The Sikh Grammar School Sydney
C/- PMDL

Preliminary Salinity and Geotechnical Assessment: The Sikh Grammar School - 151 and 161 Tallawong Road, Rouse Hill, NSW

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ENVIRONMENTAL

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All enquiries regarding this project are to be directed to the Project Manager.

Contents

1 DEVELOPMENT AND INVESTIGATION SCOPE 5

2 GENERAL SITE DETAILS AND SUBSURFACE CONDITIONS 6

3 SALINITY ASSESSMENT 7

3.1 Documented Salinity Risk Potential 7

3.2 Broad Scale Salinity Processes 7

3.3 Signs of Potential Saline Soils at the site 7

3.4 Assessed Salinity Risk Potential 7

3.5 Laboratory Testing 9

3.5.1 Overview 9

3.5.2 Results – Salinity Classification 9

3.5.3 Results – Exposure Classification 10

3.6 Conclusions and Recommendations 12

4 GEOTECHNICAL ASSESSMENT 13

4.1 Soil Reactivity 13

4.2 Preliminary Soil and Rock Strength Properties 13

4.3 Risk of Slope Instability 14

4.4 Geotechnical Recommendations 15

4.5 Allowable Bearing Capacities 19

5 PROPOSED ADDITIONAL WORKS 21

5.1 Works at Construction Certificate 21

5.2 Construction Monitoring and Inspections 21

6 REFERENCES 23

7 ATTACHMENT A – FIGURES 24

8 ATTACHMENT B – BOREHOLE LOGS 27

9 ATTACHMENT C – DCP 'N' COUNTS 39

10 ATTACHMENT D – LABORATORY TEST CERTIFICATES 41

11 ATTACHMENT E – GENERAL GEOTECHNICAL RECOMMENDATIONS 50

12 ATTACHMENT F – NOTES ABOUT THIS REPORT 53

1 Development and Investigation Scope

The proposed development details and investigation scope are summarised in Table 1.

Table 1: Summary of proposed development and investigation scope.

Item	Details
Property Address	151 and 161 Tallawong Road, Rouse Hill, NSW ('the site')
Lot / DP	Lots 42 and 43, DP 30186
LGA	Blacktown City Council (BCC)
Assessment Purpose	Preliminary salinity and geotechnical assessment to support a State Significant Development Application (SSDA).
Site Area	Approximately 4.05 ha (CSS, 2013)
Proposed Development	We understand from a brief by client and proposal plans (PMDL, 2019) that the development will include: o Construction of a new school comprising primary and secondary schools, an early learning centre, a community centre Gurdwara (a place of worship) and Langar (community dinning space), a boarding house, a library and staff lounge, a village green, sports pavilion and field and school drop-off and parking area. Some of the proposed buildings will likely extend into an area occupied by an existing site dam. The north east 1/3 as well as western corner of the new school footprint will include a basement carpark, requiring bulk excavations up to 4.0 m below ground level (mBGL). o Future subdivision for low density residential developments to the south west of new school.
Investigation Scope of Work	o A general site walkover survey. o Eleven boreholes (BH101 to BH111) up to 4.0 metres below ground level (mBGL) (refer Attachment B, and associated explanatory notes in Attachment F). o Collection of soil and weathered rock samples for laboratory testing and future reference. o Eleven Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP111) up to 2.6 mBGL (refer DCP 'N' counts in Attachment C). Investigation locations are shown in Figure 1, Attachment A.
Laboratory Testing	Testing, carried out by National Association of Testing Authorities (NATA) accredited laboratories, included: o Soil Reactivity testing on three soil samples by Resource Laboratories. o Chemical testing (Electrical Conductivity (EC), pH and soluble SO_4) on twenty four soil samples by Envirolab Services. Laboratory test certificates are provided in Attachment D.

2 General Site Details and Subsurface Conditions

General site details and encountered subsurface conditions are summarised in Table 2.

Table 2: Summary of site details based on desktop review, site walkover and site investigations.

Item	Comment
Topography	Within slightly undulating terrain
Typical slopes, aspect, elevation	The site generally has a north westerly and westerly aspect across the southern portion and south westerly aspect across the northern portion, with typical grades of between approximately 5 % and 15 %. Site elevation ranges between approximately 41.5 mAHD (western corner) and 54 mAHD (eastern corner).
Existing development	Current development at the site includes a single storey brick residential dwelling, located in the eastern corner. A dam is located in the central portion of the site. An east to west aligned drainage depression extends across the central portion of the site and connects the site dam to other dams located to the southwest and northeast of the site.
Vegetation	Grass, shrubs and scattered trees
Drainage	Via overland flow into the drainage depression and existing dam.
Expected soil landscape	The NSW Environment and Heritage eSPADE website identifies the site in the Blacktown soil landscape, consisting of shallow to moderately deep hard setting mottled texture contrasting soils, red and brown podzolic soils on crests grading to yellow podzolic soils on lower slopes and in drainage lines.
Sub-surface soil / rock units	<u>Unit A</u>: Topsoil comprising silt / clayey silt with consistencies varying from soft to very stiff up to approximately 0.2 mBGL. <u>Unit B</u>: Residual soil comprising generally very stiff grading to hard silty clay / clay up to between approximately 0.55 mBGL and 3.8 mBGL. A deeper soil profile in BH105 and BH110, encountered up to 3.8 mBGL and 2.7 mBGL, respectively, is inferred to be the result of more extensive rock weathering due to surface water infiltration along the drainage depression. <u>Unit C</u>: Weathered and inferred very low to low grading to low strength shale below V-bit refusal depths of between 0.55 mBGL and 3.8 mBGL. For the purpose of this report, rock below TC-bit refusal depths of 2.7 mBGL in BH101 and 3.2 mBGL in BH103 is assumed to be of medium strength shale with possible lower and / or higher strength bands. This should be confirmed / revised by further assessment, as necessary. Fill was not encountered during the drilling of the boreholes. However, it may be present in limited portions of the site, such as areas within and / or nearby previous / existing site developments.
Groundwater	Groundwater inflow was not encountered during drilling of the boreholes up to 4.0 mBGL. Ephemeral perched groundwater may be encountered in the soil profile and / or at the soil / rock interface originating from infiltration of surface water during prolonged or intense rainfall events. Should further information on permanent site groundwater levels, particularly across the zone of influence of dam and drainage depression, be required, additional investigation would need to be carried out (i.e. installation of groundwater monitoring wells).

