Appendix A

Douglas Partners Geotechnical Report

Douglas Partners
Geotechnics • Environment • Groundwater

Integrated Practical Solutions

UPDATED REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED ROOKWOOD ROAD SUBSTATION
Lot 101 ROOKWOOD ROAD
POTTS HILL

Prepared for
TRANSGRID

Project 71486
April 2010

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Project 71486
28 April 2010

UPDATED REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED ROOKWOOD ROAD SUBSTATION
LOT 101 ROOKWOOD ROAD, POTTS HILL

1. INTRODUCTION

This updated report presents the results of a geotechnical investigation carried out by Douglas Partners Pty Ltd (DP) for a proposed 330/132 kV Substation at Lot 101 of the Sydney Water Reservoir facility, located at Rookwood Road, Potts Hill. A report was previously issued on 25 February 2010, however, the proposed layout and scheme has changed. This updated report supersedes the previous report and reflects the current substation plans. The work was commissioned by TransGrid, owners and developers of the substation site.

The proposed substation will include the construction of three new 330/132 kV transformers, two gas insulated switchgear (GIS) buildings with possible cable basements, three 132 kV series reactors, one 330 kV shunt reactor, one 132 kV shunt reactor and two new in-ground primary and secondary oil containment tanks. Associated works will also include cable trenches linking the GIS buildings with the various bays and control buildings, noise walls and palisade fencing, concrete poles for overhead power lines and internal access roads.

Geotechnical investigation was undertaken to provide information on the expected subsurface conditions across the site for design and planning of earthworks, foundations and pavements.

The investigation comprised eleven boreholes and ten test pits at the approximate locations nominated by TransGrid, the installation of three groundwater monitoring wells and laboratory testing of selected soil samples followed by engineering analysis. Details of the field and laboratory work are given in this report, together with comments relating to design and construction practice.

Investigation or assessment of soil or groundwater contamination was not included in TransGrid brief or the scope for this geotechnical investigation.

2. BACKGROUND

Environ Pty Ltd prepared a Site Audit Report (Project No. AS120939) in June 2009 for Lot 101 and Part Lot 102 of the Potts Hill Reservoir facility. The report was commissioned by Sydney Water Corporation and included reference to historical records and a number of environmental investigations carried out by various consultants. The report indicates that the southern part of the site was occupied by a former Sydney Water Corporation metalising yard which was used for grit blasting and the application of protective coatings to metallic items. The southern part of the site was previously surfaced with varying concentrations of grit which was apparently removed during remediation work carried out in December 2008. Previous buildings on the southern part of the site were removed prior to the remediation works. The northern part of the site comprises vegetated slopes and embankments with no known former usage. Reference should be made to the Environ report for specific details on previous site development and associated contamination issues at the site.

In 2006 Douglas Partners (DP) prepared a report for the Potts Hill Reservoir Redevelopment (Project 43544) which included a review of information on previous investigations by DP, various other consultants and Sydney Water.

More recently, DP has been involved with geotechnical investigation and construction supervision for the adjacent commercial development to the west of the site (Project 45905). This included assessment and testing of impact rolling compaction of deep filling, which was encountered on the site to depths of up to 10 m.

3. SITE DESCRIPTION

The site of the proposed substation (Lot 101) is a rectangular-shaped property covering an area of approximately 2.72 hectares. At the time of the investigation the site was bounded by the Sydney Water "Reservoir 1" to the north, Rookwood Road to the east, an access road (under construction) to the south, and open space and a commercial building (under construction) to the west. The Potts Park Greyhound Track is located to the south of the site on the southern side of the access road. The location of the site is shown on Drawing 1 in Appendix A.

The site generally comprises two relatively level platforms (referred to as the northern and southern sections in this report) which are separated by a vegetated slope/embankment running east-west across the centre of the site. It is understood that the embankment was partly formed with spoil from construction of Reservoir 1. The embankment is approximately 7 m high and falls to the south (and also to the east along the Rookwood Road frontage) from approximately RL55 - 56 m to RL48 - 49 m relative to Australian Height Datum (AHD). On the eastern side of the site the embankment slopes at 20 - 30 degrees below horizontal, gradually reducing to 10-15 degrees on the western side of the site.

The upper northern section of the site comprises a relatively level area above the embankment with surface levels typically falling to the south-east at approximately 1 - 2 degrees from approximately RL56 - 57 m to RL55 - 56 m AHD. At the time of the investigation a large stockpile of clayey filling was located on the north-western part of the site and was being moved and spread out along an access track on the western part of the embankment. A number of mature tress are scattered across the northern part of the site.

The lower southern section of the site comprises a relatively level area at the base of the embankment with surface levels typically falling to the south-east at approximately 2 - 3 degrees from approximately RL47 - 48 to RL46 - 47 m AHD. A large stockpile of material was located at the toe of the embankment towards the eastern end of the site. The stockpile was covered with vegetation which limited visual inspection, however, it appeared to comprise clayey soil.

It is understood that the Sydney Water Corporation "City Tunnel" runs approximately east to west below the centre of the site. Historical drawings provided by TransGrid indicate that the tunnel is a 96 inch (i.e. 2.4 m) diameter, concrete lined tunnel, with an invert of approximately RL13.9 to RL4.4. On this basis the tunnel is expected to be at least 30 m below the site.

4. REGIONAL GEOLOGY

The Sydney 1:100 000 Series Geological Sheet indicates the site is underlain by Bringelly Shale which typically comprises shale, siltstone and fine to medium grained sandstone. The Soil Landscape Series Sheet 9130 indicates the site is underlain by soils of the Blacktown Group which are typically moderately reactive, highly plastic soils with low fertility and poor site drainage. With the exception of deep filling encountered on the upper northern part of the site, the geological mapping was confirmed by the field work, which identified filling and residual soils overlying shale.

