strength order should not be unexpected. The results of the moisture content tests are presented on the attached Soil Test Services Pty Ltd (STS) Table A. The strength of the cored bedrock was assessed from examination of the recovered rock core, and the results of subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests. The results of the $I_{S(50)}$ tests are presented on the attached STS Table C.

Groundwater observations were made in the boreholes during auger drilling and at completion of the auger drilling. We note that water is introduced into the borehole during coring and therefore the water levels measured at completion of coring would be artificially high and should not be considered indicators of groundwater. Slotted PVC standpipes were installed in BH1, BH7 and BH10 to allow longer term groundwater monitoring to be completed. The water was pumped out from the standpipes on 11th of August 2016 to allow the groundwater to recharge. Groundwater levels in the standpipes were then measured on the 18th of August 2016 (7 days later). No longer-term monitoring has been carried out.

Our engineering geologist and geotechnical engineer set out the borehole locations, directed the electromagnetic scanning, nominated the sampling and testing and prepared the borehole logs. The borehole logs are attached to this report along with our Report Explanation Notes which define the logging terms and symbols used and describe the investigation techniques adopted. The reduced levels shown on the borehole logs are estimated from spot levels and contours shown on the supplied survey drawing and are therefore approximate. The datum is the Australian Height Datum (AHD).

Selected soil samples were returned to STS, a NATA registered laboratory, for moisture content Atterberg Limits, linear shrinkage, Standard Compaction, four day soaked CBR testing. The results of the testing are presented on the attached STS Tables A, B and C. Selected samples were also sent to Envirolab Services, NATA registered laboratory, to determine pH values, sulphate and chloride contents and resistivity and these results are summarised in Section 3.3, based on Envirolab Services Report No.151779.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is part of the Oran Park subdivisional works and has undergone extensive cut and fill earthworks. The site is located within gently undulating regional topography situated on slope that falls to the east at approximately 2°-3°. Cut and fill earthworks have been carried out within the site and the surrounding lots. The site itself falls from around RL99.50m in the west down to around RL92.40m in the east.

At the time of the fieldwork the site was vacant with a predominantly grassed covered surface. However, some of the higher elevation areas in the west of the site had gravelly cobbles and sand fill at surface, and residual soils were present at the surface in the north western corner of the site. The top of the embankment along the southern boundary had a gravel surface.

Several fill stockpiles or mounds (refer to section 3.2 for description of the materials and to the test pit logs) were present in the central parts of the western portion of the site where Test Pits 19, 20 and 21 were completed.

Batters are located in the central portion of the site and each slope falls at between 5° to 10° down to the east. As mentioned above an embankment is located along the southern boundary and is about 0.5m in height at the western end and 1.5m height at the eastern end.

The site is bound by Holden Drive and South Circuit to the south and west, respectively. On the eastern boundary of the site there is a gully that channels water to the north. Water was observed draining from the southern side of Holden Drive via a concrete culvert beneath road surface into the gully. To the north is Oran Park Primary School, comprising both one and three storey predominantly brick buildings, the ground surface has been modified by cut and fill earthworks creating relatively level building platforms for the structures. An oval is present to the rear of the school and has been cut into the hillside, the top of the batter is approximately 2m-3m from the fence line between the sites.

A detention basin is also present on the Primary school land abutting the north-eastern corner of the fence line.

3.2 Geology and Subsurface Conditions

The 1:100,000 Geological Map of Wollongong-Port Hacking indicates the site to be underlain by Bringelly Shale (shale, carbonaceous claystone, laminite, coal in places) of the Wianamatta Group. This subsurface profile does not take into account the soils derived from insitu weathering of the shale or earthworks (e.g. filling) that have previously been undertaken at the site.

Generally, the boreholes encountered a shallow to moderate depth cover of fill over natural silty clays, most of residual origin, which graded into shale bedrock at highly variable depths ranging from 1.4m to 6.5m below existing ground surface levels. Graphical summaries of the borehole information are presented in attached Figures 3 to 5. The more pertinent details of the encountered subsurface profile are discussed below. Reference should be made to the attached borehole logs for detailed descriptions of the subsurface conditions and the graphical borehole summary.

Stockpile Materials

The test pits (TP19 to TP21) and BH15 and BH3 (side of stockpile) were undertaken within the stockpiles in the upper western part of the site and revealed fill comprising of silty clay and sandy clay with varying amounts of topsoil and organic material, rootlets and root fibres interspersed or mixed in with the soil materials. Medium to coarse grained shale gravel and shale cobbles were also noted. Some smaller mounds of asphalt, concrete, sand and gravel were observed.

Topsoil Fill

A shallow layer of root affected (topsoil) fill was revealed from surface level to depths ranging from 0.05m and 0.3m in BH3, BH5, BH6, BH11 and BH16.

Fill

Fill was encountered beneath the topsoil fill layer and ground surface to depths of between 0.3m and 4.0m below existing surface levels. In the western part of the site (BH2, 15, 16 and 17) the fill was shallow (less than 1m) but elsewhere ranged from 1m to 4m depth. The fill mostly comprised silty clay with minor inclusions of ironstone and shale gravel and was of medium plasticity. The fill was generally assessed to be well compacted. This assessment is based on in-situ SPT tests, and our observations during drilling, which do not give a precise determination of in-situ densities since they are affected by friction during driving/pushing, the presence of gravel within the fill and the moisture content of the fill. Nonetheless, they provide a qualitative guide.

Natural Clays

The natural clay profile, of residual origin from the decomposition of the underlying shale bedrock, was encountered from surface level in BH2 and below the fill in the remaining boreholes.

The natural profile comprised silty clays, which contained iron indurated shale nodules in places and were generally assessed to be moderately to highly plastic and generally of very stiff to hard strength.

The residual clays in some boreholes were interbedded with extremely weathered shale as a gradation zone into the continuous shale bedrock profile (which is described below).

Shale Bedrock

Shale bedrock was encountered at Reduced Levels (RLs) in the range of approximately RL98.1m in BH2 (depth 1.4m) in the north-western corner dropping to about RL85.6m in BH11 (depth 6.5m) in the south eastern corner of the site.

The upper section of the bedrock profile was generally extremely to distinctly weathered being extremely low (EL) to low (L) strength.