3 Salinity Assessment

3.1 Documented Salinity Risk Potential

The 1:100,000 *Salinity Potential in Western Sydney Map* (DIPNR, 2002) maps the site in an area of moderate salinity potential with high salinity potential along surface drainage lines, e.g. creeks and at the lower slopes in Wianamatta shales (Figure 2, Attachment A).

3.2 Broad Scale Salinity Processes

In producing the Salinity Potential Map, the Western Sydney Regional Organisation of Councils (WSROC) developed a number of alternative models of processes by which salinity may occur in Western Sydney (WSROC, 2003, pgs. 16 to 20).

Table 3 presents a list of key broad scale salinity processes likely to impact the site, including summarised descriptions of each process.

3.3 Signs of Potential Saline Soils at the site

No obvious signs of possible saline conditions were observed at the site; for example:

- No water marks or salt crystals were observed on the ground surface.
- Site surface drainage appeared generally good.
- No evidence of concentrated surface erosion was observed.
- Grass appeared healthy and uninhabited.

3.4 Assessed Salinity Risk Potential

In Table 3, the broad scale salinity processes have been assessed in terms of likelihood of occurring at the site, considering the proposed development, site observations and investigation findings.

Table 3: Potential for broad scale salinity processes at the site.

Key salinity process	Description	Potential at subject site
Localised concentration of salinity	Localised concentration of salts due to relatively high evaporation rates. Usually associated with waterlogged soil and poor drainage. Exacerbated by increased water use and/ or blocking of surface and subsurface water flow associated with urban development.	Moderate to High – No evidence of localised salt concentration and waterlogged soil and poor drainage observed. However, site dam, drainage depression, irrigation of gardens as well as dams nearby the site may have influenced site soil salinity.
Shale soil landscapes	In poorly drained duplex (texture contrast) soils, shallow subsurface water flows laterally across a clayey upper B-Horizon with salt usually accumulating in the clayey subsoil. Salt concentrations may increase where subsurface water accumulates and evaporates, e.g. on lower slopes or natural and constructed flats in mid-slope. Exacerbated by subsoils exposure through deep cutting, by installing buildings into the B-horizon and by impeding subsurface water flows. Highly dispersive, erodible and poorly draining sodic soils due to salinity.	Moderate to High – The site is underlain by low permeable clays, overlying shale. Water accumulation and evaporation of perched water in the existing dam and drainage depression on site as well as nearby dams may have resulted in salt accumulation in clays.
Deep groundwater salinity	Brackish or saline groundwater rises to a level where, through capillary action in the soil, the water with dissolved salts reaches the ground surface and evaporates, resulting in localised salt concentration. Groundwater rises are typically caused by increased water infiltration, e.g. above average rainfall, vegetation loss, irrigation, increased water use in urban areas, construction of surface pits. Exacerbated by buildings or infrastructure intercepting the zone of groundwater level fluctuation.	Low – Groundwater was not encountered in boreholes to 4.0 mBGL. The proposed development is not expected to intercept or raise groundwater levels. Proposed structures are to be constructed with appropriate drainage measures installed.
Deeply weathered soil landscape	High salt loads with high sulphate levels related to un-mapped deeply weathered soil landscapes beneath fluvial gravel, sand and clay. Usually in mid-slope or on hilltops affected by perched saline groundwater.	Moderate – No evidence of deeply weathered soils beneath alluvial soils was observed. Encountered soils on the site are residual. Deeper weathering is likely to be present within / nearby existing drainage depression and dam. Perched saline groundwater may have influenced site soil salinity.

3.5 Laboratory Testing

3.5.1 Overview

The chemical testing (Electrical Conductivity (EC), pH and soluble SO_4) was carried out for salinity classification and to assess an exposure classification for design of buried concrete structures. Sampling was targeted to achieve a representative coverage of site conditions in line with assessed subsurface profiles and the limited investigation scope.

3.5.2 Results – Salinity Classification

Laboratory test results for salinity classification are summarised in Table 4.

Table 4: Salinity test results.