5. FIELD WORK

5.1 Methods

The field work included eleven boreholes drilled to depths of 3.0 m to 10.0 m using a truck-mounted drilling rig and ten test pits excavated to depths of about 3.0 m using a backhoe fitted with a 0.3 m wide bucket. The borehole and testpit locations are shown on Drawing 1 in Appendix A.

The boreholes were drilled using solid flight augers fitted with a tungsten carbide (TC) drill bit. Standard penetration tests (SPTs) were undertaken in the boreholes to assess the soil strength and to obtain samples. Soil and rock samples were also collected from the tip of the auger at regular depth intervals.

The boreholes and test pits were logged and sampled by a geotechnical engineer on site and were backfilled immediately upon completion.

Dynamic cone penetrometer tests were carried out at each pit location to assess the soil strength (or in-situ compaction level) and were taken to depths of between 1.2 m to 2.4 m.

Groundwater observations were recorded during drilling of the boreholes and the excavation of the test pits. Groundwater monitoring wells were installed in boreholes BH3, BH15 and BH18 at the completion of the drilling to allow the future measurement of groundwater levels. The groundwater wells were constructed using 50 mm diameter PVC pipe installed to the base of each borehole and slotted over the lower 3 m.

The spatial coordinates of the bores were measured using a global positioning system (GPS) receiver which typically has an accuracy of 3 m to 7 m in the horizontal plane. The ground surface levels at the test locations were measured using optical survey equipment relative to Australian Height Datum (AHD) with State Survey Mark (SSM 91844) used as a reference benchmark. SSNM91844 was located on the corner of Rookwood Road and Brunker Road and a Reduced Level (RL) of 42.604 m AHD was indicated.

5.2 Field Work Results

The subsurface conditions encountered are presented in the borehole and test pit logs in Appendix B, together with notes defining descriptive terms and classification methods. Photographs of the test pits are also provided in Appendix B.

The general sequence of subsurface conditions encountered at the test locations is summarised below and the depths and reduced levels (to AHD) at which the different materials were encountered are shown in Tables 1A, 1B and 1C.

FILLING – encountered to depths of between 2.7 m to 6.7 m at test locations 1 to 12 inclusive (generally over the northern half of the site) and to depths of between 0.3 m to 1.4 m at test locations 13 to 21 inclusive (generally over the southern half of the site). Test pits 2, 4, 5, 6 and 12 were terminated in filling at depths of 3.0 m to 3.1 m. The deeper filling at test locations 1 to 12 generally included ripped shale (predominantly shale gravel) with varying proportions of clay, silt and sand and occasional boulders to 0.2 - 0.3 m diameter. The shallower filling at test locations 13 to 21 included predominantly clay and sandy clay with varying proportions of shale and sandstone gravel and cobbles, and brick fragments. The results of the SPTs and DCPs within the filling were generally consistent with stiff cohesive soils or medium dense granular soils.

NATURAL SILTY CLAY – below the filling, stiff to very stiff silty clay was encountered generally to depths of between 4.5 m to 8.5 m at Locations 1 to 11 and to depths of between 0.8 m to 2.3 m at Locations 13 to 21, inclusive. The clay was firm in BH18 and hard shaly clay was encountered in TP21 below 0.8 m depth.

SHALE – shale bedrock was encountered below the silty clay at depths ranging from 0.9 m to 8.5 m. The shale generally comprised extremely low to low strength rock approximately 1 m to 2 m thick over low to medium strength rock. Borehole 10 encountered practical auger refusal within the shale at a depth of 10 m.

Table 1A – Summary of Material Strata Levels at Test Locations

Material Strata	RL of Top of Material Strata (AHD, m)								
	BH1	TP2	BH3	TP4	TP5	TP6	BH7	BH8	BH9
Filling (Ground Surface)	56.7	56.5	54.8	56.0	56.0	56.0	56.3	53.4	56.0
Natural Silty Clay	-	-	52.1	-	-	-	51.2	49.4	49.3
EL-VL Shale	-	-	50.1	-	-	-	50.1	-	-
VL-L Shale	-	-	-	-	-	-	-	-	47.5
L-M Shale	49.8	-	48.7	-	-	-	48.7	46.6	-
Base of Borehole	48.7	53.4	48.3	53.0	53.0	52.9	48.3	45.9	47.0

Notes: EL = Extremely low strength; VL = Very low strength, L = Low strength, M = Medium Strength

Table 1B – Summary of Material Strata Levels

Material Strata	RL of Top of Material Strata (AHD, m)								
	BH10	BH11	TP12	BH13	BH14	BH15	TP16	TP17	BH18
Ground Surface	55.8	53.3	52.1	48.9	47.7	46.6	48.6	47.6	47.7
Residual Soil	49.9	49.5	-	48.1	47.4	46.2	47.7	46.2	47.2
EL-VL Rock	44.5	-	-	47.0	-	45.7	46.3	45.4	45.5
VL-L Shale	-	-	-	46.5	46.3	44.7	-	-	-
L-M Shale	43.3	47.1	-	-	44.0	41.6	45.8	-	-
Base of Borehole	43.3	46.3	49.0	44.9	43.7	40.6	45.6	44.6	44.7

Notes: EL = Extremely low strength; VL = Very low strength, L = Low strength, M = Medium Strength

Table 1C – Summary of Material Strata Levels

Material Strata	RL of Top of Material Strata (AHD, m)							
	TP19	TP20	TP21					
Ground Surface	47.7	46.7	46.1					
Residual Soil	46.7	45.9	45.8					
EL-VL Shale	45.8	45.2	44.2					
VL-L Shale	45.3	-	43.3					
L-M Shale	-	-	-					
Base of Borehole	44.6	43.7	43.1					

Notes: EL = Extremely low strength; VL = Very low strength, L = Low strength, M = Medium Strength

Groundwater was not observed during drilling of the boreholes or excavation of the test pits within the short period of time in which they remained open prior to backfilling. Groundwater was not present within the monitoring wells installed in BH3, 15 and 18 when they were inspected on 26/11/09 and 11/12/09.