The cored portions of BH4, BH7, BH103 and BH104 revealed the shale to improve to moderate strength at depths of 6.0m (RL90.0), 9.9m (RL84.7), 7.7m (RL87.8) and 9.0m (RL83.7) respectively. High strength shale was encountered in BH1 and BH103 at depths of 8.4m (RL91.1m) and 9.4m (RL83.3) respectively. Conversely the shale in BH101 was of extremely low strength to the termination depth of 10.6m and in BH102 there was no consistent strength increase with depth.

The recovered rock cores contained significant defects including thick core loss zones that can comprise extremely weathered rock and residual clay. The shale contained a number of inclined joints dipping between 35° and 90° and crushed seams.

Sandstone and interbedded shale and sandstone was encountered in BH101, BH102, BH103 and BH104 at depths of between 7.7m to 8.0m below existing ground surface. Within BH1, a sandstone band of extremely low to very low strength was encountered at a shallower depth of 4.5m.

Groundwater

Groundwater was not encountered during auger drilling or at completion of the auger drilling. We note that water is introduced into the borehole during coring and therefore, the water levels measured at completion of coring would be artificially high and should not be considered indicators of groundwater. Slotted PVC standpipes were installed in BH1, BH7 and BH10 to allow longer term groundwater monitoring to be completed. The water was pumped out from the standpipes on 11th of August 2016 to allow the groundwater to recharge. Groundwater levels in the standpipes were then measured on the 18th of August 2016 (7 days later):

Borehole Standpipe	Surface RL (m AHD)	Depth (m) To Groundwater Level	RL Groundwater Level (m AHD)
BH1	RL99.5	6.5	RL93.0
BH7	RL94.6	6.3	RL88.3
BH10	RL92.4	3.1	RL89.3

3.3 Laboratory Test Results

The results of the Atterberg Limits tests on a sample of the clay fill from BH7 and BH11 confirmed the clay to be of medium plasticity (refer to attached Table A). Tests carried out on samples of the residual clay produced varying plasticities: medium plasticity for the sample from BH1 and high plasticity for the sample from BH6. Based on these plasticities and correlations with the results of the linear shrinkages tests the residual silty clays and fill are considered to be moderately to highly reactive to changes in moisture content change.

The results of the four day soaked CBR tests on bulk samples of the residual clay from BHs 2 and 15 returned CBR values of 1% and 2% for the residual silty clays and 2% for the silty clay fill from BH13, when compacted to 98% of their respective Standard Maximum Dry Density (SMDD) at close to their respective Standard Optimum Moisture Content (SOMC) (refer to attached Table B). The residual clay CBR samples swelled by 2.5% and 5% during soaking, and by 3% for the clay fill, corroborating that these clays soils at the site to be moderately to highly reactive to moisture content change.

The moisture content tests on the recovered rock fragment samples correlated well with our field assessment of the in-situ bedrock strength. The results of the $I_{S(50)}$ tests also correlated well with our field assessment of the in-situ bedrock strength. The approximate Unconfined Compressive

Strength (UCS) of the bedrock was estimated from correlations with the $I_{S(50)}$ results to range from 1MPa to 34MPa, with the higher values occurring deeper in the rock profile (refer to attached Table C).

Four samples taken from the silty clay fill, natural soils and extremely weathered shale were tested for soil pH, soil sulphate, soil chloride and resistivity. A summary of the test results is presented in the table below, based on Envirolab Report Certificate 151779. The designer should take these results into consideration when design footings and other structural items in contact with the soils and rock of the site, with reference to the criteria for concrete piling exposure classification given in Table 6.4.2 (C) of AS2159-2009 “Piling Design and Installation”. Any concrete exposed to these conditions (e.g. piles) should have a characteristic concrete strength and cover as recommended in Table 6.4.3 of the standard.

Sample Description	Soil pH (pH Units)	Soil Sulphate (mg/kg)	Soil Chloride (mg/kg)	Electrical Resistivity (Ohm.cm)
BH1 (1.5-1.95) **Silty Clay Fill**	6.0	73	230	3900
BH9 (1.5m-1.95m) **Silty Clay (Residual)**	6.0	140	600	1800
BH11 (3.0m-3.45m) **Silty Clay (Residual)**	7.8	200	1800	700
BH14 (3.0m-3.45m) **XW Shale**	5.7	150	120	4600

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Findings and Issues

As discussed in more detail in Section 3.2, the boreholes penetrated high variable subsurface profile, essentially comprising apparently well compacted clay fill with gravel down to significantly variable depths of 0.3m to 4m, underlain by generally very stiff to hard residual clays of medium to high plasticity. The clays grade into a shale bedrock profile that does not consistently improve in weathering and strength with increasing depth below surface. Shale bedrock was encountered at Reduced Levels (RLs) in the range of about RL98.1m in BH2 (depth of 1.4m) in the north-western corner dropping to about RL85.6m in BH11 (depth of 6.5m) in the south-eastern corner of the site.

Groundwater levels were also recorded at highly variable depths of 3.1m to 6.5m or at about RL93m to RL88.3m.

Based on the borehole and test results, our observations and the proposed development information (refer to Section 1), we consider the following to be the principal geotechnical issues to be considered in the planning, design and construction of the development:

1. The deep fill in the higher western part of the site appears to be well compacted based on the penetration test results and our observations during drilling; some of the shallow fill cover in parts of the site is simply a root affected soil (i.e. topsoil that requires full removal). Although the fill appears well compacted, we have not seen records that document the manner of placement, compaction specification and control of the fill. Accordingly, we consider this existing material to be ‘uncontrolled’ fill, because of not seen or the absence of these records. Because of this fill, the site is considered to be Class P (‘problem’) in accordance with AS2870-2011.
2. The stockpile or mounds of fill in the central parts of the site,
3. The clay fill and underlying residual silty clays have a moderate to high potential for shrink/swell with changes in moisture content, i.e. Class M to H2 in terms of AS2870-2011. More details are provided below.
4. The most competent foundation stratum beneath the site for the proposed building footings is the shale bedrock. In view of the reactivity of the clay fill and residual clays and the relatively moderate depths to the bedrock, we recommend that the proposed buildings, some of which are four levels and heavily loaded (e.g. 2500kN), are supported on pile footings uniformly founded into the bedrock.
5. Both the existing clay fill and residual clay subgrade have low soaked CBR values. These clay subgrades are considered to be “poor” subgrades for the pavements and slabs. The use of thick pavements or lime stabilisation would be expected if the pavements are to be supported on the clay subgrade.
6. The fill present in the existing stockpile or mounds at the site where Test Pits 19 to 21 were completed should not be reused as engineered fill for any proposed development earthworks, since the clay soils are too mixed in with organic materials, roots etc..
7. All fill placed on site to replace existing or unstable areas must be engineered, controlled fill.
8. All cut and/or fill faces that might be incorporated in the proposed development must be rendered stable by appropriate batters and/or retention systems.