Sample ID [1]	Material	$EC_{(1:5)}$ (dS/m)	EC_e (dS/m) [2]	Salinity Classification [3]
6439/BH101/0.1/S/1	SILT	0.100	1.00	Non-saline
6439/BH101/0.3/S/1	Silty CLAY	0.044	0.308	Non-saline
6439/BH102/0.1/S/1	Clayey SILT	0.089	0.801	Non-saline
6439/BH102/0.3/S/1	Silty CLAY	0.061	0.427	Non-saline
6439/BH102/0.8/S/1	Silty CLAY	0.180	1.260	Non-saline
6439/BH103/0.1/S/1	Clayey SILT	0.088	0.792	Non-saline
6439/BH103/0.5/S/1	Silty CLAY	0.200	1.400	Non-saline
6439/BH105/0.1/S/1	Clayey SILT	0.082	0.738	Non-saline
6439/BH105/0.5/S/1	Silty CLAY	0.084	0.588	Non-saline
6439/BH105/1.5/S/1	Silty CLAY	0.290	1.740	Non-saline
6439/BH106/0.1/S/1	Clayey SILT	0.056	0.504	Non-saline
6439/BH106/0.6/S/1	Silty CLAY	0.042	0.294	Non-saline
6439/BH107/0.1/S/1	SILT	0.051	0.510	Non-saline
6439/BH107/0.5/S/1	Silty CLAY	0.030	0.210	Non-saline
6439/BH107/1.0/S/1	Silty CLAY	0.059	0.413	Non-saline

Sample ID [1]	Material	$EC_{(1:5)}$ (dS/m)	EC_e (dS/m) [2]	Salinity Classification [3]
6439/BH108/0.1/S/1	SILT	0.022	0.220	Non-saline
6439/BH108/0.5/S/1	Silty CLAY	0.091	0.637	Non-saline
6439/BH108/1.3/S/1	CLAY	0.430	2.580	Slightly saline
6439/BH109/0.1/S/1	SILT	0.120	1.200	Non-saline
6439/BH109/0.5/S/1	Silty CLAY	0.026	0.182	Non-saline
6439/BH110/0.1/S/1	Clayey SILT	0.086	0.774	Non-saline
6439/BH110/0.5/S/1	Silty CLAY	0.028	0.196	Non-saline
6439/BH110/1.0/S/1	CLAY	0.028	0.168	Non-saline
6439/BH110/2.0/S/1	CLAY	0.057	0.342	Non-saline

Notes:

1 Project#/Borehole#/Depth (mBGL).

2 Based on EC to EC_e multiplication factors from Table 6.1 in DLWC (2002).

3 Based on Table 6.2 of DLWC (2002) where EC_e <2 dS/m = non-saline, EC_e of 2-4 dS/m = slightly saline, EC_e of 4-8 dS/m = moderately saline, EC_e of 8-16 dS/m = very saline and EC_e of >16 dS/m = highly saline.

Results indicate that residual silty clay at the tested locations can generally be categorised as non-saline. Only one sample returned slightly saline results (i.e. clay sample from 1 m depth in BH108).

3.5.3 Results – Exposure Classification

Sulphate and pH test results for exposure classification are summarised in Table 5.

Table 5: Exposure classification test results.

Sample ID[1]	EC_e (dS/m) [2]	pH	Sulphate (SO_4) (mg/kg)	Exposure Classification [3]
6439/BH101/0.1/S/1	1.00	6.1	25	A1
6439/BH101/0.3/S/1	0.308	5.9	20	A1
6439/BH102/0.1/S/1	0.801	5.6	22	A1
6439/BH102/0.3/S/1	0.427	5.7	40	A1
6439/BH102/0.8/S/1	1.260	6.7	70	A1

Sample ID[1]	EC_e (dS/m) [2]	pH	Sulphate (SO_4) (mg/kg)	Exposure Classification [3]
6439/BH103/0.1/S/1	0.792	7.6	< 10	A1
6439/BH103/0.5/S/1	1.400	6.2	190	A1
6439/BH105/0.1/S/1	0.738	6.4	30	A1
6439/BH105/0.5/S/1	0.588	6.4	120	A1
6439/BH105/1.5/S/1	1.740	5.2	< 10	A2
6439/BH106/0.1/S/1	0.504	7.6	10	A1
6439/BH106/0.6/S/1	0.294	7.9	< 10	A1
6439/BH107/0.1/S/1	0.510	5.9	22	A1
6439/BH107/0.5/S/1	0.210	5.8	37	A1
6439/BH107/1.0/S/1	0.413	5.8	69	A1
6439/BH108/0.1/S/1	0.220	5.9	10	A1
6439/BH108/0.5/S/1	0.637	5.6	61	A1
6439/BH108/1.3/S/1	2.580	5.4	< 10	A2
6439/BH109/0.1/S/1	1.200	5.8	22	A1
6439/BH109/0.5/S/1	0.182	6.1	10	A1
6439/BH110/0.1/S/1	0.774	6.7	39	A1
6439/BH110/0.5/S/1	0.196	6.7	20	A1
6439/BH110/1.0/S/1	0.168	6.6	26	A1
6439/BH110/2.0/S/1	0.342	5.8	63	A1

Notes:

1 Project#/Borehole#/Depth (mBGL).

2 From table 4.

3 Exposure classification for buried reinforced concrete based on Tables 4.8.1 and 4.8.2 of AS 3600 (2009).

The following exposure classifications should be adopted for preliminary design of buried concrete structures in accordance with AS3600 (2009):

- A1 for buried concrete structures up to 1 mBGL.
- A2 for buried concrete structures below 1 mBGL.

3.6 Conclusions and Recommendations

We conclude and recommend the following:

- o Subsurface materials at tested locations are generally categorised as non-saline with the exception of one sample in BH108, which is slightly saline. No specified saline soil management strategies are considered to be required.
- o Exposure classifications of 'A1' and 'A2' should be adopted for preliminary design of buried concrete structures up to 1.0 mBGL and below 1.0 mBGL, respectively, in accordance with AS3600 (2009).
- o Further assessment should be carried out, including laboratory testing, to confirm characterisation of site salinity conditions, particularly in proposed development areas, and assess potential ensuing implications following consideration of final development details.