6. LABORATORY TESTING

6.1 Geotechnical Testing

Selected soil samples were tested for a range of soil classification and engineering properties including Atterberg limits and linear shrinkage, Emerson Crumb (dispersion potential) and California bearing ratio (CBR). The results of these laboratory tests are presented in Appendix C and are summarised in Table 2.

Table 2 – Summary of Physical Laboratory Test Results

Test Location	Sample Depth (m)	Description	W_L (%)	PI (%)	LS (%)	Emerson (Class No.)	CBR (%)	Swell (%)
TP2	0.5-1.0	Filling: Ripped shale	33	16	9	4	3.5	0.8
TP5	0.5-1.0	Filling: Ripped shale	-	-	-	-	7	-0.1
TP6	0.5-1.0	Filling: Ripped shale	-	-	-	2	1.5	3.5
BH13	1.0-1.45	Silty clay: red grey	62	42	14.5	-	-	-
BH14	1.0-1.45	Silty clay: red grey	44	23	12	-	-	-
TP16	1.0-1.3	Silty clay: orange brown	-	-	-	4	0.5	4.3
TP17	0.1-0.4	Filling: brown grey silty clay	43	24	13	2	-	-
BH18	0.5-0.6	Silty clay: red grey	50	31	15.5	-	-	-
TP20	0.8-1.0	Silty clay: red brown	54	35	13.5	4	0.5	8.5
TP21	0.5-1.0	Silty clay: red brown	-	-	-	6	1.0	4.0

Notes: W_L = Liquid limit; PI = Plasticity index; LS = Linear shrinkage; CBR = California bearing ratio (4 day soak, 4.5 kg surcharge); Swell = vertical expansion of sample in CBR mould due to sample saturation.

6.2 Soil Aggressivity Testing

Four soil samples were selected for laboratory chemical analyses to determine the aggressivity of the site soils to buried structural elements (e.g. steel and concrete). The suite of analyses included pH, sulphate (SO_4^{2-}) and chloride (Cl^-) ion concentrations within the soils. The laboratory test results are included within Appendix C and are summarised in Table 3.

Table 3 – Summary of Chemical Test Results

Bore Location	Sample Depth (m)	Description	SO_4^{2-} (mg/kg)	Cl^- (mg/kg)	pH (pH units)
BH3	3.0	Silty clay: red grey	280	44	4.4
BH14	0.1	Filling: grey brown silty clay	540	40	9.4
BH15	0.5	Silty clay: red brown	590	1300	4.6
TP17	2.4	Shale	420	210	4.9

Notes: SO_4^{2-} = Sulphate ion; Cl^- = Chloride ion

7. GEOLOGICAL MODEL

The site comprises two relatively level areas (the northern and southern sections) which are separated by an embankment approximately 7 m high. It is understood that the embankment was formed using material sourced from construction of Reservoir 1 to the north of the site.

The upper, northern part of the site is underlain by deep filling to depths of between 3 m and 7 m then residual silty clay overlying weathered shale at depths of 4.5 m to 8.5 m. The lower, southern part of the site is underlain by shallower filling to depths of approximately 0.5 m to 1.5 m, then residual silty clay over weathered shale at depths of 1 m to 2.5 m. Within the site, the shale surface appears to fall towards the east and south-east from approximately RL50 to RL44 AHD.

The depth of filling encountered at each of the test locations is shown on Drawing 2 and a plan showing the inferred rock surface contours (to AHD) is shown on Drawing 3 in Appendix A. The rock surface contours are approximate only and variations between the test locations should be expected. The rock surface in BH8 and BH9 is slightly inconsistent and differs by about 0.5-1.5 m with the apparent trend of the rock surface. Five interpreted geological cross-sections (A-A' to E-E') are presented in Drawings 4 to 8, respectively, in Appendix A. The orientation of these cross-sections is shown in Drawing 1. Again, the subsurface conditions are accurate at borehole and test pit locations only and variations should be expected away from the test locations.

The filling on the northern part of the site appears to comprise mostly ripped shale whilst the filling on the southern part of the site comprises mostly clay and sandy clay. The limited testing within the filling indicates it is generally consistent with stiff clay or medium dense granular soil and suggests that some compaction has probably been applied during placement. Some areas of poor compaction and unsuitable materials were encountered (e.g. soft zones in TP17 with organic material and a tree branch). Previous investigations on the site to the west encountered deep and variably compacted filling with some zones of apparently uncontrolled filling.

The residual silty clays are generally stiff to very stiff although some localised areas of firm clay may also be expected on the lower part of the site, as indicated by BH18.

The Atterberg limits generally indicate that the filling contains cohesive soil of medium plasticity whilst the residual clays are generally of medium to high plasticity and would therefore be expected to be moderately to highly susceptible to volume changes in response to changes in the moisture content, due to climatic or other factors.

The Emerson dispersion testing indicates the filling varies from non-dispersive to moderately dispersive whilst the residual clays appear to be generally non-dispersive.

