

POSTED
20/10/06

**334-356 GREGORY STREET
SOUTH WEST ROCKS**

Macleay Valley Property Group
Liverpool, NSW

GEOTPMAQ00064AA-AB
19 October 2006

Geotechnical Assessment

19 October 2006

Macleay Valley Property Group
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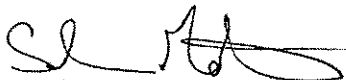
Attention: Peter Anderson

Dear Peter

**RE: 334 -356 GREGORY STREET, SOUTH WEST ROCKS
GEOTECHNICAL ASSESSMENT**

Please find enclosed a report describing the geotechnical assessment carried out on the above site
If you have any questions regarding this matter, please do not hesitate to contact the undersigned.

For and on behalf of Coffey Geotechnics Pty Ltd



Steve Morton

Regional Manager – Northern NSW

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1 INTRODUCTION

This report describes a site assessment carried out by Coffey Geotechnics Pty Ltd (Coffey) at 334 - 356 Gregory Street, South West Rocks, Lot 2 DP 581117.

The work was requested by Peter Anderson of the Macleay Valley Property Group in a facsimile dated 4 September 2006.

It is understood that a 44 lot subdivision is proposed for the site which is located on Gregory Street and is bounded by Arakoon Road and Cooper Street.

The scope of work for the geotechnical assessment included the following:

- Typical site foundation conditions i.e. soil reactivity and settlement issues and anticipated site classifications for residential building foundations;
- Foundation preparation prior to filling;
- Fill placement and compaction requirements;
- Pavement thickness requirements for the proposed sections of road works;
- Possible poor or wet subgrade conditions;
- Presence of acid sulfate soils;
- Presence of hard rock and resultant excavation conditions;
- Existing groundwater levels.
- Assessment of sub soil permeability and infiltration rates in areas proposed for stormwater detention basins.
- Suitability of proposed cut material for re-use as filling in the regrade areas

The following report presents the results of the field investigations and provides discussion and recommendations relevant to the above scope of work.

2 FIELDWORK

Field work was completed on 15 September 2006 and comprised of;

- Fourteen dynamic penetrometer tests.
- Fifteen test pits excavated with a mini-excavator to depths ranging from 1.3m to 2.2m with samples collected in sealed plastic bags for testing at the laboratory.
- Observation and mapping of relevant site features.

All field work was completed in the full time presence of an Engineering Geologist who located the test pits, collected the samples and produced field logs for the boreholes. The engineering logs are presented in Appendix A, together with explanation sheets defining the terms and symbols used in their preparation.

The locations of the investigations were measured from existing site features. A site sketch plan is shown in Figure 1.

3 SITE CONDITIONS

3.1 Surface Conditions

The site is situated in South West Rocks, off Gregory Street and bounded by Arakoon Road and Cooper Street, Lot 2 DP 581117 and occupies an 'L' shaped block of 3.98ha. An existing dwelling ("Rosalie") is located in the portion (Lot 1 DP581117) to the north east of the site.

Forty four lots with sizes ranging from 504m² to 1815m² are proposed for the site which will be completed in three stages. The proposed access road will enter the site from Coopers Street and then culminate in two cul de sacs in the east of the site.

The land has an overall slope to the west. Surface elevations are approximately 18m at the eastern side of the site and 3m at the western side of the site, towards Gregory Street. It is understood that the low lying areas of the site will be raised to 3.6m RL.

The site is situated at the toe to mid slope of a large hill, with ground slopes in the order of 11% on the east of the site which then flattens out to a 3% slope towards Gregory Street. A low ridge is situated in the north of the block where the existing dwelling is built. Large flat boulders of granite sub-crop are present on the surface of the small ridge which extends from the north for about 70m into the centre of the site. The western side of the ridge is relatively steep, approaching 17% slope but then quickly flattens out to a 3% slope.

Drainage of the site primarily occurs by way of overland flow and through the upper slopewash materials following the natural slope towards the west. The sandy soil in the west of the site allows any surface water to drain away quickly and there was no evidence of surface ponding, despite recent rainfall.

Reference to the Macksville & Nambucca 1:100,000 Soil Landscape Sheet (Eddie 2000) indicates that the lower slope and flatter areas of the site are Korogoro Soil Landscape, which consists of low coastal sand dunes. The higher hill slopes are the Big Smoky Erosional Soil Landscape, rolling hills of granite derived soils with slopes 10 - 25%.