4 Geotechnical Assessment

4.1 Soil Reactivity

A summary of soil reactivity test results is presented in Table 6.

Table 6: Summary of laboratory Soil Reactivity test results.

BH ID / Depth	Material	Atterberg Limits (%)				Plasticity Classification	Potential Volume Change [2]
		LL[1]	PL[1]	PI[1]	LS[1]		
BH103/0.5	Silty CLAY	55	18	37	11.5	High	Medium
BH108/0.5	Silty CLAY	46	16	30	13.5	Medium	Medium
BH108/1.3	CLAY	60	18	42	17.5	High	Medium

Notes:

1. LL = Liquid limit, PL= Plastic limit, PI=Plasticity index, LS = Linear shrinkage
2. Based on Hazelton and Murphy, 2016.

Laboratory test results indicate that the tested soil samples are generally of medium and high plasticity and will likely experience moderate ground movement due to soil moisture changes.

4.2 Preliminary Soil and Rock Strength Properties

Preliminary soil and rock strength properties, estimated from field test results in conjunction with borehole derived soil / rock profile data, as well as engineering assumptions, are summarised in Table 7.

Table 7: Preliminary material properties.

Layer	Material	$\gamma_{in-situ}$ [1] (kN/m³)	UCS [2] (MPa)	Ø' [3] (deg)	Cu [4] (kPa)	E' [5] (MPa)
TOPSOIL	SILT / Clayey SILT	15-17	NA [6]	NA [6]	NA [6]	NA [6]
RESIDUAL:	Silty CLAY / CLAY (stiff to very stiff)	17-18	0.1-0.2	NA [6]	50-100	10-20
RESIDUAL:	Silty CLAY / CLAY (hard)	19	0.4	NA [6]	200	30-40
WEATHERED ROCK:	SHALE (very low to low strength)	22	0.5 – 1.0	28	NA [6]	50-100
	SHALE (low strength)	23	1.0 – 3.0	28	NA [6]	100-300
	SHALE (medium strength)	23	3.0 – 10.0	32	NA [6]	300-500

Notes:

1. Material in-situ unit weight, based on visual assessment (±10 %).
2. Expected range of unconfined compressive strength of intact material.
3. Effective internal friction angle (±2 ˚) estimate, assuming drained conditions; may be dependent on rock defect conditions.
4. Undrained shear strength (±5 kPa) estimate assuming normally consolidated clay.
5. Expected range of effective elastic modulus (±10 %).
6. Not applicable.

4.3 Risk of Slope Instability

Evidence of extensive subsidence or recent gross slope instability was not observed on site. We consider the risk to property and loss of life by potential slope instability, such as landslide or soil creep, to be very low subject to the recommendations in this report and adoption of relevant engineering standards and guidelines. A detailed slope risk assessment in accordance with Australian Geomechanics Society's Landslide Risk Management Guidelines (2007) was not undertaken.

The proposed excavations may extend into the zone of influence of neighbouring properties, structures and / or other infrastructure to the north east and south east. The zone of influence is defined as an area of soil / rock, supporting features in question, below a nominal angle of between 30° and 45° for soils and 45° for rock, extending down and away from the base of the feature.

Recommendations presented in this report are provided to mitigate risks associated with potential excavation instability during construction

4.4 Geotechnical Recommendations

The following recommendations are provided for the proposed development. Further general geotechnical recommendations are provided in Attachment E.

1. Excavation: Proposed basement excavations will encounter residual soils over weathered rock. In light of this, we recommend the following excavation equipment:

   - Soils and low strength rock: These should be readily excavated using conventional earthmoving equipment. A 'toothed' bucket or a ripping tyne (or similar) may be required to excavate rock.

   - Medium and higher strength rock: Hydraulic earthmoving equipment with rock hammer attachment or dozer fitted with a ripping tyne.

2. Excavation Support: Excavations in soils and extremely low to low strength rock must be temporarily and permanently battered back / shored / retained to maintain excavation stability and limit potential adverse impacts on neighbouring properties or other infrastructure located within the zone of influence of proposed excavations. Medium and / or higher strength rock, where encountered, may remain unsupported subject to confirmation on site by a geotechnical engineer. Appropriate support and / or excavation methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer.

   If there is sufficient room to remain outside the zone of influence of existing structures / infrastructure, excavations in soils and weathered rock may be temporarily battered back at:

   - 1V:2H for soil and 1V:1.5H for rock, or

   - 1V:1H, if covered with an appropriate protection facing, e.g. by soil nails and shotcrete.

   Provided batters are subject to inspection and approval by a geotechnical engineer on site and followed by construction of permanent retaining structures.

   Excavations in soils and weathered rock may be permanently battered back at grades of no greater than 1V:3H.

Temporary shoring may include cantilevered or anchored soldier pile walls with shotcrete infill panels. For shoring inside the zone of influence of adjoining properties / infrastructure, closer spaced soldier piles may need to be adopted.

Retaining wall design may adopt active (k_a), at rest (k_0) and passive (k_p) earth pressure coefficients of 0.4, 0.55 and 2.5, respectively, for soils and 0.33, 0.50 and 3.0, respectively, for weathered rock.

Tieback anchors must not be installed across site boundaries unless written confirmation of acceptance is obtained from neighbouring property / asset owners.