Managing Urban Stormwater - Soils and Construction, Volume 1 (4^{th} Edition, March 2004), commonly referred to as the "Blue Book", provides classification of soils in terms of a Soil Texture Group (i.e. Type C, D and F). It is considered that the residual clay and clay filling on the site would generally be "Type D" soils, based on the proportion of silt and clay fines, however, it is also noted that the laboratory testing has indicated that these soils are moderately to non-dispersive. In its current state, the ripped shale filling is considered to be a "Type C" soil based on the proportion of coarse grained material. The shale filling may need to be reassessed if reworking and compaction of the material results in a considerably higher proportion of fines.

The CBR results indicate that the clay soils and even the ripped shale filling lose substantial strength upon saturation. Three samples of silty clay (TP16, 20 and 21) and one sample of ripped shale filling (TP6) experienced excessive swelling and strength reduction during the CBR testing. The reduction is strength due to saturation is evident from the comparative penetration tests carried out on the bottom of these samples, as shown on the test result sheets included in Appendix C. For example, the penetration testing on the bottom of these four samples yielded a "non-saturated" CBR value of between 4.5% to 7%%, compared to 0.5% to 1.5% for the top of the sample. The soil at the bottom of the CBR mould typically remains at, or close to, the optimum compaction moisture content, whereas the top of the sample becomes significantly wetter, due to the absorption of water over the four-day soaking period.

Groundwater was not observed within the test pits or boreholes and was not measured within the groundwater monitoring wells which were installed to depths of 3 m to 6 m. The Site Audit Report by Environ Pty Ltd indicated that previous investigations encountered groundwater at depths of greater than 8 m. The previous investigation by DP on the property to the west of the site encountered groundwater seepage within the filling at some locations at depths of about 4 m to 6 m. Although not encountered during the current investigation, localised and perched groundwater could still be expected within the filling and groundwater seepage may occur along the top of the clay and rock surface, particularly following periods of extended wet weather.

8. PROPOSED DEVELOPMENT

The proposed substation will include the construction of three new 330/132 kV transformers, two gas insulated switchgear (GIS) buildings with possible cable basements, three 132 kV series reactors, one 330 kV shunt reactor, one 132 kV shunt reactor and two new in-ground primary and secondary oil containment tanks. Associated works will also include cable trenches linking the GIS buildings with the various bays and control buildings, noise walls and palisade fencing, concrete poles for overhead power lines and internal access roads.

9. COMMENTS

9.1 Site Preparation & Earthworks

9.1.1 Excavation Conditions

The floor levels for the various structures are shown on a drawing provided by TransGrid (Plan – Option C, Ultimate, Figure 2.3). These proposed floor levels are shown on the interpreted geological cross-sections presented in Drawings 4 to 8, inclusive and show the extent of excavation and filling. Note that the cross-sections are presented at an exaggerated vertical scale (i.e. 5 times), such that the batter slopes look significantly steeper than would be possible, or necessary. A summary of the expected depths of excavation and filling for the various structures is provided below:

- **Capacitor bank, Shunt reactor and 132 kV GIS building** (RL50.0) on the upper northern end of the site - excavation to depths of about 5 m to 6 m.
- **132kV Series Reactors** (RL46, RL47 and RL48) close to the embankment on the northern end of the site - excavation into the slope to depths of approximately 7 m to 8 m.
- **PST** (RL49.5) on the western side of the site - excavation to depths of 5 m with the depth of excavation reducing in depth to the south.
- **PST** (RL49.5) on south-western corner of the site - excavation to depths of about 1.5 m at the northern end and filling up to 2 thick on the southern side
- **Transformer building** (RL46, RL47 and RL48) towards the toe of the embankment - excavation to depths of 2 m to 3 m reducing in depth towards the south.
- **330kV Reactor Building** (RL48.75) towards the centre of the site – excavation into the slope to depths of approximately 3 m to 4 m.

- **330kV GIS Building** (RL46.5) on the southern-eastern corner of the site – generally close to existing surface level with some excavation and filling to less than 1 m depth.

The excavations will be primarily through the filling and natural clay with the deeper excavations expected to encounter extremely low then low to medium strength shale. It is possible that some medium to high strength ironstone bands may be encountered within the weathered rock sequence.

It is considered that excavation in the filling, natural clays and extremely low to low strength rock should be readily carried out using conventional earthmoving equipment (e.g. bulldozers and hydraulic excavators). Deeper excavations through medium strength rock, where encountered, will generally require moderate ripping in conjunction with medium-sized hydraulic rock hammers.

9.1.2 Goundwater

Although not encountered during the investigation, groundwater seepage may still be encountered along the top of the clay and rock surface, particularly following periods of extended wet weather. Localised and perched groundwater seepage may also occur with the deeper filling, particularly through zones of poorly or uncontrolled filling. Some provision should be made for handling seepage inflows during deeper excavations. It is anticipated that this seepage could be readily controlled by "sump-and-pump" dewatering techniques and perimeter drainage used to direct seepage to the stormwater drainage system.

9.1.3 Disposal of Excavated Materials

The materials that will be derived from the excavation works will generally include significant amounts of filling, natural soil and weathered shale. It should be noted that any off-site disposal will require assessment for re-use or classification of the excavated material in accordance with the "*Waste Classification Guidelines" by DECC (2008),* updated July 2009, prior to disposal at an appropriately licensed landfill.

9.1.4 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings, pavements and structures before the commencement of any subgrade preparation (i.e. using heavy compaction plant) or excavation work in order to document any existing defects so that any claims for damage due to vibrations or construction related activities can be accurately assessed.

9.1.5 Unsupported Excavations

Where space permits, excavations may be battered for the bulk excavation works and new structures. The maximum temporary and permanent batter slopes, for cuts up to 4 m high, are given in Table 7. Specific details of the bulk excavation levels across the site and between structures have not been confirmed. Further geotechnical review and stability analysis of slopes exceeding 4 m in height may be required once specific details of the proposed ground levels and structures/pavements have been confirmed.