9. In regards to waste and contamination classification, reference should be made to the separate EIS report (refer to Section 1).

Further comments on the above issues are provided in the subsequent sections of this report.

4.2 Further Work

Although only a limited subsurface investigation was completed, we believe sufficient information has been gained to be reasonably confident as to residual clay and shale bedrock conditions. There is less confidence in respect to the deep fill areas of the site where we recommend further testing if the fill is to act as foundation support for new school structures unless the earthworks testing certificate can be obtained. Such additional testing to be considered would be penetration testing (e.g. closely spaced grid of continuous Cone Penetration Tests) to assess with more confidence the load carrying capacity and potential settlements of the existing fill mass.

In addition, it will be essential during earthworks and construction works that geotechnical inspections be commissioned to check initial assumptions about excavation, foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or ‘milestone’ inspections by a geotechnical engineer are often not adequate for earthworks and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections.

4.3 Site Classification and Foundation Design Parameters

The site classifies as Class P in terms of AS2870-2011 due to the presence of uncontrolled deep fill. The upper, western parts of the site where the fill may be stripped since it is shallow and root affected, these parts may be rated as Class HI or H2 due to the highly reactive residual clays above the rock profile. We advise that in the strict sense AS2870 site classification does not apply to buildings (e.g. four level buildings that may form part of the development) or structures larger than specified in the code, but it is a useful guide in highlighting the potential foundation problems of this site.

The most competent foundation stratum at the site is the bedrock. Given the presence of deep, potentially uncontrolled fill in some areas, the underlying highly reactive clays and considering the moderate depths to rock, we prefer and recommend that buildings be uniformly supported on the shale bedrock to avoid potential for uneven foundation movements, especially reactive clay movements. The bedrock is the recommended foundation stratum for structure with movement sensitive finishes and/or for structures that have columns or walls with moderate to heavy loads. Notwithstanding, if required after further development details are known, we can provide the option of using spread footings on the clay provided the structure is designed to withstand potential reactive clay movements. The shrink-swell reactivity is dependent on the nature of the depth of the fill and whether any excavation has occurred in the last two years due to the effects on the presence of a cracked zone in the clays. Thus each building must be considered on the basis of the proposed earthworks as well as any recent earthworks.

An indicative engineering classification of the shale bedrock encountered in the rock cored boreholes has been carried out (in accordance with Pells et al. 1998) and is tabulated below:

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Borehole	**Depth to top of (m) / Approximate RL (m AHD) of Rock Class**			
	Class V	**Class IV**	**Class III**	**Class II**
1	4.5 / 95.0	6.4 / 93.1	8.4 / 91.1	8.4 / 91.1
2	1.4 / 98.1	3.6 / 95.9	5.5 / 94.0	N/A
4	6.0 / 90.0	6.6 / 89.4	6.6 / 89.4	8.3 / 87.7
7	5.6 / 89.0	8.6 / 86.0	9.5 / 85.1	9.8 / 84.8
10	5.5 / 86.9	6.0 / 86.4	7.2 / 85.2	N/A
101	4.5 / 91.0	N/A	N/A	N/A
102	4.0 / 91.0 9.2 / 85.8	8.2 / 86.8	6.4 / 88.6 8.5 / 86.5	N/A
103	N/A	4.0 / 91.5	6.8 / 88.7	8.0 / 87.5
104	4.8 / 87.9 7.0 / 84.7	5.5 / 87.2	9.0 / 83.7	N/A

The following table provides our recommended geotechnical parameters for design of footings bearing on the aforementioned different classes of shale bedrock. Below the table we provide further comments relating to footing design.

Shale Rock Class	**Allowable Bearing Pressure (kPa)**	**Allowable Shaft Adhesion (kPa)**
Class V	700	70

Class IV	1000	100
Class III	3000	300
Class II	5000	500

IMPORTANT NOTES

1. The allowable shaft adhesion value shown in the above table is for compressive loads. Under uplift loads the value should be halved. Pile sockets must be appropriately roughened. The minimum embedment of footings into the rock class should be around 0.3m.
2. Class II shale was not encountered in all test boreholes and will be at considerable depth in some areas. Another consideration is that designing and founding footings on Class II shale will entail inspections with spoon testing (if spread footings) or rock coring (if pile footings) in at least 50% of footing locations, taken down 1.5 times the footing width/diameter or 2.5m, whichever is the lesser, to confirm that no adverse defects are present below the founding levels. During construction inspections the geotechnical engineer should nominate the testing. The presence of significant defects would require a reduction in allowable bearing capacity or an increase in footing depth.
3. On the other hand using the more consistent Class III or IV rock as the founding stratum will only require visual inspection of the footing bases.

Pile footings would be required to reach the Class III and Class II shale, as well some of the Class IV at some locations. Assuming that Class III foundation is adopted for the reasons set out in NOTES (2) and (3) above, then most piles would be taken down below the groundwater levels measured in the three standpipes (refer to groundwater table in Section 3.2). Although, it is likely that conventional bored piers would be a suitable pile type, due to potential groundwater ingress issues we suggest that trial piers be completed under geotechnical inspection to assess their suitability. In the worst case CFA (grout injected augered) piles could be adopted.

As a minimum requirement, the initial stages of footing excavation should be inspected by a geotechnical engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit. As there is significant variability in the level/depth to the different classes of rock more frequent or full-time inspections should be considered.

4.4 Floor Slabs and Pavements

The main geotechnical issue with earthworks, including subgrade preparation under ground floor slabs and external pavement areas, is to do with the areas of deep existing fill that appear to be well compacted but must be considered uncontrolled due to the absence of compaction control records. Another issue is to do with the reactivity of the clay fill (as well as the underlying residual clay). We suggest that generous time and budget allowances be provided for subgrade improvement works.

Based on the SPT values and our observations from the boreholes, we estimate that the long term Elastic Modulus (E) of the fill in its assessed compacted state might be in the range of 20MPa to 40MPa. These values of E may be used for design of the slab but we also recommend that the following be taken into consideration.