The site is moderately vegetated on the higher slopes with scattered large eucalypts and blady grass as surface cover. The minor ridge in the north of the site has a thick cover of native shrubs and eucalypts, while the lower slopes and flat areas within the site are relatively open and vegetated with short grass and scattered small eucalypts.

A sewer pump station is situated on the western boundary of the property and an electricity transmission easement extends from the pump station to the south and the intersection with Arakoon Road. A Telstra cable is located in the north of the block, inside the property boundary, running parallel to Cooper Street.

3.2 Subsurface Conditions

The subsurface profiles encountered in the boreholes are presented on the appended engineering logs, with a summary of the typical profiles encountered shown in Table 1 and Table 2. Groundwater was encountered in several of the test pits.

TABLE 1 - SUMMARY OF SOIL TYPES

UNIT	SOIL TYPE	DESCRIPTION
1(a)	TOPSOIL/FILL	Silty SAND, brown/pale brown, fine to coarse grained, trace Clay, low plasticity, friable.
1(b)	TOPSOIL	SAND to Silty SAND, brown grey, fine to coarse grained Sand, trace fine to coarse Gravel, some roots, friable.
1(c)	TOPSOIL/COLLUVIAL	Silty SAND to Silty Gravelly SAND, brown grey, fine to coarse grained Sand, fine to coarse Gravel, some cobbles to 400mm, some roots, friable.
2	COLLUVIAL	Sandy Gravelly CLAY to Gravelly Clayey SAND, low plasticity, brown, fine to coarse grained Sand, fine to coarse grained subrounded Gravel, friable to hard.
3 (a)	AEOLIAN	SAND, fine to medium grained, white/pale grey/grey/dark grey, some orange mottling, with some organic fines, no plasticity, trace roots, loose to moderately dense.
3 (b)	AEOLIAN (INDURATED)	Indurated SAND, fine to medium grained, dark brown/black/dark grey, some orange mottling with organic fines, no plasticity, moderately dense to dense.
3(c)	AEOLIAN	SAND, fine to medium grained, white/pale grey/grey/dark grey, some orange mottling, with some organic fines, trace clay, low plasticity, moderately dense.
4 (a)	RESIDUAL	Clayey SAND, low plasticity, pale brown with orange mottles, fine to coarse grained Sand, with trace fine to medium grained sub rounded Gravel, firm to very stiff. Sand assumed to be of granitic origin.
4 (b)	RESIDUAL	Sandy CLAY, low to medium plasticity, pale brown / orange with red mottles, fine to coarse grained Sand, with trace fine to medium grained sub rounded Gravel, firm to very stiff.
4 (c)	RESIDUAL (EW Granite)	Sandy Gravelly CLAY, low plasticity, white with orange/red mottles, fine to coarse grained Sand, fine to medium grained sub-rounded Gravel, friable to very stiff. Extremely weathered granite

TABLE 2: SUMMARY OF SOIL TYPES INTERSECTED

LOCATION	UNIT 1 (a) Topsoil/Fill	UNIT 1 (b) Topsoil	UNIT 1 (c) Topsoil/ Colluvial	UNIT 2 Colluvial	UNIT 3 (a) Aeolian	UNIT 3 (b) Aeolian (Indurated)	UNIT 3 (c) Aeolian	UNIT 4 (a) Residual	UNIT 4 (b) Residual	UNIT 4 (c) Residual EW Granite
TP1		0 - 0.1			0.1 - 0.9	0.9 - 2.1				
TP2		0 - 0.1			0.1 - 1.0		1.0 - 1.9			
TP3		0 - 0.2			0.2 - 0.8	0.8 - 1.4	1.4 - 2.2			
TP4		0 - 0.1		0.1 - 0.6			0.6 - 1.8			
TP5		0 - 0.3					0.3 - 1.4		1.4 - 1.8	
TP6		0 - 0.2					0.2 - 1.0		1.0 - 1.6	1.6 - 1.8
TP7		0 - 0.2					0.2 - 1.0		1.0 - 1.7	1.7 - 1.9
TP8			0 - 0.2						0.4 - 1.1	1.1 - 1.6
TP9			0 - 0.4						0.5 - 1.5	1.5 - 1.7
TP10	0 - 0.2	0.2 - 0.6						0.6 - 2.2		
TP11		0 - 0.1			0.1 - 1.3					
TP12		0 - 0.1		0.1 - 0.6					0.6 - 1.8	
TP13		0 - 0.3		0.3 - 0.9					0.9 - 1.8	
TP14		0 - 0.3			0.3 - 1.7					
TP15			0 - 0.2					0.2 - 1.7		

Dynamic penetration testing was carried out adjacent to each testpit, results are summarised in Table 3.