Table 4 – Maximum Recommended Batter Slope Ratios

Material	Temporary Batter Slope Ratio (H:V)	Permanent Batter Slope Ratio (H:V)
Filling	1.5:1	3:1
Natural Clay	1.5:1	3:1
Weathered Rock[1]	0.5:1[1]	1:1[1]

1. Dependent on the presence and orientation of joints, such that an inspection and assessment by an experienced geotechnical practitioner should be carried out during construction.

All batters should be subject to geotechnical inspection for every 1.5 m of vertical excavation to confirm the adequacy of the slopes indicated above. Where there is insufficient space for batters, some form of shoring or permanent retention would generally be required as described in Section 8.2.

9.1.6 Trafficability & Erosion

The silty and clayey fine grained soils at the site were not shown to be highly dispersive but may still be prone to erosion from concentrated water flows and disturbance from construction machinery. Trafficability conditions on the clayey soils are expected to be generally poor, particularly following wet weather. The ripped shale filling, on the northern part of the site, will provide improved trafficability but will deteriorate when exposed to wet weather and prolonged traffic.

The clayey soils (both natural and filling) have indicated very low soaked CBR values of between 0.5% to 1.5% and therefore some form of stabilisation will be required to produce a soil that is less susceptible to moisture softening and to allow the use of a higher CBR value for pavement design purposes (this is described in more detail in Section 8.4).

Where the clayey soils are exposed during the construction phase, it is recommended that the surface is completely covered (after proof rolling) with a protective, running-surface layer. This protective layer may not necessarily be required for the ripped shale on the northern part of the site but should still be considered to minimise the effects of wet weather, particularly for areas subject to heavy construction traffic. The protective layer should be at least 150 mm thick and should comprise hard, durable crushed rock or recycled concrete with a maximum particle size of 60 mm. The rockfill material should also have less than 5% of particles passing through the 75 micron sieve. This type of material could also be used to provide a temporary surface layer for unformed roads.

Working platforms for heavy tracked plant, such as piling rigs, may require an additional thickness of crushed rock, possibly with geofabric layers for extra support and should be the subject of a specific geotechnical assessment when plant and earthworks details are confirmed.

The results of the dispersion testing indicate that the natural site soils are typically non-dispersive, with Emerson Class Numbers of 4 or 6. One sample of clayey filling (TP17 at 0.1 - 0.4 m) and one sample of ripped shale filling (TP6 at 0.5 - 1.0 m) yielded a Class Number of 2, which is indicative of moderately dispersive soil. It is noted, however, that the sample of ripped shale is predominantly granular material within a matrix of silt, sand and clay of varying proportions. It is therefore likely that this sample is principally an erodible material, which due to the test method (involving dropping the soil sample into water), has experienced some sample disintegration rather than true dispersion. The difference between an erodible and a dispersive soil can be viewed in terms of the velocity of the water affecting the material. For example, a soil for which the soil matrix remains intact when immersed in still water is generally non-dispersive. However, the same soil could be disturbed and eroded when subjected to concentrated water flows or run-off. Sand, and to a lesser extent silt, are inherently erodible soils due to the weak bonds holding the soil matrix together, when compared to clays.

So, while the existing site soils are expected to be generally non-dispersive, some of these materials contain significant proportions of silt and also sand are thus likely to be prone to erosion.

Permanent batter slopes in cut or for filled platforms should be vegetated as soon as possible, to reduce the risk of significant soil erosion occurring. As shown in Table 4, where soil erosion is of concern, permanent batters should be no steeper than 3H:1V. For temporary batters, it would be prudent to cover the exposed soils with (heavy-duty) plastic or a thin layer of shotcrete. Additional contour drains, particularly along the crest of cut or filled batters may also be required to reduce the potential for erosion. Such drains should be vegetated or concrete-lined, as appropriate for the expected flows.

New general bench areas should also be protected from concentrated water flows with a layer of durable gravel and good drainage measures such as adequate surface catch drains and cross falls. Table drains around the perimeter of these benched areas should be protected with grassed topsoil and jute-mesh, as appropriate.

To ensure adequate compaction on the outside face of earthfill batters, for protection against erosion, fill construction should be extended several metres beyond the design extent of the slope and then be trimmed back to final batter face. A minimum 100 mm thick layer of topsoil should be installed as quickly as possible, with the appropriate (grass) hydro-seeding and jute mesh coverage.

9.1.7 Subgrade Preparation for Deeply Filled Areas

The northern part of the site is underlain by deep filling whilst the southern part of the site is underlain by relatively shallow filling. Based on the testing and also previous experience on the adjacent site to the west, the filling is variably compacted and may include some areas of uncontrolled filling. The variable compaction within the existing filling could give rise to differential settlement unless some form of treatment is adopted. In the absence of detailed information on the filling placement and compaction or further detailed investigations to confirm otherwise, the existing filling is not considered suitable to support structures or floor slabs without the risk of excessive differential settlement. It is noted that the buildings on the northern part of the site generally involve excavation to RL50 which is generally below the filling identified in the boreholes. As shown on Drawing 7, the filling may extend to about 1 m below the 132 KV GIS building on the north-eastern part of the site.

Where the filling is relatively shallow it may be possible to remove and replace the filling with engineered filling, however this option is unlikely to be economical for areas underlain by deep filling. Where deep filling is present the use of high energy impact compaction (HEIC) may be considered to improve the near surface compaction and provide more uniform support. Based on experience, it is expected that the HEIC may improve the compaction of the filling to a depth of about 1 m to 2 m. The use of HEIC may be considered to allow construction of lightly loaded floor slabs and pavements on ground, however, all movement-sensitive structures should generally be supported on the underlying rock.