In the area of the slabs, we recommend that the compaction of the existing clay fill be further checked that it meets minimum engineered fill standards as detailed below. Notwithstanding, the floor slabs must be designed and constructed to withstand potential shrink/swell movements of the existing clay fill subgrade. The clay fill is considered to be at moderately to highly reactive and may be subject to free surface movements equivalent to those quoted in AS2870-2011 for Class M to H2 clay subgrades. In areas not covered by the existing clay fill or where it is removed or of limited thickness then the slab design must anyway take into consideration the similar potential reactivity of Class M to H2 for the residual clay subgrade.

Alternatively, the slab can be fully suspended on the footings bearing on the bedrock. With the latter option, if the slab is to be fully suspended on bored pier footings then it would be unnecessary to complete any particular subgrade preparation other than stripping any top layer of root affected soil. A void former should be used below suspended slabs to prevent uplift pressures resulting from clay heave.

As a minimum, if the slab is to be supported on existing fill subgrade and if some differential movements can be tolerated, the subgrade for the ground floor slab and for external pavements should undergo the following basic treatment. Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1. Strip the upper layer of fill that contains deleterious materials or organics (roots), and stockpile this separately since these materials are not suitable for re-use as engineered fill. This upper layer is irregular in depth and occurrence but for budgeting purposes allow

for an average stripping depth of about 0.2m but deeper as deemed necessary after inspection by a geotechnician or geotechnical engineer.

2. The exposed subgrade at the base of the excavation should be proof rolled with at least 8 passes of a heavy (not less than 12 tonne) smooth drum vibratory roller. The purpose of the proof rolling is to detect any soft or heaving areas.
3. The final pass should be undertaken in the presence of a geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in existing fill compaction.
4. Unstable subgrade detected during proof rolling should be locally excavated down to a stiff or sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill.
5. The subgrade should be moisture conditioned to be within 2% of SOMC. Very dry subgrades may pass proof rolling but remain poorly compacted and/or likely to undergo heave when moisture equilibration occurs. Heave of dry, over-compacted clay may be the bigger potential problem on this site judging by the high SPT blow counts that were obtained in the fill.
6. If any shrinkage cracks are present in the subgrade the area should be tyned, watered and rolled until the cracks no longer appear.

Particular attention must be paid to the provision of adequate site drainage. The main objective of the drainage should be to minimise the risk of water entering the subgrade and to prevent water from ponding next to the slabs. To minimise the possibility of moisture ingress into the clay subgrade, we recommend that surface and subsurface drains be constructed at an early stage in the program to intercept water from the higher areas of the site. The surface drain should discharge well away from the building area, say, to adjoining stormwater drainage systems. Furthermore, we recommend that close attention be paid to the design and construction of the water and sewage services beneath the proposed buildings, to minimise the risk of breakage, leakage and subsequent moisture ingress into the fill subgrade. These services should be designed with flexible joints and they should be repaired promptly when ruptured.

4.5 Engineered Fill Specifications

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in layers not greater than 200mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density (SMDD).

The existing clay fill (with exception of that from the current stockpiles) at the site are acceptable for re-use on condition that the soils used are clean (i.e. free of organics and inclusions greater than 75mm size), free of contaminants. The existing clay fill soils should be compacted in maximum 200mm loose layers to a density strictly between 98% and 102% of SMDD and at moisture content within 2% of Standard Optimum. All clay fill should preferably be used in the lower fill layers. Thus, the use of clay materials for engineered fill will entail more rigorous earthwork supervision and compaction control, time for drying out the soils and hence, possibly a greater eventual cost for earthworks.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. We recommend that at least Level 2 control of fill compaction, as defined in AS3798-2007, be adhered to on this site. However, if a floor slab is to rely on engineered fill for support then we recommend a higher compaction control of Level 1. We can complete the abovementioned testing and supervision if required. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

During construction of the fill platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program.

4.6 Excavation/Fill Batter Slopes and Retention

We do not have details if retaining walls or batter slopes are proposed as part of the development. Notwithstanding, we provide some recommendations below on these matters, assuming that the change in levels at the site might require the use of batters or retaining walls.

Fill edges or excavation sides should be supported by forming these at stable batter slopes and/or provision of suitable retention systems. For well compacted or engineered fill and residual clay slopes, a short term batter no steeper than 1 Vertical (V) to 1 Horizontal (H) may be adopted but this batter must be flattened to 1V in 2H for long term stability. The temporary batter slope would be applicable for short term situations during construction of various building structures (e.g. retaining walls). If the batter slope is to be left exposed for a period of say more than one month, or is affected by heavy rainfall, we recommend that the batter slope be flattened to 1V to 2H. If permanent batter slopes of soil cuts or of fill platforms are proposed, we recommend that they be graded at no steeper than 1 Vertical (V) on 2 Horizontal (H). Surface erosion protection, for example, quick establishing grass or proprietary systems (such as those provided by Geofabrics Australasia Pty Ltd), must be provided to the permanent batter slopes. Dish drains should be provided along the crest and toe of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system.

Any free standing retaining walls that are required at the site should be designed for a coefficient of active earth pressure of at least 0.35 and a bulk unit weight of 20kN/m3, assuming a horizontal backfill surface. The retaining wall should be designed to withstand full hydrostatic pressure unless measures are taken to introduce complete and permanent drainage of the ground behind the walls. All surcharge loads, including footing loads, which affect the walls should be considered in their design.

4.7 External Pavements

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as bus turning and maneuvering. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice

After stripping to nominal subgrade or formation level, the proposed pavement area should be prepared as outlined in the above Sections 4.4 and 4.5, with respect to proof rolling, compaction and treatment of shrinkage cracks.

We recommend that thickness design of pavements be based on a soaked CBR value of not greater than 1.5%, which is slightly lower than the average value obtained from the three laboratory test results. Both the existing clay fill and the residual clay subgrades have produced quite low soaked CBR values of 1% to 2% (refer to attached Table B).

Hence, with these very low CBR and relatively poor subgrade conditions the main options are:

1. Design the pavements for a CBR value of 1.5% or an estimated subgrade reaction modulus (for concrete slabs or pavements) of 20kPa/mm (750mm diameter plate).

OR

2. Provide an appropriate select fill layer as part of the overall pavement thickness. The select fill may be well graded ripped or crushed sandstone or good quality shale with a minimum soaked CBR value of 10%. The pavement sections where imported fill is used to raise site levels may be designed taking into account the thickness and soaked CBR value of the imported fill material.