**TABLE 3: DYNAMIC PENETRATION TEST RESULTS
(PERTH SAND HEAD)**

LOCATION	MATERIAL	DEPTH	DPT (average 150mm)
TP10	Unit 1b	0 – 0.30	4
	Unit 1b/ Unit 4a	0.30 - 0.90	1.5
DCP16	Unit 1b / Unit 4a	0 – 0.75	1.6
	Unit 4a		2.8
TP11	Unit 1b/Unit3a	0.0 – 1.05	1
	Unit 3a	1.05 - 1.5	3
DCP17	Unit 1b/Unit3a	0.0 – 0.6	2.2
TP12a	Unit 1b/ Unit 2	0 – 0.6	3
DCP18	Unit 1b / Unit 4a	0 – 1.05	3.4

Dynamic cone penetration testing was carried out at one location, results are summarised in Table 4.

**TABLE 4: DYNAMIC PENETRATION TEST RESULTS
(CLAY – CONE PENETROMETER)**

LOCATION	MATERIAL	DEPTH	DCP (average 150mm)	CBR Value (Estimate*)
TP12b	Unit 4b	0.6 - 1.05	9.7	13

*Based on Scala (1956)

4 LABORATORY TESTING

Samples retrieved during field work for the current geotechnical assessment were returned to our NATA registered Port Macquarie laboratory for testing. Samples selected for testing are discussed below.

Laboratory testing for road and pavement design comprised four (4) four-day soaked CBR tests on samples of materials expected to be encountered at subgrade level. The results of the laboratory testing are presented in Appendix B and are summarised in Table 5.

TABLE 5 - RESULTS OF CALIFORNIA BEARING RATIO (CBR) TESTING

TEST PIT LOCATION	SAMPLE DEPTH (m)	OPTIMUM MOISTURE CONTENT (OMC) (%)	CBR % (2.5mm / 5mm)	SWELL After 4 Day soak (%)
TP10	0.2 – 0.5	13.6	25 / 25	0.1
TP11	0.3 – 0.5	14.7	14 / 16	0
TP12	0.3 – 0.6	11.5	25 / 30	0
TP13	0.4 – 0.7	9.7	35 / 45	0

Laboratory testing for foundation design included the measurement of soil volume change over an extreme moisture content range on three undisturbed samples from four test pits (shrink/swell tests). The results of the laboratory testing are presented in Appendix B and are summarised in Table 6.

TABLE 6 - RESULTS OF SOIL VOLUME CHANGE TESTING

BOREHOLE LOCATION	SAMPLE DEPTH (m)	SWELL Under load (%)	SHRINKAGE Oven dried (%)	SHRINK/ SWELL INDEX (ISS)
TP6	1.0 – 1.4	0	2.26	1.26
TP7	1.1 – 1.5	0	1.7	0.94
TP8	0.5 – 0.9	0	1.5	0.83
TP9	0.6 – 1.0	0	2.6	1.44

Laboratory testing for acid sulfate soils are presented in Appendix B and are summarised in Table 7.

TABLE 7 - RESULTS OF ACID SULFATE SOIL SCREEN TESTING

TEST PIT LOCATION	SAMPLE DEPTH (m)	INITIAL pH (pH _F)	ACTUAL ACID SULFATE SOIL (pH<4)	REACTION	FINAL pH (pH _{FOX})	POTENTIAL ACID SULFATE SOIL (pH _{FOX} < 3)
TP4 (Aeolian)	0.3 – 0.5m	5.8	No	Slight	5.3	No

Laboratory testing for Emerson values are presented in Appendix B and are summarised in Table 8.