Where installation of anchors is not possible, retaining walls may be supported by a suitable bracing system (i.e. wales and rakers).

Temporary walls may be designed to provide long term retention, with lateral restraint provided by building floor and roof slabs.

Retaining wall design should consider additional surcharge loading from live loads, new structures, construction equipment, backfill compaction and static water pressures unless subsoil drainage is provided behind retaining walls.

3. Rock Support: Unstable rock wedges as a result of presence of clay seams, weakly cemented (extremely weathered) seams, steeply dipping joints and other rock defects may have an adverse effect on unsupported rock face stability and construction safety. Geotechnical mapping of the excavation should be conducted in 1.5 m height increments to identify such features and allow early mitigation of risks of rock movement, such as by installation of rock bolts and / or sprayed concrete surfacing.

   If full height retaining walls are to be constructed for long term stability, then such measures are not likely to be required, however temporary measures may be required to stabilise excavations during construction.

   Rock bolts and sprayed concrete support should be specified in terms of performance requirements and installed / placed by contractors experienced in ground anchor technology and on advisement by an experienced geotechnical engineer. Rock support should not extend beyond property boundaries unless approval has been granted by relevant property owners or stakeholders. The actual amount of stabilisation which will be required cannot be quantified at this stage and can only be determined at the time of construction. MA can complete the necessary mapping and provide advice on support requirements.

4. Footings and Foundations: Allowable end bearing capacities for design of shallow and deepened footings are presented in Table 8, Section 4.5. Individual pad footings and all footings within building footprints should not span the interface between different foundation materials. Alternatively, inclusion of movement joints may mitigate impacts of differential movements.

   All foundation excavations should be inspected by a geotechnical engineer to confirm encountered conditions satisfy design assumptions.

5. Earthworks: All earthworks, including filling of dam, should be carried out in accordance with AS3798 (2007) and BCC's requirements following removal of topsoil and other unsuitable materials, such as uncontrolled fill and soft or saturated soils. Removal of any silt deposits from an existing dam and drainage depression should be carried out under the guidance of a geotechnical engineer.

   A qualified geotechnical engineer should inspect the condition of the exposed material to assess suitability of the prepared surface as foundation for footings or fill placement. This shall include proof rolling of exposed materials in accordance with Clause 5.5 of AS3798 (2007). If soft spots are identified, these shall be treated until conditions are assessed by the geotechnical engineer to be suitable. This may include one of the following methods:

   - Removal and replacement with granular fill approved by the geotechnical engineer.

   - In-situ stabilisation with cement / lime or similar binding agent to a depth of at least 300 mm. Use of this method and extent will depend on the condition of material to be stabilised.

   Should re-use of excavated materials be considered for site filling, we recommend limiting material re-use to general fill areas, given the medium to high plasticity of the clays. Where site-won material is used as structural fill, stringent placement specifications are to be developed to ensure adequate compaction, including moisture conditioning of material and controls to limit over compaction. We recommend the use of low plasticity clay or granular material as structural fill.

Imported fill shall:

- Exclude any unsuitable materials as outlined in Clause 4.3 of AS3798 (2007), such as material containing toxic substances or soluble compounds.

- Imported fill shall contain < 20 % by mass of particles greater than 37.5 mm after field compaction in accordance with Clause 5.2 of AS3798 (2007).

Fill material shall be placed in horizontal layers of generally not more than 300 mm loose thickness and with a mixture of materials as uniform as possible. Layer thickness shall be appropriate for the compaction plant adopted and may be varied by the supervising engineer.

For areas likely to be subjected to a loading of up to 20 kPa, fill material shall be moisture conditioned and compacted to a minimum density index (DI) of 75% or density ratio (DR) of 98% at a standard compactive effort within -3% and +1% of optimum moisture content (OMC). For areas loaded to greater than 20 kPa, such as under pavements trafficked by heavy vehicles, the upper 300 mm of subgrade material shall be compacted to a DI of 80% or DR of 100% at a standard compactive effort within -3% and +1% of OMC.

For general fill areas (e.g. landscaped areas and playing fields), fill should be compacted to a DI of 70% or DR of 95% at a standard compactive effort within -3% and +1% of OMC.

6. <u>Drainage requirements</u>: Groundwater inflow, if encountered during excavation, is expected to be limited and manageable by sump and pump methods. Appropriate drainage measures should be provided behind the retaining walls to divert ephemeral seepage water away from structures and discharge into council approved discharge points downslope.

   For soldier pile walls, strip drains with a non-woven geotextile filter fabric should be installed behind the shotcrete to dissipate the pore pressure build up behind the walls.

7. <u>Site Classification</u>: A preliminary site classification of 'H1' should be adopted for design of lightly loaded shallow footings, in accordance with AS 2870 (2011), subject to the recommendations presented in this report and CSIRO guidelines (CSIRO BTF 18, 2003). A preliminary site classification of 'P' should

be adopted, where footings are likely to be impacted by the presence of uncontrolled fill or soft foundation material, by cutting and filling of > 0.4 m thickness or by environments that could lead to exceptional moisture condition variations within foundation material, such as areas impacted by dam and drainage depression.

8. Trafficability and Construction Assess: Trafficability across exposed soil/subgrade materials is expected to be adequate in dry weather for most construction plant such as conventional rubber tyre plant, four-wheel drive plant and track mounted plant.