OR

3. Stabilise the subgrade to a depth of 200mm to 300mm by the addition of lime. When thoroughly mixed and recompacted to a minimum of 98% of SMDD, a reduction in reactivity along with substantial increase in strength will be achieved. As a guide, the addition of approximately 4% lime by dry weight of clay should result in a soaked CBR value of around 6% or an equivalent subgrade reaction modulus of 40kPa/mm. This should, however, be confirmed by laboratory testing. If lime stabilisation is undertaken, an experienced contractor with appropriate equipment should complete it. We note that use of quicklime close to residential areas is generally not preferred unless an acceptable method of dust suppression can be adopted.

Concrete pavements should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RTA Specification 3051 unbound base (e.g. DGB20) or similar quality, and compacted to a minimum density of 98% of MMDD. The sub-base material will provide more uniform slab support and would reduce “pumping” of subgrade “fines” at joints. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

Where the pavement edge is unconstrained by landscaping strips, we recommend that an edge thickening be provided for protection against erosion and the effects of possible future topsoil stripping.

Density tests should be regularly carried out on the sub-base layer to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 1000m^2, or three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007.

Subsoil drains should be provided along the upslope edges of all proposed external pavement areas (where there are significant cuts) with invert levels at least 200mm below subgrade level.

The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

5 GENERAL COMMENTS

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

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.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

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SOIL TEST SERVICES
ABN 43 002 145 173

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

Client: JK Geotechnics
Project: Proposed Oran Park High School
Location: South Circuit, Oran Park, NSW

Ref No: 29593V
Report: A
Report Date: 24/08/2016
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	3.00-3.45	14.9	40	16	24	10.5
1	4.50-5.00	11.4				
3	2.50-3.00	9.3				
4	6.00-6.45	5.4				
5	4.00-4.20	11.5				
6	3.00-3.45	22.4	67	21	46	16.0
6	4.50-5.00	9.8				
6	5.50-6.00	9.0				
7	1.50-1.95	17.3	46	18	28	13.5
7	7.00-7.50	10.4				
8	4.00-4.50	8.2				
9	4.00-4.50	12.9				
11	0.50-0.95	13.5	46	18	28	14.0
11	6.50-7.00	8.6				
13	5.50-6.00	14.3				
14	4.00-4.50	18.0				
15	2.00-2.50	13.5				
16	1.30-1.50	10.5				
16	2.00-2.50	5.8				
17	4.00-4.50	8.6				
18	3.00-3.50	17.0				
18	4.00-4.50	15.7				

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 12/08/2016

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Oran Park High School
Location: South Circuit, Oran Park, NSW

Ref No: 29593V
Report: B
Report Date: 24/08/2016
Page 1 of 1

BOREHOLE NUMBER	2	13	15
DEPTH (m)	0.20 - 0.50	0.50 - 1.00	0.50 - 1.00
Surcharge (kg)	4.5	4.5	4.5
Maximum Dry Density (t/m^3)	1.71 STD	1.80 STD	1.60 STD
Optimum Moisture Content (%)	17.4	14.9	19.4
Moulded Dry Density (t/m^3)	1.68	1.76	1.56
Sample Density Ratio (%)	98	98	98
Sample Moisture Ratio (%)	101	101	101
Moisture Contents			
Insitu (%)	26.4	14.5	22.5
Moulded (%)	17.6	15.0	19.7
After soaking and			
After Test, Top 30mm(%)	29.1	27.8	34.7
Remaining Depth (%)	19.7	17.2	24.4
Material Retained on 19mm Sieve (%)	0	0	0
Swell (%)	2.5	3.0	5.0
C.B.R. value: @2.5mm penetration	2.0	2.0	1.0

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample: 12/08/2016

Authorised Signature / Date
(D. Treweek)

24/8/16

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	29593S
Project:	Proposed Oran Park High School	**Report:**	C
Location:	South Circuit, Oran Park, NSW	**Report Date:**	30/01/2017
		Page 1 of 2	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
1	6.46-6.51	0.2	4
	6.87-6.92	0.2	4
	7.34-7.39	0.2	4
	8.55-8.59	1.7	34
	9.28-9.31	1.4	28
2	3.00-3.04	0.04	1
	3.90-3.94	0.3	6
	4.10-4.13	0.3	6
	4.86-4.89	0.2	4
	5.53-5.56	0.4	8
4	6.60-6.64	0.7	14
	6.96-7.00	0.7	14
	7.18-7.23	0.7	14
	7.95-8.00	0.6	12
	8.39-8.42	0.6	12
	8.96-9.00	0.5	10
	9.23-9.26	0.7	14
7	8.90-8.94	0.2	4
	9.57-9.61	0.3	6
	9.95-10.00	0.4	8
	10.24-10.28	0.5	10
	10.88-10.92	0.4	8
10	6.10-6.15	0.05	1
	7.34-7.39	0.2	4
	7.95-8.00	0.4	8
	8.40-8.44	0.2	4
	8.83-8.88	0.1	2

NOTES: See Page 2 of 2

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Oran Park High School
Location: South Circuit, Oran Park, NSW

Ref No: 29593S
Report: C
Report Date: 30/01/2017
Page 2 of 2

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	6.05-6.09	0.5	10
102	6.53-6.56	1.2	24
	7.17-7.20	1.0	20
	7.97-8.01	1.2	24
	8.09-8.13	3.6	72
	8.56-8.59	1.9	38
	9.06-9.10	2.2	44
103	6.83-6.87	0.9	18
	7.30-7.33	0.5	10
	7.75-7.78	0.9	18
	8.12-8.16	0.6	12
	8.94-8.97	0.9	18
	9.26-9.29	1.1	22
	9.62-9.65	4.2	84
104	6.85-6.88	0.1	2
	9.08-9.11	0.6	12

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 $I_{S\ (50)}$

Borehole No.
1
1 / 2

BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V **Method:** SPIRAL AUGER **R.L. Surface:** ~99.5 m
Date: 8/8/16 **Datum:** AHD
Plant Type: JK300 **Logged/Checked By:** T.C./F.V.