Compaction of the surface using a high energy impact roller would be carried out from the existing surface to compact the underlying filling and to provide a bridging layer. This method involves the use of a three to five sided heavy roller to compact the ground. The high energy impact roller typically has a greater depth of influence than that of a conventional drum roller.

Impact compaction is generally effective in granular soils (such as ripped shale), however difficulties, are sometimes encountered using impact compaction in clayey soils. Accordingly, the approach should initially be carried out over a trial area to assess the number of passes required and the effectiveness of the process. During impact rolling of the exposed filling, it is suggested that surface levels are taken at regular intervals. As a guide levels could be measured prior to rolling, after 10 and 20 passes and then after every 5 passes to measure the amount of induced settlement. A typical acceptance criterion is less than 5 mm settlement per 5 passes. Experience would suggest that the number of passes required over the same area could be between 30 and 40.

Pre and post testing of the filling using cone penetration tests (CPTs) and possibly plate loading tests should be carried out to assess the effectiveness of HEIC.

The final surface should be levelled and compacted to between 100% and 102% of Standard maximum dry density (SMDD) and within 2% of optimum moisture content (OMC). The strict control on SMDD and moisture content is required to reduce the risk of long-term swelling of the soil due to over compaction of dry cohesive soils.

Past experience suggests that vibrations due to high energy impact rollers will probably be of the order of 10 mm/s at about 6 m distance reducing to about 5 mm/s at 20 m distance. A vibration trial should be carried using the HEIC equipment on site in order to assess vibration levels at varying distances and to establish safe working distances/clearances.

The HEIC treatment will improve the compaction of the near surface filling but is unlikely to have any effect on filling below about 1 m to 2 m depth. For this reason, even after HEIC treatment, there still remains a risk of post construction settlements, which may lead to some differential ground movement over the longer term. In the absence of more detailed investigation and comprehensive information on the filling consistency and compaction, it is not possible to provide a reasonable estimate of likely settlements.

As a guide, consolidation settlements of 0.5% to 2% of the filling thickness may be expected for compacted and uncompacted filling, respectively, due to the self weight of the material. By way of example, settlements of between 25 mm to 100 mm could be expected for a 5 m thick layer of compacted and uncompacted filling, respectively. The filling on the northern part of the site has been in place for many years, and therefore much of the consolidation settlement has probably already occurred, however, some ongoing creep settlement could still be expected. In addition to the creep settlement, there will also be settlement of the filling due to the increased loading from the floor slabs and pavements.

9.1.8 New Engineered Filling

Areas on the site that are to be filled should be stripped of vegetation, organic topsoil and existing filling (unless HEIC is used as an alternative). The exposed subgrade should be proof-rolled in the presence of an experienced geotechnical professional using a minimum 12 tonne pad-foot or steel smooth drum roller. Any soft or spongy areas should be excavated to depths as directed by an experienced geotechnical practitioner and backfilled with clean engineered filling.

Engineered filling required for replacement of soft areas or raising of site levels should be placed in layers not exceeding a loose thickness of 250 mm and compacted to a dry density ratio of between 98% and 102% SMDD and strictly within 2% of the OMC.

The existing filling, natural soils and weathered rock at the site should be suitable for re-use from a geotechnical perspective, providing that all topsoil, oversize (particles greater than 100 mm) or other deleterious material is removed prior to compaction. Some of the shale cobbles and boulders may be effectively broken down during the compaction process. Preference should be given to the use of granular material (such as ripped shale) closer to the subgrade surface.

The Atterberg limit and linear shrinkage test results indicate the soils on the site are moderately to highly reactive and therefore significantly susceptible to changes in moisture content. It will therefore be important to place this material as close to optimum moisture as possible, and to avoid over-compaction, to reduce the potential for significant shrink/swell movements.

Density testing of the filling should be carried out according to AS 3798 *"Guidelines for earthworks for commercial and residential developments"* at a minimum rate of 1 test per layer, per 500 m^2. It is recommended that all compaction control testing in areas that will support structures and pavements be undertaken with a Level 1 responsibility, as defined in AS 3798.

Deep engineered filling compacted in accordance with this report may experience long-term settlements in the order of 0.5% of the filling depth. This is in addition to settlement caused by the application of loads on the surface of the filling.

9.2 Excavation Support

9.2.1 General

Where space does not permit unsupported cut batters for excavations, shoring support of the excavations would generally be required. Cantilevered, braced or anchored soldier piles, installed using bored or continuous flight auger (CFA) methods may be used to provide temporary excavation support. These walls consist of rock-socketed, reinforced concrete piles spaced at centres of up to about 2.5 m, with shotcrete infill panels. Permanent drainage measures such as strip drains and "spitter pipes" exiting the wall face just above the bulk excavation level should be incorporated behind the walls to prevent hydrostatic (groundwater) pressure build up.

The use of proprietary excavation support systems such as trench or shoring boxes may be considered for shallower, temporary trench excavations which are not located close to adjacent structures or pavements.

It is anticipated that steel sheet piles would not be preferred for this site as they may have difficulty penetrating cobbles and boulders within the filling and may not be able to penetrate the underlying shale bedrock to achieve the desired embedment below bulk excavation level. The use of sheet piles will also be subject to assessment of noise and vibrations on surrounding structures.

9.2.2 Design

The design of the shoring walls will depend somewhat upon whether it is cantilevered or restrained by multiple rows of temporary rock anchors. The preliminary design of cantilevered shoring systems (or shoring with a single row of "tie-back" ground anchors) may be based on the following triangular earth pressure distribution:

$$h_z = Kz\gamma$$

where h_z = horizontal pressure at depth z
γ = unit weight of soil or rock
K = earth pressure coefficient

Suitable parameters for the design of shoring and retention systems are given in Table 5.