   During wet weather, trafficability of all heavy machinery on exposed soil/sub-grade materials, particularly residual clay / silty clay, may be reduced. Provision for site grading, temporary open drains or toe/crest drains is suggested to collect any overland flow, prevent water ponding and hence minimise potential for any further soil/sub-grade softening or erosion, and to help improve trafficability. The use of granular fill or aggregate for temporary construction roads may be necessary to allow works during and immediately following wet weather.

4.5 Allowable Bearing Capacities

Table 8 presents allowable bearing capacities that may be adopted for design of shallow and deepened footings.

Table 8: Allowable bearing capacities.

Unit	Shallow Footings	Piles [1]	
	ABC [2, 4]	ABC [2, 4]	ASF [3, 4]
RESIDUAL: Silty CLAY / CLAY (stiff to very stiff) / 'Engineered' FILL [5]	100	NA [8]	5
RESIDUAL: Silty CLAY / CLAY (hard)	250	NA [8]	20
WEATHERED ROCK: SHALE (very low to low strength)	350	700	60
WEATHERED ROCK: SHALE (low strength)	500	1000	150
WEATHERED ROCK: SHALE (medium strength)	1000	1500	250

Notes:

1. Assuming bored cast in-situ pile.
2. Allowable end bearing capacity (kPa) for footings embedded at least 0.3 m for lightly loaded footings, and piles embedded at least 0.5 m or 1 pile diameter, whichever is greater, into design material type subject to confirmation on site by a geotechnical engineer of inferred foundation conditions.
3. Allowable skin friction (kPa) below 1 m depth for bored pile in compression, assuming intimate contact between pile and foundation material. For up lift resistance, we recommend reducing ASF by 50% and checking against 'piston' and 'cone' pull-out mechanisms in accordance with AS2159 (2009).
4. ABC and ASF are given with estimated factors of safety of 3 and 2 respectively, generally adopted in geotechnical practice to limit settlement to an acceptable level for conventional building structures (< 1% of minimum footing width).
5. Refer to earthworks in Section 4.4.

5 Proposed Additional Works

5.1 Works at Construction Certificate

We recommend the following additional geotechnical assessments are carried out to develop the final design and prior to construction:

1. If higher end bearing pressures are required or to gain better understanding of rock conditions, carry out rock coring and point load testing of collected rock samples to assess rock strength.
2. Further salinity testing to confirm / revise preliminary salinity and exposure classifications and to delineate salinity conditions across soil profiles in development areas, if required, following consideration of final development details.
3. Review of the final design by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.

5.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 9).

Table 9: Recommended inspection / monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall/anchoring) installations to assess need for additional support requirements.	Daily / As required [2]	Builder / MA [1]
Monitor groundwater seepage from excavation faces, if encountered, to assess stability of exposed materials, suitability of proposed drainage and additional drainage requirements.	When encountered	Builder / MA [1]
Inspect exposed material at foundation / subgrade level to verify suitability as foundation / lateral support / subgrade.	Prior to reinforcement set-up and concrete placement or fill placement	MA [1]
Quality Assurance of earthworks	During earthworks	NATA laboratory with MA audit and supervision
Monitor sedimentation downslope of excavated areas.	During and after rainfall events	Builder

Scope of Works	Frequency/Duration	Who to Complete
Monitor sediment and erosion control structures to assess adequacy and for removal of built up spoil.	After rainfall events	Builder

Notes:

1. MA = Martens and Associates engineer
2. MA inspection frequency to be determined based on initial inspection findings in line with construction program.

6 References

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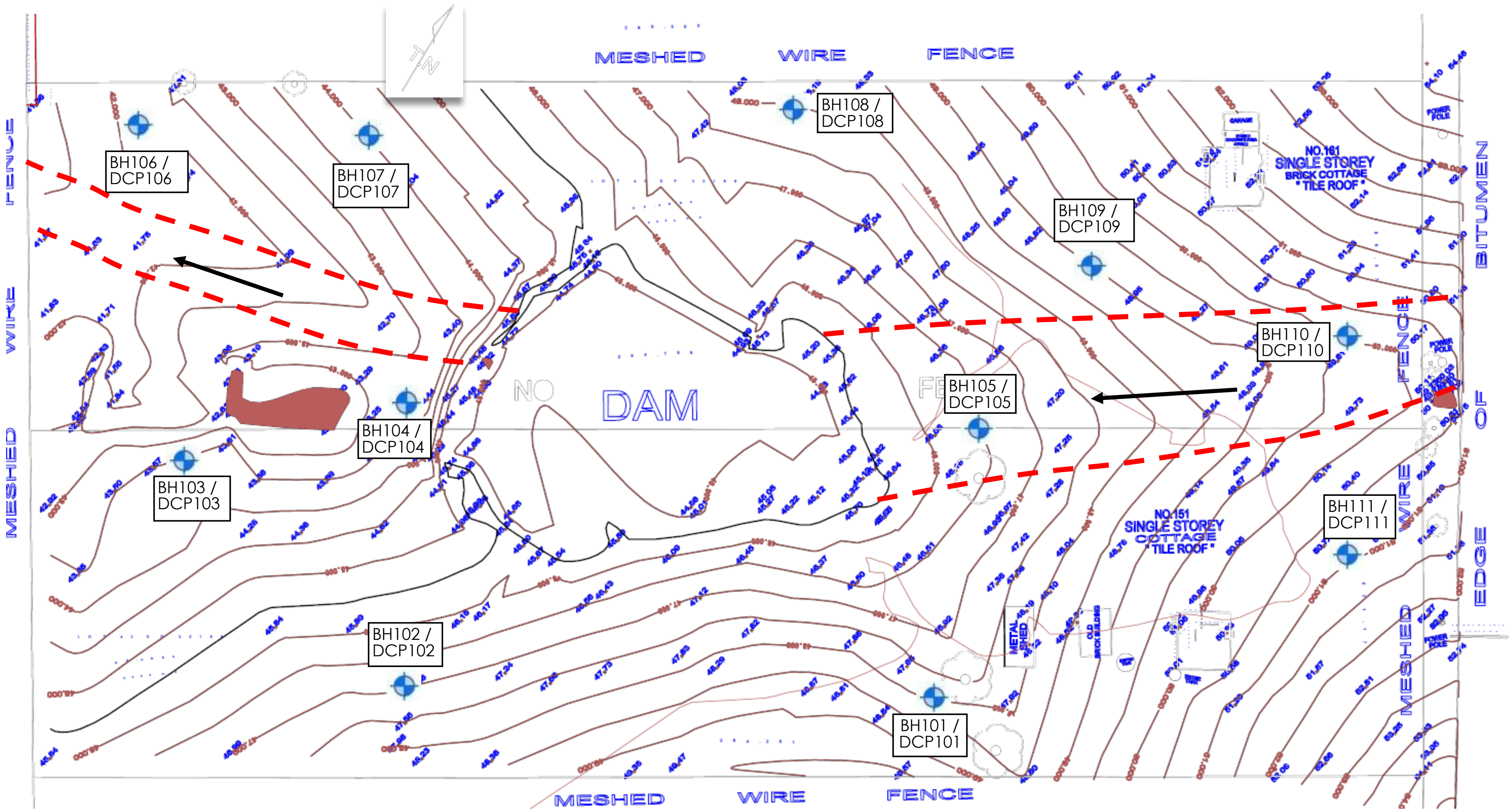
7 Attachment A – Figures