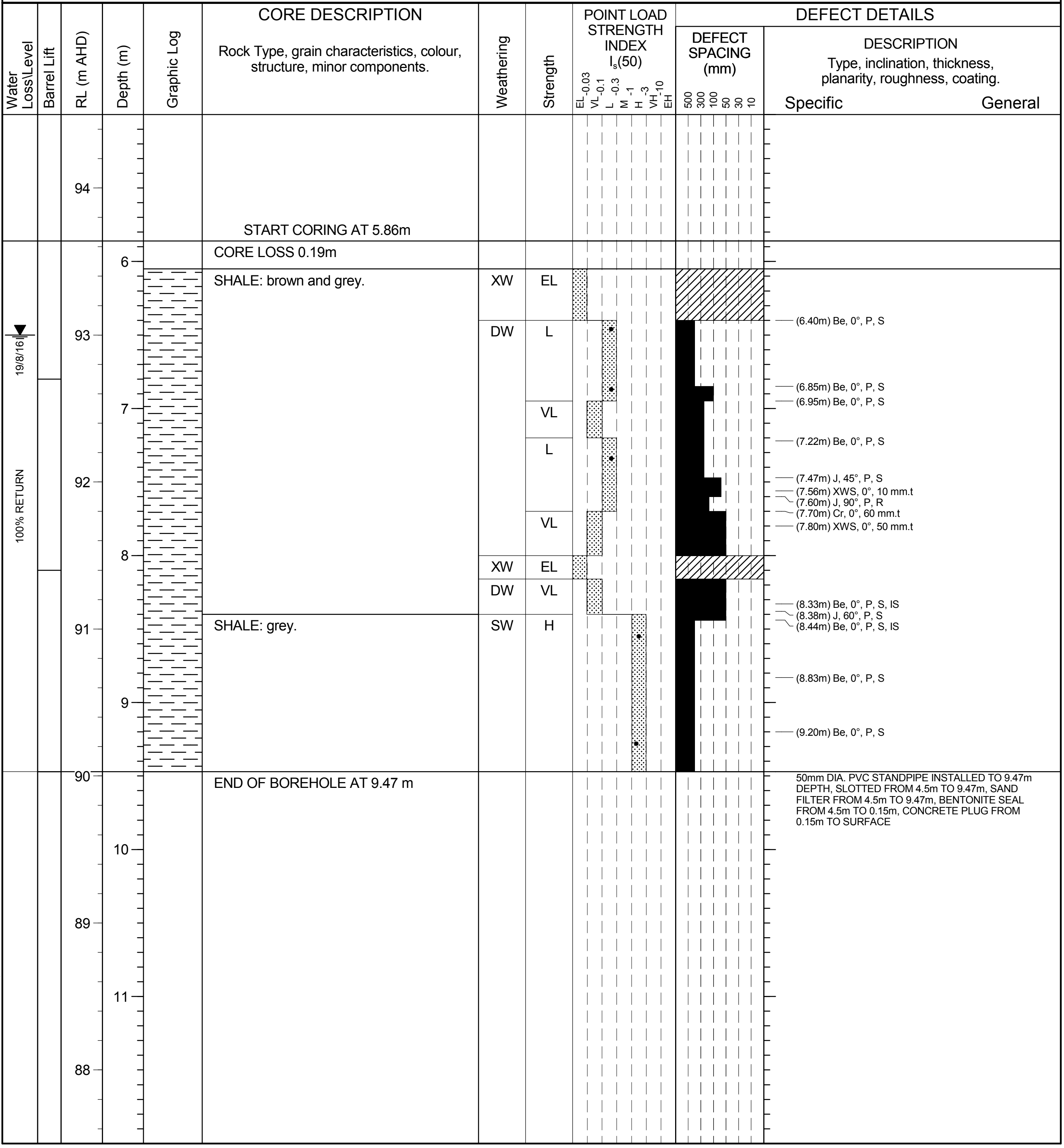
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■									FILL: Silty clay, low plasticity, brown and grey, with fine to medium grained shale gravel.	MC<PL			SOIL COVER AT SURFACE APPEARS WELL COMPACTED
				■	N = 36 8,16,20	99							>600 >600	
							1							
				■	N = 23 10,13,10	98							>600 >600	
						97	2		CH	SILTY CLAY: medium plasticity, red brown.	MC<PL	VSt		RESIDUAL SOIL
	■			■	N = 15 11,7,8	96	3						350 350	
							4							
				■		95 94	5		-	SHALE: grey and orange brown.	XW	EL		VERY LOW 'TC' BIT RESISTANCE WITH HIGH BANDS
						93	6			REFER TO CORED BOREHOLE LOG				

Borehole No.
1
2 / 2

CORED BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Core Size: NMLC
R.L. Surface: ~99.5 m
Date: 8/8/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: T.C./F.V.



CORE DESCRIPTION	Weathering	Strength
START CORING AT 5.86m		
CORE LOSS 0.19m		
SHALE: brown and grey.	XW	EL
	DW	L
		VL
		L
		VL
	XW	EL
	DW	VL
SHALE: grey.	SW	H
END OF BOREHOLE AT 9.47 m		

(6.40m) Be, 0°, P, S
(6.85m) Be, 0°, P, S
(6.95m) Be, 0°, P, S
(7.22m) Be, 0°, P, S
(7.47m) J, 45°, P, S
(7.56m) XWS, 0°, 10 mm.t
(7.60m) J, 90°, P, R
(7.70m) Cr, 0°, 60 mm.t
(7.80m) XWS, 0°, 50 mm.t
(8.33m) Be, 0°, P, S, IS
(8.38m) J, 60°, P, S
(8.44m) Be, 0°, P, S, IS
(8.83m) Be, 0°, P, S
(9.20m) Be, 0°, P, S

50mm DIA. PVC STANDPIPE INSTALLED TO 9.47m DEPTH, SLOTTED FROM 4.5m TO 9.47m, SAND FILTER FROM 4.5m TO 9.47m, BENTONITE SEAL FROM 4.5m TO 0.15m, CONCRETE PLUG FROM 0.15m TO SURFACE


JK Geotechnics
#29593V BH1 START CORING AT 5.86m
5
CL: 0.190m
6
7
8
9
END OF BOREHOLE AT 9.47m

Borehole No.
2
1 / 2

BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Method: SPIRAL AUGER
R.L. Surface: ~99.5 m
Date: 8/8/16
Datum: AHD
Plant Type: JK300
Logged/Checked By: T.C./F.V.