TABLE 8 - RESULTS OF EMERSON CLASS TESTING

TEST PIT LOCATION	SAMPLE DEPTH (m)	SOIL UNIT	EMERSON CLASS	SOIL CATEGORY (as per AS 1547:2000)
TP3	0.7 – 0.8m	Unit 3(a) – Aeolian Sand	n/a	1
TP4	1.3 – 1.4m	Unit 4(a) – Residual Clayey Sand	n/a	2
TP7	1.5 – 1.6m	Unit 4(b) – Residual Sandy Clay	5	4
TP9	0.5 – 0.6m	Unit 4(b) – Residual Sandy Clay	5	4

5 DISCUSSION AND RECOMMENDATIONS

5.1 General Site Assessment

The investigation indicates that the slopes of the granite hill have residual sandy clays underlying the topsoil and colluvial (slopewash) material. Granite subcrop was present on the minor ridge in the north of the site. In several locations on the hill slope a layer of Aeolian sands was present beneath the surface topsoil and colluvial material and overlying the residual sandy clays. These sands are remnant sand dune material deposited on the sheltered side of the hill. The low lying and flat areas of the site have loose aeolian sands present at surface, with an indurated sand (coffee rock) horizon present in two locations, approximately one metre from surface.

5.2 Site Preparation

It is understood that forty four residential allotments are proposed for the site and that development will include the construction of an access road that enters the subdivision from Cooper Street and culminates in two cul de sacs in the east of the site. The low lying section of the site that fronts Gregory Street will be filled to 3.6m AHD from an existing height of approximately 2.8m AHD.

Site preparation and earthworks for pavement and structure support should consist of:

- Prior to construction of pavements and placement of any fill, the proposed areas should be stripped to remove all unsuitable material, vegetation, topsoil, root affected or other potentially deleterious material, which should be spoiled or stockpiled for re-use as landscaping material only. The depth of unsuitable material requiring stripping as encountered by the test pits was typically between 0.1m to 0.2m
- Following stripping, all exposed subgrade materials should be proof rolled to identify any wet or excessively deflecting material. Any such areas should be over excavated and backfilled with an approved select material;
- Fill may then be placed over the stripped surface, and compacted in the manner outlined below. Should shallow footings be required the fill should be placed under Level 1 control (AS3798-1994).
- Following exposure of suitable natural material, or placement of an adequate select working platform, approved fill beneath roads and building areas should be placed in layers not exceeding 300mm loose thickness and compacted to a minimum density ratio of 95% Standard Compaction in accordance with AS1289.5.1.1 or equivalent. Clay subgrade fill should be placed and maintained within 2% of Optimum Moisture Content;
- The top 300mm of natural subgrade below pavements or the final 300mm of road subgrade replaced should be compacted to a minimum density ratio of 100% Standard Compaction or equivalent within the above stated moisture range;
- Fill beneath structures should be compacted to a minimum density ratio of 95% Standard Compaction within $\pm 2\%$ of OMC;
- All fill should be supported by properly designed and constructed retaining walls or else battered at 1V:2H or flatter and protected against erosion.
- Earthworks should be carried out in accordance with the recommendations outlined in AS3798-1994 *"Guidelines for Earthworks for Commercial and Residential Developments"*.

5.3 Excavation Conditions

Where excavation is required, it is anticipated that all site materials could be excavated by conventional dozer blade, excavator or backhoe bucket at least to the depths indicated on the appended test pit logs. The depths of sand and soil materials and depth to indurated sand (coffee rock) where encountered during fieldwork investigations are summarised in Table 2.

Large granite cobbles were present on the surface and in colluvial soils on the crest and slopes of the minor ridge in the north of the site to depths of 0.4m in the test pits, but they should still be excavatable by conventional dozer blade or excavator.

Density of indurated sands was variable and rippability may vary across the site. Thickness of the indurated sand layer ranged from 0.6m to 1.2m and depth from surface varied from 0.8m to 0.9m from surface. Sediments encountered beneath indurated sands were found to be medium dense sands. It is expected that indurated sands will be excavatable by backhoe, excavator bucket or ripping.

Slope stability in excavation batters within the upper sand material in the flat area of the site could be impacted by ground-water inflow, which generally occurs just above the indurated sand layer. This can be controlled by implementation of dewatering measures and /or retention measures such as sheet piling if required.

Maximum batters of 1.5H:1V are recommended for temporary batters in the upper loose sand zone. Permanent batters should not exceed 2.5H:1V without provision of retention measures. In the indurated sand layer, residual clayey sands or sandy clays, temporary batters of 1H:1V would be appropriate. Permanent batters in these materials sand should not exceed 2H:1V.