Table 5 – Design Parameters for Retaining/Shoring Walls

Material	γ (kN/m^3)	K_a	c_u (kPa)	c′ (kPa)	Ø′ (°)
Filling and Natural Soils	20	0.35	50	2	25
Weathered Rock (extremely low to medium strength)	22	0.25	200	10	30

Notes: γ= bulk unit weight; K_a = coefficient of active earth pressure; C_u = undrained cohesion; c´= effective cohesion; ø' = effective friction angle.

The effective strength parameters shown in Table 8 may be adopted for the design of retaining/shoring walls and slope stability analysis (if required). Undrained cohesion should only be used for the analysis of temporary slopes and cuts and should not be relied on in situations where worker safety is at risk.

The above earth pressure coefficients assume a level area behind the wall and do not include allowance for inclined backslopes, footings or construction related activities, all of which must considered in addition to the above earth pressures. Care should be taken when using earthworks or rolling plant near retaining/shoring walls and hand-held plate compactors may be required directly behind such walls to reduce surcharge loading, possibly with a reduction in filling layer thickness to maintain the level of compaction required.

Preliminary design for lateral earth pressures for walls with more than one row of anchors may be based on a uniform rectangular earth pressure distribution. This may be applicable for the deeper excavations on the northern end of the site for the capacitor banks, shunt reactors, 132kV GIS building, and 132kV series reactors and also for the PST on the western end of the site. The additional lateral pressures due to surcharge loading behind the wall and hydrostatic pressures (if appropriate) must also be considered. Where lateral movement is less critical (as generally expected for this site) a uniform lateral pressure distribution of 4H may be considered (where H is the height of the wall). For situations where movements are critical, a higher uniform pressure of 6H or 8H may be adopted, depending on how sensitive the existing structure is behind the wall. For detailed design of walls greater than 5 m high a computer analysis package such as WALLAP, FLAC, PLAXIS or similar should be used to model the excavation and anchoring sequence, to refine the design and provide estimates of possible lateral movements.

Lateral restraint will also be developed by embedding the shoring piles below the bulk excavation level and developing passive pressure or resistance. The ultimate passive resistance available by embedding the piles below bulk excavation level and the minimum "toe-in" (i.e. embedment depth) may be estimated using the values in Table 6.

Table 6 – Passive Pressures and Coefficients

Material	Passive Pressure (P_p) (kPa)
Shale :extremely low to very low strength	400
Shale : low to medium strength	2000

The above values may be considered from a depth of one pile diameter below the bulk excavation level, or any adjacent excavations in front of the wall (e.g. drainage trenches). It is noted that the above values are 'ultimate' values mobilised following significant movement of the pile and should therefore incorporate an appropriate factor of safety to reduce wall deflections.

Where required, the preliminary design of ground anchors may be based on a maximum allowable bond stress of 200 kPa and 400 kPa for extremely low to very low strength and low to medium strength shale, respectively. This bond stress is applicable for the grouted length of the anchor that is behind the "active" zone, estimated by a line rising at 45 degrees from the base of the excavation to intersect the ground surface. Proof stressing of trial anchors should be used to confirm that the bond stress values adopted in the design are appropriate and/or to check if higher or lower values are warranted. Anchor holes should be properly cleaned and free of smear prior to installation of the anchor strands and grouting.

The effects of hydrostatic pressure should also be included in lateral pressure calculations for wall design purposes, where drainage behind retaining walls is not sufficient to prevent the build-up of groundwater. The buoyant weight of the soil may be considered below the adopted groundwater level for design purposes.

9.3 Foundations

9.3.1 Site Classification

Due to the presence of deep filling the current site classification in accordance with AS 2870 – 1996 is Class “P”. The final site classification will, however, vary in some areas where excavation is carried out to expose natural clay or shale. As a guide, sites underlain by natural clay at least 1.5 m thick would be classified as Class M or Class H. Once cut and fill works have been undertaken the site classification could be re-assessed to reflect the actual subsurface profile.

9.3.2 Spread Footings

Structures should not be founded in the existing filling, due to the variability of its composition and inferred variable level of compaction. Rather, footings should be uniformly founded on either controlled engineered fill, stiff natural clay or bedrock. Shallow or spread footings (e.g. pads or strips) may be designed using the parameters given in Table 7.

Table 7 – Design Parameters for Spread Footings

Material	Maximum Allowable Bearing Pressure (kPa)
Controlled Filling	100
Natural Stiff Clay	100
Shale : extremely low to very low strength	700
Shale : low to medium strength	1500

The settlement of a spread footing is dependent on the dimensions of the footing and the load applied. As a guide, a 1 m square pad footing founded in soils designed using the parameters provided in Table 10 may experience settlement in the order of 10 mm to 15 mm due to the application of the design working load. Differential settlements between footings founded in similar strata are expected to be less than 50% of the value of total settlement.

Minimum founding depths should be incorporated for footings supporting lightly loaded structures, given the relatively high shrink-swell potential indicated for the natural clays at the site. For example, a minimum founding depth of 1.2 m would be required for footings that are unprotected by perimeter slabs or paving.

All footings should be inspected and where in soil materials, tested by a geotechnical practitioner conducting Dynamic Cone Penetrometer (DCP) tests prior to concrete placement.

9.3.3 Piles

Heavily loaded or settlement-sensitive structures should be uniformly supported on the underlying shale bedrock. Piles will generally be required to reach the level of weathered bedrock on the north-eastern part of the site and in some areas on the southern part of the site where only minor excavation is carried out, or where the site is raised with filling.

Bored piers should be suitable for deep foundation systems at the site. The use of temporary casing will generally be required over the depth of filling to prevent collapse of the pier holes and the ingress of groundwater seepage. Alternatively, the use of CFA concrete or grout-injected piling could be used to eliminate the need for casing or dewatering of pile holes.

Rock-socketed bored piles could be designed using the parameters outlined in Table 8.

Table 8 – Allowable Geotechnical Parameters for Bored Piles (Working Stress Approach)

Material	End-Bearing Pressure (kPa)	Shaft Adhesion (Compression)* (kPa)	Shaft Adhesion (Tension)* (kPa)
Shale : Extremely low to very low strength	700	70	50
Shale : Low to medium strength	1500	150	120

** Only applicable where adequate socket roughness is achieved*

Settlement of piles subjected to vertical loads will vary depending on the loads applied and the foundation conditions below the pile toe. As a guide, piles designed using the parameters outlined in Table 8 can expect settlements of less than 5 mm under design loads.

Alternatively, piles designed using the limit-state approach could be proportioned using the ultimate geotechnical design parameters outlined in Table 9.

Table 9 – Ultimate Geotechnical Parameters for Bored Piles (Limit State Approach)

Material	End-Bearing Pressure (kPa)	Shaft Adhesion (Compression)* (kPa)	Shaft Adhesion (Tension)* (kPa)	Typical Young's Modulus (E) (MPa)
Shale : Extremely low to very low strength	3000	150	100	100
Shale : Low to medium strength	6000	350	250	300

** Only applicable where adequate socket roughness is achieved*

The selection of a geotechnical strength reduction factor (ϕ_g) will be required in accordance with the revised AS2159-2009. Selection of the ϕ_g value is based on a series of individual risk ratings (IRR) which are weighted and lead to an average risk rating (ARR). The IRR and final value of ϕ_g depend on factors such as; the type of investigation/testing, design methods and parameter selection, pile testing regime and redundancy of the design. For preliminary design purposes, it is anticipated that a ϕ_g of 0.5 may be appropriate, however, this will be subject to review and consideration of the factors described above.

It should be noted that the use of the limit state approach for the design of piles generally results in higher pile capacities compared to the working stress approach. These higher loads are likely to result in increased settlements and therefore the serviceability requirements of the piles will need to be considered in the design process. The Young's Modulus (E) values in Table 9 can be used with the serviceability loads to estimate settlements.

The design of vertical piles required to resist uplift loads should be checked against the "cone-pullout" failure mode as well as the shaft capacity of the pile.

For a rock-socketed, bored pier, any contribution to bearing capacity from shaft adhesion within the filling and soil overburden should be ignored due to the incompatibility of strains required to mobilise shaft adhesion within the overburden and the weathered bedrock.

All bored piles should be inspected by an experienced geotechnical professional to check to adequacy of the foundation material and the roughness of the rock-socket.

9.4 Pavements

Laboratory test results have indicated soaked CBR values of 0.5% to 1.0% for the natural silty clay and 1.5% to 7% for the ripped shale filling. The lower CBR value of 1.5% for the ripped shale is probably only applicable for areas where the ripped shale filling may contain a higher clay content or where it is broken down significantly during compaction and construction traffic.

A subgrade CBR of 3% could be adopted for pavement design purposes although it should be noted that some form of subgrade improvement will be required to achieve the design CBR on the southern half of the site where clayey soils are expected at the subgrade surface. Areas that require stabilisation or improvement could be delineated following inspection by a geotechnical engineer at the time of bulk earthworks and subgrade preparation.

Subgrade improvement could involve providing a select layer to effectively increase the subgrade CBR. For example, placing and compacting 300 mm of granular material with a CBR of at least 20% would increase the effective subgrade CBR from 0.5%-1% to 3%. Alternatively, lime stabilisation could be undertaken to increase the strength of the subgrade. Additional laboratory analysis will be required to determine an appropriate dosing rate, although a preliminary rate of 4% (by dry weight) could be assumed for planning purposes.

Appropriate cross-falls and subsurface drainage should be installed to reduce the risk of the clayey subgrade becoming saturated during periods of wet weather. Subsoil drainage should be installed to not less than 500 mm depth below subgrade level on the high side of the new pavement(s) and along any gravel or garden areas, where the ingress of water beneath the pavement subgrade is likely.

9.5 Soil Aggressivity

Based on the results of the pH, sulphate and chloride test results (refer to Table 3) the site soils and shale may be assumed to have a ‘mild’ to ‘moderate’ exposure classification with respect to buried, reinforced concrete, structural elements (Table 6.4.2(C), AS 2159 - 2009).

Reference should be made to Table 6.4.3 of AS 2159 – 2009 to determine the minimum concrete cover to reinforcement required, based on this exposure classification, minimum concrete strength and design life of either 50 or 100 years.

10. LIMITATIONS TO THIS REPORT

DP has prepared this report for this project at Lot 101 Rookwood Road, Potts Hill as per DP's proposal dated 9/11/09 and acceptance from TransGrid (email of 10/11/09), and in accordance with the Period Contract between DP and TransGrid (ref: Order Number 642877). This report is provided for the exclusive use of the TransGrid for the specific project and purpose outlined. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

The testing methods adopted are indicative of the site's subsurface conditions to the depths penetrated at the specific sampling and/or testing locations in this investigation, and only at the time the work was carried out. The accuracy of geotechnical engineering advice provided in this report may be limited by unobserved variations in ground conditions across the site in areas between and beyond test locations and by any restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. These factors may lead to the possibility that actual ground conditions and materials behaviour observed at the test locations may differ from those which may be encountered elsewhere on the site. Should such variations in subsurface conditions subsequently be encountered, then additional advice should be sought from DP.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from others' review of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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