Key:

Approximate borehole and DCP test location

Indicative path of drainage depression

Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	HN	**GEOTECHNICAL SITE TESTING PLAN** **151 and 161 Tallawong Road, Rouse Hill, NSW** (Source: CSS, 2013)	Drawing: **FIGURE 1**
Approved:	RE		
Date:	22.05.2019		
	NA		File No: P1806439JR01V02

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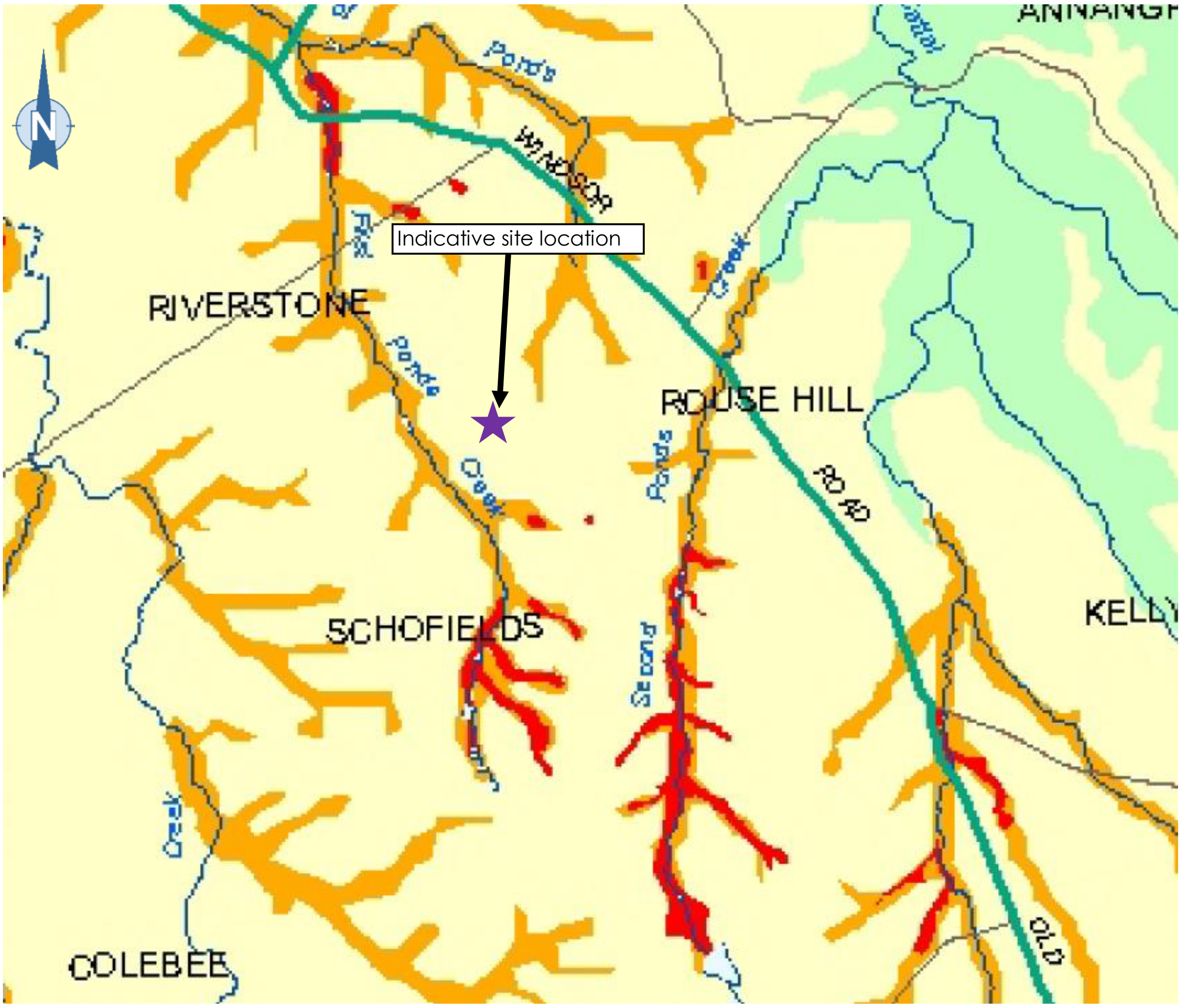