5.4 Material Reuse

The following comments are made regarding the suitability of the site materials for reuse in filled areas:

- Where site regrade is proposed, all existing topsoil, vegetation or other potentially deleterious material should be removed to spoil or stockpiled for reuse as landscaping materials only. Stripping is generally expected to be required to depths of about 0.1m to 0.2m.
- Stripping of colluvial materials (UNIT 1c and 2) may be required to some greater depth depending on moisture contents at the time of construction and presence of colluvial cobbles.
- Oversize cobbles will need to be removed prior to re-use of the materials as engineered fill.
- The underlying residual clayey sands, sandy clays and weathered rock (UNIT 4a, 4b and 4c) should be carefully stripped as necessary and stockpiled for reuse as general site fill. These materials should be placed and compacted close to the specifications below to minimise reactive soil movements.

5.5 Preliminary Site Classification

Based on the soil profiles encountered during the field investigations and the results of laboratory testing as summarised in Table 6, the proposed residential allotments on the hill slopes will probably be classified as Class 'S' in accordance with Australian Standard AS2870-1996. A characteristic free surface movement of 10mm to 20mm is estimated for those lots in their existing condition. The lots on the sand soils at the base of the hills would also be Class S on the condition that the fill is essentially non-reactive and is placed under Level 1 control (AS3798-1994).

The effects of changes to the soil profile by additional cutting and filling and the effects of past and future trees should be considered in selection for the design value for differential movement. Footings for the proposed development should be designed and constructed in accordance with engineering principles.

The classification presented above assumes that:

- All footings are founded in controlled fill (if applicable) or in natural soils below all uncontrolled fill, topsoil material and root zones, and other deleterious material such as wet or disturbed surface soil. Fill under slab panels should meet the requirements of AS2870-1996, in particular, the root zone must be removed prior to the placement of fill materials beneath slab floors;
- The performance expectations set out in AS2870-1996 are acceptable;
- Site maintenance complies with the provisions of CSIRO Sheet BTF 18, 'Foundation Maintenance and Footing Performance: A Homeowner's Guide', a copy of which is attached;
- All structural footings (including edge beams, internal beams and load support thickenings) are to be founded outside of or below all zones of influence resulting from existing or future service trenches. That is service trenches backfilled with uncontrolled fill do not extend below a line extending out and down at 45° from the ground surface at the edge of the building.

Adequate surface and stormwater drainage should be installed and maintained on the building site. All collected stormwater and roof run-off should be discharged into the inter-allotment or street stormwater disposal system.

Large granite cobbles and boulders were present on the surface and colluvial material on the minor ridge in the north of the site and care should be taken that foundations are not set on isolated cobbles or boulders.

5.6 Road Pavement Design

A new road is proposed as part of the development proposal. CBR and DCP testing was undertaken at selected test pits and at intervals along the proposed road which was marked by centreline pegs, see results in Tables 3, 6 & 7.

The following traffic design loadings have been adopted for roads within the subdivision, in accordance with reference to ARRB Special Report No. 41 and APRG Report No. 21.

TABLE 9: DESIGN TRAFFIC LOADINGS

PROPOSED ROAD	ROAD TYPE	DESIGN TRAFFIC LOADING (ESA's)
Local Access	No buses (40 years)	5×10^4
<i>ESA's – Equivalent Standard Axle Repetitions (refer Austroads 1992)</i>		

Flexible pavement designs have been provided for the design traffic loadings and subgrade conditions previously outlined in this report.

Flexible pavement design for the proposed subdivision roads have been based on Auspec No.1 Design Guidelines, with reference to Austroads, ARRB Special Report No.41 and APRG Report No.21. The recommended material, construction specification and pavement makeup are presented on the attached Pavement Thickness Design Summary Sheets.

With the soil profiles encountered on the hill slopes the soil moisture will tend to concentrate in the upper part of the residual sandy clay near the interface with overlying colluvial material, as encountered in TP13. It should therefore be anticipated that some moisture conditioning of the subgrade may be necessary prior to compaction and placement of pavement materials. Alternatively, the material could be replaced with a select subgrade as described below. Subgrade moisture contents and the required time period to prepare the subgrade is likely to be dependent on the prevailing weather conditions at the time of construction.

As an alternative the upper 300mm of subgrade may be over excavated and replaced with a minimum depth of 300mm of well graded select quality granular material with a minimum CBR of 15% and Plastic Limit (I_p) <12, as specified in the attached Pavement Thickness Design Summaries.