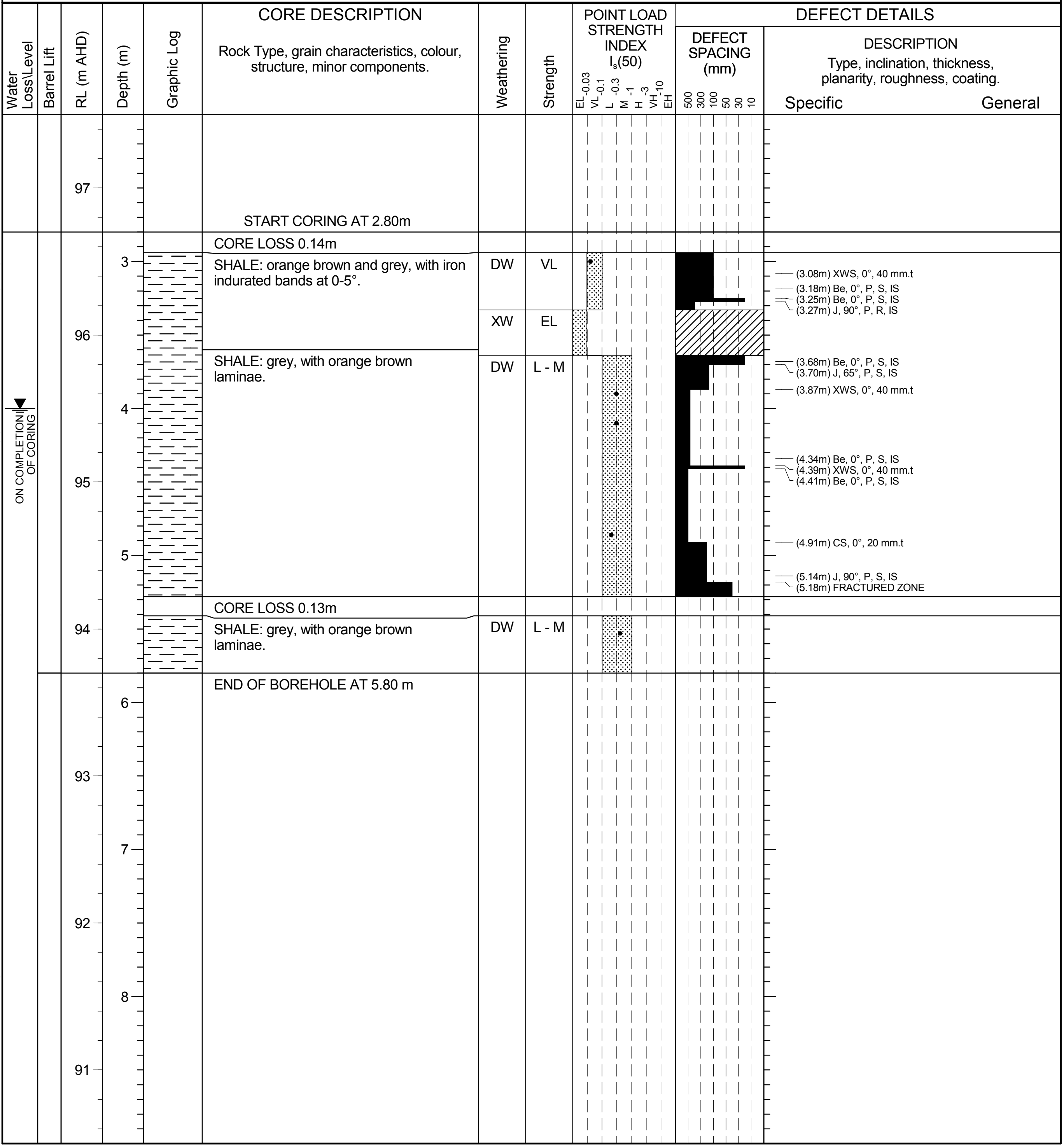
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■		■					▨	CH	SILTY CLAY: high plasticity, red brown and grey.	MC~PL	(F - St)		RESIDUAL SOIL AT SURFACE
				■	N > 22 8,12,10/ 75mm REFUSAL	99	1	▨				H	>600 >600	
				■		98 97	2	☰	-	SHALE: orange brown and grey, with iron indurated shale bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE WITH HIGH BANDS
						96 95 94 93	3 4 5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
2
2 / 2

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Core Size: NMLC
R.L. Surface: ~99.5 m
Date: 8/8/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: T.C./F.V.



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
		START CORING AT 2.80m				
		CORE LOSS 0.14m				
ON COMPLETION OF CORING		SHALE: orange brown and grey, with iron indurated bands at 0-5°.	DW	VL	(3.08m) XWS, 0°, 40 mm.t (3.18m) Be, 0°, P, S, IS (3.25m) Be, 0°, P, S, IS (3.27m) J, 90°, P, R, IS	
			XW	EL		
		SHALE: grey, with orange brown laminae.	DW	L - M	(3.68m) Be, 0°, P, S, IS (3.70m) J, 65°, P, S, IS (3.87m) XWS, 0°, 40 mm.t (4.34m) Be, 0°, P, S, IS (4.39m) XWS, 0°, 40 mm.t (4.41m) Be, 0°, P, S, IS (4.91m) CS, 0°, 20 mm.t (5.14m) J, 90°, P, S, IS (5.18m) FRACTURED ZONE	
		CORE LOSS 0.13m				
		SHALE: grey, with orange brown laminae.	DW	L - M		
		END OF BOREHOLE AT 5.80 m				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#29593V BH2 START CORING AT 2.80m
2
CORE LOSS: 0.140m
3
4
5
CL: 0.130m
END OF BOREHOLE AT 5.8m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
3
1 / 1

BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Method: SPIRAL AUGER
R.L. Surface: ~97.7 m
Date: 11/8/16
Datum: AHD
Plant Type: JK300
Logged/Checked By: T.C./F.V.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION	■							▩		FILL: Silty clay topsoil, low plasticity, dark brown, with root fibres.				GRASS COVER APPEARS WELL COMPACTED
				■	N = 15 4,7,8	97		▩		FILL: Silty clay, medium to high plasticity, red brown, grey and orange brown, with fine to medium grained shale gravel.	MC~PL	(H)		
	■			■	N = 21 5,8,13	96	1	▨	CH	SILTY CLAY: high plasticity, red brown.	MC~PL	VSt - H	350 400	RESIDUAL SOIL
				■		95	2	▤	-	SHALE: brown and grey.	DW	VL		VERY LOW 'TC' BIT RESISTANCE WITH HIGH BANDS
						94 93 92 91	3 4 5 6			END OF BOREHOLE AT 3.00 m				

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
4
1 / 2

BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Method: SPIRAL AUGER
R.L. Surface: ~96 m
Date: 10/8/16
Datum: AHD
Plant Type: JK300
Logged/Checked By: T.C./F.V.

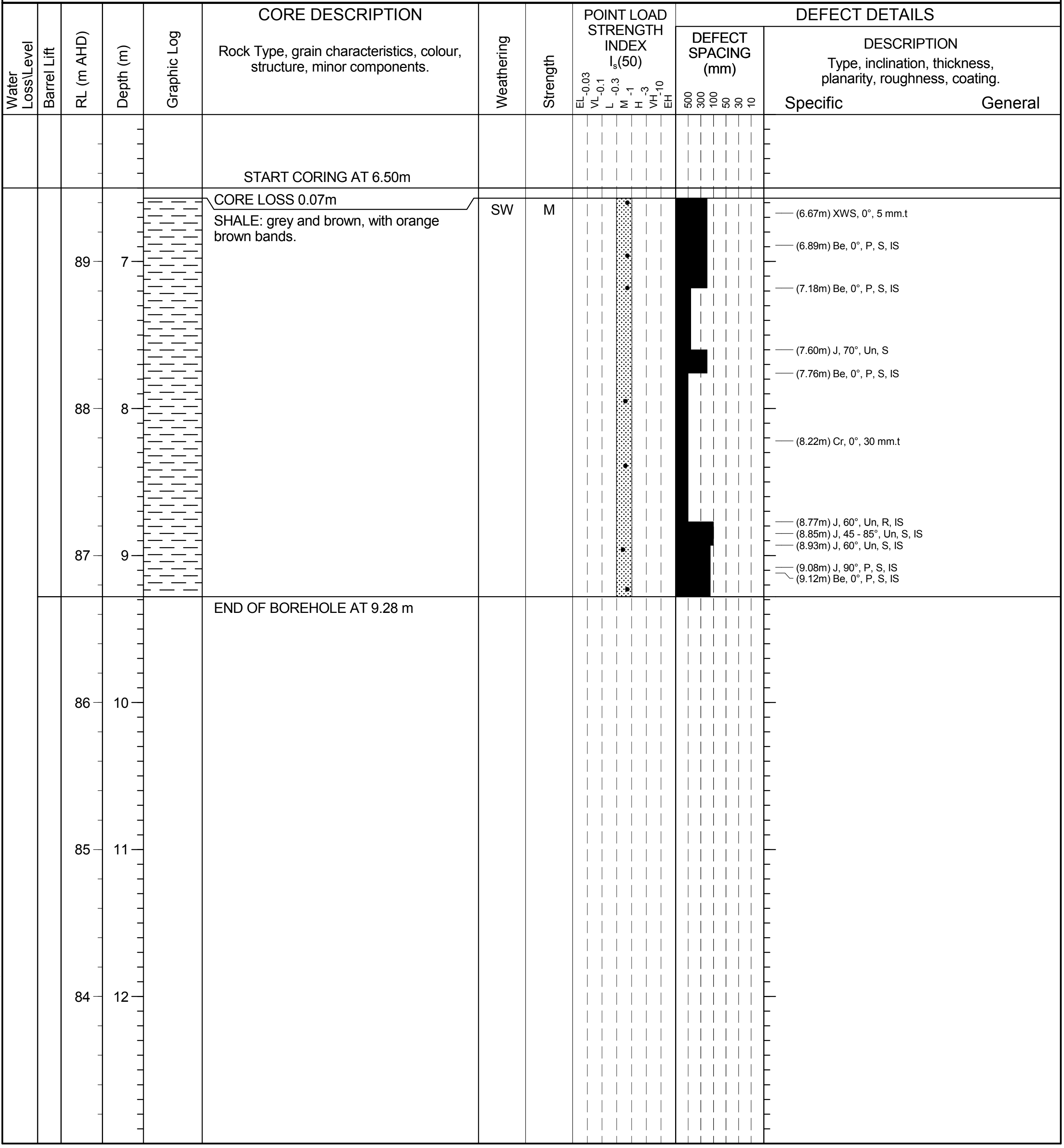
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	■									FILL: Sandy gravel, fine to coarse grained igneous and shale, grey and brown.	M			GRASS COVER
	■									FILL: Silty clay, low plasticity, brown and grey, with fine to coarse grained shale gravel and fine grained sand.	MC<PL	(H)		APPEARS WELL COMPACTED
				■	N = 28 8,13,15								600 500 550	
						95	1							
				■	N = 28 17,14,14								500 450 >600	
						94	2							
	■													
				■	N = 26 4,11,15	93	3		CL	SILTY CLAY: medium plasticity, red brown, with carbonaceous material and iron indurated shale nodules.	MC<PL	H	>600 550	RESIDUAL SOIL
						92	4							
									CL-CH	SILTY CLAY: medium to high plasticity, grey and orange brown and red brown, with iron indurated shale nodules.	MC~PL	VSt - H		
				■	N = 20 10,8,12								250 500	
						91	5							
					N=SPT 15/ 10mm REFUSAL	90	6		-	SHALE: grey and orange brown, with iron indurated bands.	DW	VL		VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
										REFER TO CORED BOREHOLE LOG				

Borehole No.
4
2 / 2

CORED BOREHOLE LOG

Client: WOOLACOTTS CONSULTING ENGINEERS
Project: PROPOSED ORAN PARK HIGH SCHOOL
Location: SOUTH CIRCUIT, ORAN PARK, NSW

Job No.: 29593V
Core Size: NMLC
R.L. Surface: ~96 m
Date: 10/8/16
Inclination: VERTICAL
Datum: AHD
Plant Type: JK300
Bearing: N/A
Logged/Checked By: T.C./F.V.



START CORING AT 6.50m

CORE LOSS 0.07m

SHALE: grey and brown, with orange brown bands. SW M

(6.67m) XWS, 0°, 5 mm.t
(6.89m) Be, 0°, P, S, IS
(7.18m) Be, 0°, P, S, IS
(7.60m) J, 70°, Un, S
(7.76m) Be, 0°, P, S, IS
(8.22m) Cr, 0°, 30 mm.t
(8.77m) J, 60°, Un, R, IS
(8.85m) J, 45 - 85°, Un, S, IS
(8.93m) J, 60°, Un, S, IS
(9.08m) J, 90°, P, S, IS
(9.12m) Be, 0°, P, S, IS

END OF BOREHOLE AT 9.28 m