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MAPPING CATEGORY	ASSOCIATED SOIL LANDSCAPES	LANDFORM - GEOLOGY
KNOWN SALINITY Areas where there is a known occurrence of saline soil, or where air photo interpretation and field observations have confirmed more than one of these: a - scalding b - salt efflorescence c - vegetation dieback d - salt tolerant plant species e - waterlogging A high relative wetness index occurs in these areas.	* Salinity outbreaks occur in Blacktown (bt), Luddenham (lu) and Richmond (ri) Soil Landscapes - common at breaks of slope, lower slopes and drainage lines. * Berkshire Park (bp) and Upper Castlereagh (up) Soil Landscapes have localised salinity due to the impermeable nature of the clay parent material. * South Creek (sc), Monkey Creek (mk), Freemans Reach (fr) and Theresa Park (tp) Soil Landscapes have common saline outbreaks due to high run-on and low local relief. * Soils in the above landscapes have high clay content in subsoils and are imperfectly to poorly drained.	* Break of slope, lower slope and drainage lines of Wianamatta Shales (Rwb, Rwa and Rwm). * Localised salinity also occurs at the geological boundary between Tertiary Gravels (Tl, Tr) and underlaying Wianamatta Shales (Rwb, Rwa/ Quaternary Alluvials (Qpd, Qpa, Qpl, Qal). * Localised salinity occurs in Quaternary Alluvium (Qal, Qpn. Qpd) which underlies many of the drainage systems and wetland margins.
HIGH SALINITY POTENTIAL Areas where soil, geology, topography and groundwater conditions predispose a site to salinity. These conditions are similar to areas of known salinity (see above). These areas are most common in lower slopes and drainage systems where water accumulation is high (ie. high relative wetness index).	* Soil Landscapes include Birrong (bi), Blacktown (bt) Berkshire Park (bp), Freemans Reach (fr), South Creek(sc0, Theresa Park (tp), Richmond (ri) and Luddenham (lu). Drainage systems and convergent slopes are areas of highest risk. * Soils in these landscapes have high clay content in the subsoils, low permeability and high run-on. * Soil profiles may display signs of high salt concentrations at depth (i.e. >0.5m).	* Salinity is most likely to occur in lower slopes, foot-slopes, floodplains and creek lines on Quaternary Sediments (Qal, Qpn, Qpd, Qpc, Qpp, Qha)/Wianamatta Shales (Rwb, Rwm, Rwa) where run-on is high, resulting in seasonally high water tables and soil saturation.
MODERATE SALINITY POTENTIAL Areas on Wianamatta Group Shales and Tertiary Alluvial Terraces. Scattered areas of scalding and indicator vegetation have been noted but no concentrations have been mapped. Saline areas may occur in this zone, which have not yet been identified or may occur if risk factors change adversely.	* Areas of Agnes Banks (ab), Berkshire Park (bp), Blacktown (bt), Luddenham (lu) and Lucas Heights (lh). * Steeper areas with moderate to high local relief and well drained subsoils such as Picton (pn), West Pennant Hills (wp) and Glenorie (gn) are at a lower risk of developing salinity. * Soils are moderate to well-drained due to their elevated position in the landscape.	* Hill-slopes and hill-crests on Wianamatta Shales (Rwb, Rwm, Rwa). * Raised abandoned alluvial terraces and drainage lines on Quaternary Alluvium (Qal, Qpn, Qpd, Qpc, Qpp) from Richmond to Camden and east to Rookwood. Localised areas of elevated, well-drained Tertiary Gravels (Ta, Tl, Tr).
VERY LOW SALINITY POTENTIAL Areas where salinity processes do not operate or are of minor significance. Soils are rapidly drained and underlaying strata (Hawkesbury/Narrabeen Sandstone) are highly permeable, resulting in continual flushing and removal of salts in the landscape. No salinity has been observed in these areas and is not expected to occur.	* Rapidly drained soil landscapes with shallow soils include Warragamba (wb) and Hawkesbury (ha). * Gymea (gy) and Faulconbridge (fb) Soil Landscapes consist of highly permeable sands with well-drained subsoils. * Soils are well to rapidly drained. * Soils have high sand content.	* Occurring on Hawkesbury and Narrabeen Sandstone (Rh, Rno). * Groundwater is relatively fresh in these areas due to the sandstone's elevated position in the landscape and highly permeable nature, resulting in continuous flushing of the system (removal of any accumulated salts).

Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	HN	1:100,000 MAP OF SALINITY POTENTIAL IN WESTERN SYDNEY (Source: DIPNR, 2002)	Drawing No: FIGURE 2
Approved:	RE		
Date:	22.05.2019		
Scale:	Not to Scale		File No: P1806439JR01V02

8 Attachment B – Borehole Logs

9 Attachment C – DCP 'N' Counts

10 Attachment D – Laboratory Test Certificates

11 Attachment E – General Geotechnical Recommendations

12 Attachment F – Notes About This Report