Based on the results of laboratory testing and our experience in the area, a design CBR value of 10% has been adopted for natural sand subgrade on the lower slopes of the hill and adjacent flat areas, while a design CBR value of 7% has been adopted for natural clay subgrade on the upper slopes of the hill.

If weathered rock subgrade materials (Granite) are encountered in the more elevated part of the site, the weathered rock should be ripped and re-compacted for a minimum depth of 300mm to break-up preferential draining paths and provide a dense homogenous surface on which to construct the pavement. Ripped and re-compacted weathered rock may be assumed to have a design CBR of 10%, however this should be confirmed by the geotechnical authority at the time of construction.

The pavement thickness designs attached are based on a minimum sub-base thickness of 150mm. This is recommended for constructability purposes. It is recommended that each construction length be boxed out to the minimum subgrade level required by the pavement thickness design. Prior to pavement construction, the exposed subgrade should be assessed by a geotechnical authority who can confirm the pavement thickness requirements for that section.

The attached Pavement Thickness Design Summary sheets assume the provision of adequate surface and subsurface drainage of the pavement and adjacent areas. It is recommended that subsoil drains be installed on both sides of road pavements.

5.7 Acid Sulfate Soils

The South West Rocks Acid Sulfate Soil Risk Map, see Figure 2, identifies the site as having "No known occurrence of acid sulfate soil materials". The elevation of the site at the lowest point is approximately 2.8m RL, while acid sulfate soils within the Macleay River estuary are generally located at or below 0m AHD. The Macksville & Nambucca 1:100,000 Soil Landscape Sheet (Eddie 2000) indicates that the lower slope and flatter areas of the site are Korogoro Soil Landscape, which consists of low coastal sand dunes and this was confirmed by the investigation.

An acid sulfate soil screening test was completed on a test pit sample as summarised in Table 7. The soil types encountered were not typical of acid sulfate soils and the sampling confirmed there were no acid sulfate soils present.

In conclusion the investigation did not identify any acid sulfate soil material present on the site and our preliminary acid sulfate soils assessment is that acid sulfate soils will not be disturbed and therefore an Acid Sulfate Soil Management Plan is not required based on our understanding of the proposed works.

5.8 Groundwater

Groundwater was present at following depths as summarised in Table 10;

TABLE 10: GROUNDWATER OBSERVATIONS

Location	Water Inflow Depth (m)	Material	Soil Category (1)	Indicative Permeability K_{sat} m/day (1)
TP2	1.1	Unit 3c - Aeolian Sand	1	>3
TP3	1.1	Unit 3a - Indurated Sand	n/a	n/a
TP4	1.0	Unit 3c - Aeolian Sand	2	1.4 – 3.0
TP12	1.6	Unit 4b – Residual Sandy Clay	4	0.12 – 0.5
TP13	0.6	Unit 4a – Residual Sand	2	1.4 – 3.0
TP15	1.6	Unit 4a – Residual Sand	2	1.4 - 3.0

(1) As per, Table 4.1.1, pg 59, AS/NZS 1547:2000 - On-Site Domestic Wastewater Management

The water inflow in TP13 was at a relatively shallow depth and was probably subsoil drainage occurring at the colluvial and residual soil interface following recent rains. It should be noted that TP13 was in the centre of a proposed cul de sac and poor subsoil drainage can have a negative impact on road base materials.

Sites TP3 and TP4 are understood to be in the vicinity of proposed stormwater detention basins. At both sites the ground water table was at a relatively shallow depth, approximately one metre from the surface. The water table in TP3, located near the existing sewer pump station, was present in a thin horizon of moderately dense indurated sand or 'coffee rock' at 0.8m from surface, overlain by loose sands. Perched water tables often occur above indurated sand horizons and this should be taken into account during storm water design.

Emerson testing results indicated that the clay soils present in TP7 and TP9, as summarised in Table 8, have low dispersion characteristics and as such the use of these clays in water retarding structures would be feasible.

6 CONSTRUCTION RISK

The extent of testing associated with this assessment is limited to discrete test pit locations and variations in ground conditions can occur between and away from such locations. If subsurface conditions encountered during construction differ from those given in this report further advice should be sought without delay.

Further advice on the uses and limitations of this report is presented in the attached document,
Important Information about your Coffey Report.

If you have any questions regarding this matter please do not hesitate to contact the undersigned at the
above office.

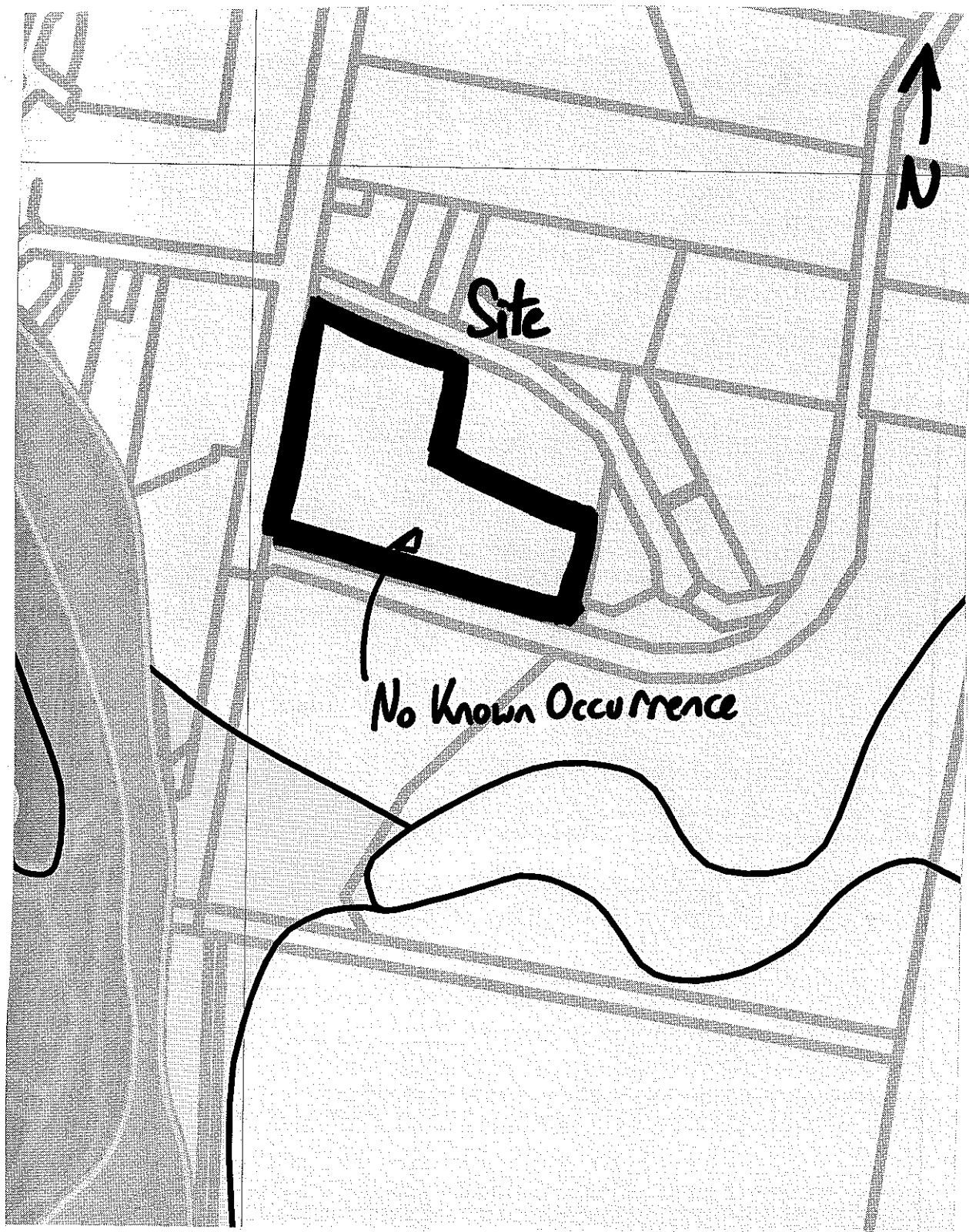
For and on behalf of Coffey Geotechnics Pty Ltd

A handwritten signature in black ink, appearing to read 'S Morton', with a stylized flourish at the end.


Steve Morton

Regional Manager – Northern NSW

Figures



From DLWC South West Rocks Acid Sulfate Soils Risk Map

drawn	TLM	 <p>coffey geotechnics SPECIALISTS MANAGING THE EARTH</p>	client:	Macleay Valley Property Group	
approved			project:	Gregory Street Subdivision	
date	17/10/2006			South West Rocks	
scale			title:	Acid Sulfate Soil Map	
original size			project no:	PMAQ00064AA	figure no: