Canomadine Creek Bridge

Geotechnical Investigation

Job No.: B21781

Submitted To:

Cabonne Council

PO Box 17

Molong NSW 2866

Attn: Wilson Wang

Geotechnical Investigation Report

Report No.: B21781

Cabonne Council – Canomadine Creek Bridge

REVISION CONTROL

Revision	Date	Details	Prepared By	Reviewed By
00	28/04/2023	Draft	S. Greene	D. Clarkson

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1 INTRODUCTION

At the request of Cabonne Council, Macquarie Geotechnical (MG) has carried out a Geotechnical Investigation for the proposed upgrade of Canomadine Creek Bridge on Canomadine Lane, Canowindra NSW.

The objective of the investigation is to provide a Geotechnical Investigation Report.

The comments and opinions expressed in this report are based on the ground conditions encountered during the site work including the results of tests carried out in the field and in the laboratory. However, there may be special conditions prevailing on the site which have not been disclosed by this investigation and which have not been taken into account by this report.

2 SCOPE OF INVESTIGATION

Undertake a desk study of the site to confirm the likely geological conditions of the site and to develop a geological model for the site.

Undertake Dial Before You Dig (DBYD) Search.

Mobilisation of one drill rig. Drilling, logging and sampling of two boreholes as per Table 1 below with rock coring at each borehole. In-situ testing comprised of Standard Penetration Testing (SPT) at 1.50m intervals in each borehole and Pocket Penetrometer (PP) tests on SPT split spoon samples.

Table 1: Borehole Scope

Hole ID	Eastings	Northings	Elevation RL (m)	Depth (m)
BH01	666461.0	6290944.9	355.0	13.34
BH02	666459.9	6290957.0	355.0	12.32

Samples were taken at selected intervals and at every change of strata to allow for laboratory testing at our NATA accredited laboratory in Sydney, NSW. Testing comprised of the following:

- 4No. Atterberg Limits & Linear Shrinkage Tests
- 2No. Soil Chemical Properties
- UCS Rock

2.1 Site Description

The site is located on Canomadine Lane at the bridge over Canomadine Creek, approximately 10.2km southwest of Cargo, NSW and 10.7km northeast of Canowindra, NSW.

Figure 1: Site Location

2.2 Desk Study

A desk study was undertaken using readily available geological and geotechnical information and included the following:

- NSW Seamless Geology Map
- NSW Department of Primary Industries – Groundwater Bore Data.
- NSW Government SEED
- Google Earth

2.3 Regional Geology

The Geological map sheet extract is shown in Figure 2 below:

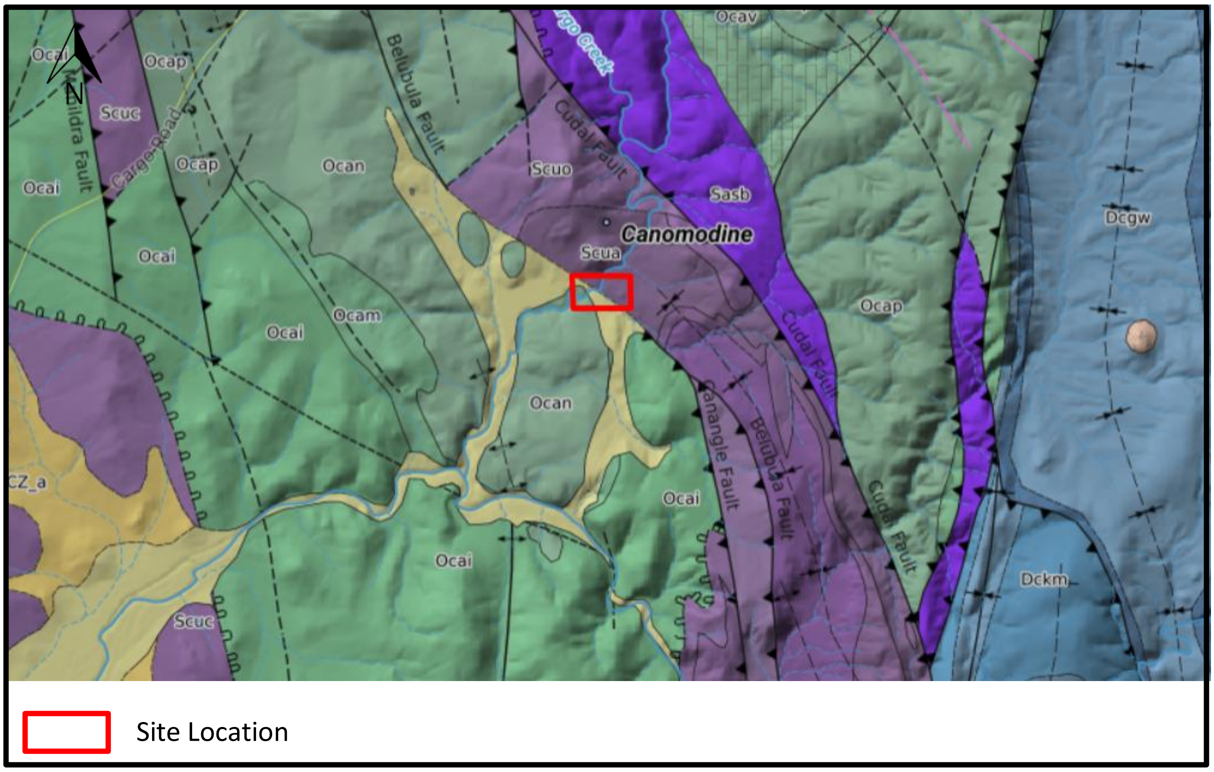

Figure 2: NSW Seamless Geological Map Sheet Extract

With reference to the NSW Seamless Geological map sheet extract, the site is underlain by the following:

Table 2: Summary of Geology

Geological Symbol	Group	Lithology
Q_af	Alluvial floodplain deposits	Silt, very fine to medium grained lithic to quartz rich sand, clay.
Scua	Avoca Valley Shale	Green and red-brown shale, coarse grained garnetiferous sandstone.
Ocan	Canomodine Limestone	Thick-bedded to massive, wackerstone, limey mudstone, minor shale and tuff.

2.3.1 Groundwater Bores

There were no records of groundwater bores located within close proximity to the site.

2.3.2 Acid Sulphate Maps

Reference is made to the NSW Government Central Resource for Sharing and Enabling Environmental Data in NSW (SEED) of Australian Acid Sulphate Soils and presented in Figure 3 below:

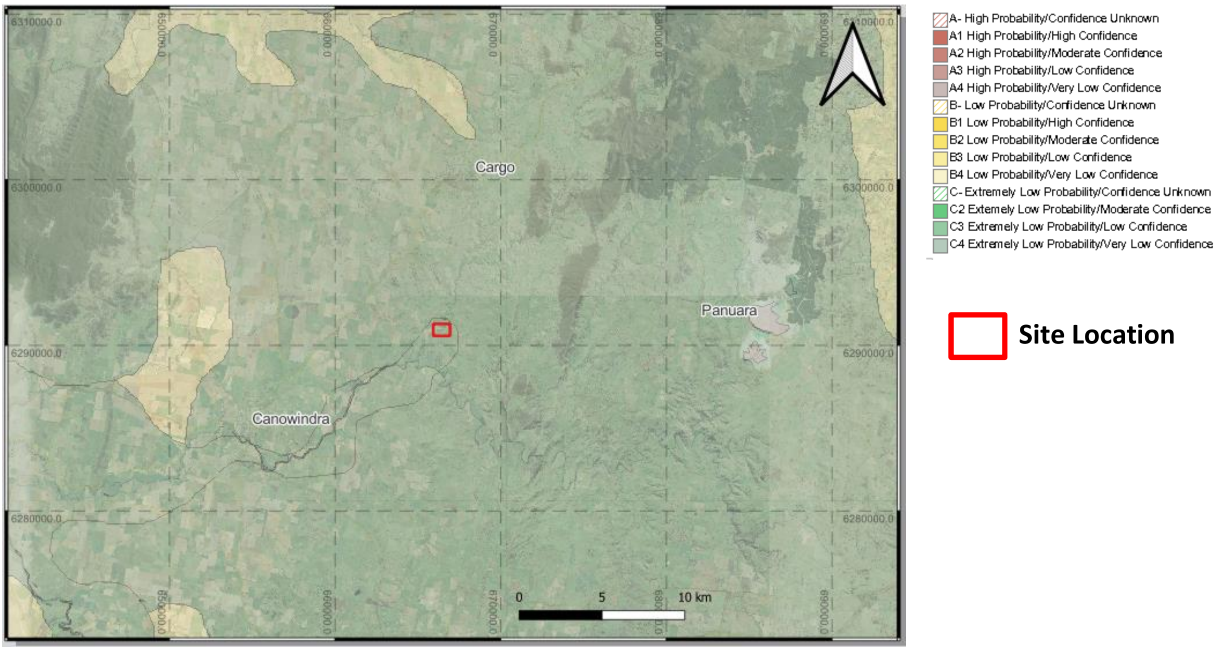

Figure 3: Acid Sulphate Risk Map

The acid sulphate map indicates an extremely low probability of acid sulphate soils within the site.

2.3.3 Topography

The topography of the site is moderately dipping to the north, approximately perpendicular to Mitchell Creek, from 353m to 367m above sea level. Canadomine Creek is a meandering tributary which flows into Belubula River to the south-west.

2.4 Fieldwork

Fieldwork was undertaken on the 3rd April to 5th April 2023 by a team of Drillers and Engineering Geologist from our Bathurst and Sydney offices. The fieldwork was undertaken in accordance with our proposal and AS1726 (2017) Geotechnical Site Investigation.

2.4.1 Service Location

Macquarie Geotechnical obtained underground services and utility plans through 'Before You Dig (BYD)' services.

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2.4.2 Survey

The test locations were surveyed using a handheld GPS with co-ordinates recorded in MGA Zone 55 format.

2.4.3 Boreholes

The boreholes were drilled at locations nominated by Macquarie Geotechnical and are summarised in Figure 4.

Figure 4: Borehole Location Plan

A track mounted Hanjin D&B 8D rig was used to drill two (2) boreholes to depths of up to 13.34m. Drilling comprised of 115mm diameter solid flight auger and HQ3 coring. In-situ testing comprised of Standard Penetration Testing (SPT) at 1.50m intervals in each borehole and Pocket Penetrometer (PP) tests on SPT split spoon samples.

The boreholes were backfilled with arising's and reinstated on completion.

The borehole logs and photographs are presented in Appendix C.

2.5 Sampling

The sampling was undertaken in accordance with AS1289 1.2.1 and based on that defined in the proposal and considered the engineering requirements of the investigation and the nature of the materials encountered.

2.6 In-Situ Testing

In-situ testing as specified by our proposal was carried out in the exploratory holes in accordance with the techniques outlined in the relevant Australian Standards and Macquarie Geotechnical Quality procedures. The results are presented on the relevant exploratory hole logs in Appendix C.

2.6.1 Standard Penetration Testing

Standard Penetration Tests (SPT) were carried out in the boreholes with techniques outlined in AS1289 6.3.1 in order to determine the relative density and consistency of the strata encountered. The SPT "N" value (number of blows per 300mm penetration) or the blow count/penetration were recorded for each test.

2.7 Laboratory Testing

The samples were returned to Macquarie Geotechnical NATA accredited laboratory at Sydney for further assessment and testing. A summary of the laboratory tests is provided in Table 3 below.

Table 3: Summary of Laboratory Tests

Hole ID	Depth (m)	Laboratory Test
BH01	1.00 – 1.50	AS1289 3.1.1 & 3.4.1 – Atterberg Limits and Linear Shrinkage
	2.50 – 3.00	AS1289 3.1.1 & 3.4.1 – Atterberg Limits and Linear Shrinkage
		APHA pH, SO4, Cl & EC
	8.67 – 8.87	UCS
BH02	0.50 – 1.50	AS1289 3.1.1 & 3.4.1 – Atterberg Limits and Linear Shrinkage
	2.00 – 3.50	AS1289 3.1.1 & 3.4.1 – Atterberg Limits and Linear Shrinkage
		APHA pH, SO4, Cl & EC
	11.24 – 11.44	UCS

3 EXISTING SUBSURFACE CONDITIONS

The subsurface conditions encountered in the boreholes are presented in detail in the attached borehole logs (refer Appendix C). The subsurface conditions encountered in all boreholes are broadly summarised in Table 4 below.

3.1 Exploratory Hole Summary

Table 4: Summary of Boreholes (BH01 and BH02)

-	BH01	BH02
Material Description	**Depth (m)**	
TOPSOIL	-	0.00 – 0.10
Gravelly SAND (FILL)	0.00 – 1.95	-
Sandy silty CLAY (ALLUVIAL)	1.95 – 6.50	0.10 – 1.00
Silty CLAY (ALLUVIAL)	-	1.00 – 3.00
Gravelly CLAY (RESIDUAL)	6.50 – 7.20	3.00 – 3.27
MUDSTONE (XW)	7.20 – 7.90	-
MUDSTONE (HW-MW)	-	3.27 – 8.27
MUDSTONE (MW)	7.90 – 8.45	-
MUDSTONE (SW)	8.45 – 13.34	8.27 – 12.32
Total Depth (m)	13.34 (LOI)	12.32 (LOI)
Groundwater Observation (m)	2.70	NFGWO

Note: Please refer to borehole logs in Appendix C for detailed descriptions.
MW – Moderately Weathered, SW – Slightly Weathered, F – Fresh.
LOI – Limit of Investigation.
NFGWO – No Free Groundwater Observed.

3.2 Groundwater

The comments on groundwater are based on the observations made at the time of the investigation. Groundwater was observed as a slow inflow at a depth of 2.70m within borehole BH01 during soil drilling. No observations of groundwater during BH02 works, this may have been masked by the use of rotary core drilling.

Seasonal variation in groundwater may be encountered and shall be considered as part of design process.

4 LABORATORY TEST RESULTS

The laboratory tests were carried out on the samples nominated by Macquarie Geotechnical. The summary of test results is shown in Tables 5 to 7 below.

Table 5: Laboratory Test Results – Classification

Hole ID	Depth (m)	Sample Description (USCS)	Atterberg Limits			Linear Shrinkage (%)
			LL (%)	PL (%)	PI (%)	
BH01	1.00 – 1.50	Clayey SAND*	25	17	8	4.0
BH01	2.50 – 3.00	Clayey SAND*	28	18	10	6.5
BH02	0.50 – 1.50	Silty CLAY*	32	19	13	9.5
BH02	2.00 – 2.50	Silty CLAY*	42	19	23	11.0

Note: * Visual description, USCS – Unified Soil Classification System.

Table 6: Laboratory Test Results – Soil Chemical Properties

Hole ID	Depth (m)	Sample Description*	Soil Chemical Properties (SCP)			
			pH	SO_4 (ppm)	Cl (ppm)	Electrical Conductivity (µS/cm)
BH01	2.50 – 3.00	Sandy silty CLAY*	7.9	20	<10	80
BH02	2.00 – 2.50	Silty CLAY*	8.0	<10	<10	74

Note: * Visual description; SO_4 – Sulphate, Cl – Chloride.

Table 7: Laboratory Test Results – Uniaxial Compressive Strength (MPa)

Hole ID	Depth (m)	Uniaxial Compressive Strength (MPa)
BH01	8.67 – 8.87	23.0
BH02	11.24 – 11.44	6.5

5 GEOTECHNICAL ASSESSMENT

5.1 Site Classification

The classification of a site involves a number of geotechnical factors such as depth of bedrock, the nature and extent of subsurface soils and any specific problems (slope stability, soft soils, filling, reactivity, etc).

In accordance with AS2870 2011 the proposed development site is classified as "Class M" and will have an anticipated surface movement (Ys) of 25 - 35 mm.

An appropriate footing system should be designed in accordance with the above code to accommodate these anticipated movements. The possibility of additional movements, due to abnormal moisture variations, should be minimised by proper "site management" procedures.

It should be noted that this assessment is based on site conditions being represented by the natural soil profile. Any change in conditions noted during development, including cut or fill should be referred to Macquarie Geotechnical for appropriate inspection and assessment.

The above classifications, based on AS2870 which relates to construction of residential dwellings, is not technically correct for the type of structures proposed and therefore it is given as a guide only with respect to soil reactivity.

5.2 Foundations

The investigation indicates that the ground conditions generally comprised of sequences of alluvial soil overlying weathered calcareous mudstone.

5.2.1 Geotechnical Design Parameters

Based on our investigation, and our experience in this region, we recommend the following geotechnical design parameters.

Table 8: Estimated Geotechnical Engineering Parameters

Depth (m)	Soil Description	Unit Weight (KN/m3)	Angle of Friction (degrees)		Cohesion (KPa)		Concrete to Soil Friction Angle δ (degrees)
			Drained ϕ'	Undrained ϕ	Drained c'	Undrained Cu	
Varying Depth	Clayey gravelly SAND (FILL)*	18	30	30	0	-	23
	Sandy silty CLAY – Soft to Firm	17	17	0	0	12	13
	Gravelly CLAY – Hard	20	32	32	0	200	25

Table 9: Bearing Pressure

Depth (m)	Soil Description	Allowable Bearing Pressure (KPa)	Ultimate Bearing Pressure (KPa)	Modulus of Subgrade Reaction (MN/m3)
Varying Depth	Clayey gravelly SAND (FILL)*	-	-	4
	Sandy silty CLAY – Soft to Firm	20	60	2
	Gravelly CLAY – Hard	340	1020	40

Table 10: Pile Design Parameters

<table>
<tr><th rowspan="2">Depth (m)</th><th rowspan="2">Soil Description</th><th rowspan="2">Ultimate End Bearing Pressure (KPa)</th><th rowspan="2">Serviceability End Bearing Pressure (KPa)</th><th rowspan="2">Ultimate Shaft Adhesion (KPa)</th><th colspan="2">Modulus of Subgrade Reaction (MN/m³)</th></tr>
<tr><th>Vertical</th><th>Horizontal</th></tr>
<tr><td rowspan="7">Varying Depth</td><td>Clayey gravelly SAND (FILL)*</td><td>-</td><td>-</td><td>-</td><td>4</td><td>8</td></tr>
<tr><td>Sandy silty CLAY – Soft to Firm</td><td>-</td><td>-</td><td>4</td><td>2</td><td>4</td></tr>
<tr><td>Gravelly CLAY – Hard</td><td>1800</td><td>600</td><td>60</td><td>70</td><td>140</td></tr>
<tr><td>Mudstone (EW)</td><td>3000</td><td>700</td><td>75</td><td>120</td><td>240</td></tr>
<tr><td>Mudstone (HW)</td><td>3000</td><td>1000</td><td>150</td><td>120</td><td>240</td></tr>
<tr><td>Mudstone (MW)</td><td>9000</td><td>3000</td><td>350</td><td>240</td><td>480</td></tr>
<tr><td>Mudstone (SW)</td><td>30000</td><td>6000</td><td>600</td><td>1200</td><td>2400</td></tr>
</table>

Note: EW – Extremely Weathered, HW – Highly Weathered, MW – Moderately Weathered, SW – Slightly Weathered.
* No skin friction support should be derived from the existing fill material.
Preliminary design parameters to be confirmed by a detailed design analysis.
Pile design parameters based on bored piles.
A bearing capacity factor Nc equal to 9 for clay can be used provided that the pile has been embedded at least to a depth of five diameters into the bearing stratum.
Socket roughness of R2 or better.
For strong rock, the pile carrying capacity should not be greater than the safe load on the material of the pile at the point of minimum cross section.

For foundations bearing on soil or rock, weaker soil or rock layers present below the base of the foundation within the zone of influence of the foundation should be taken into account in the design of the foundation.

5.3 Geotechnical Strength Reduction Factor (AS2159)

The geotechnical strength reduction factor for pile design is defined in the Piling Code. Selection of the geotechnical strength reduction factor (ϕ_g) is based on a series of individual risk ratings (IRR) which are weighted and lead to an average risk rating (ARR). The individual risk ratings and final value of (ϕ_g) depend on the following factors:

- Site: the type, quantity and quality of testing.
- Design: design methods and parameter selection.
- Installation: construction control and monitoring.
- Pile testing regime.
- Redundancy.

Without clear details about the pile type, design method, testing regime and other construction factors it is not possible to calculate the appropriate (ϕ_g) value. Assuming no pile testing, limited specialist geotechnical supervision during construction, and the limited/basic investigation and testing, an ϕ_g value of 0.48 is considered appropriate.

Nevertheless, with geotechnical supervision and pile integrity testing ϕ_g value can be increase to 0.52.

5.3.1 Foundation Settlements

For shallow foundations bearing on the alluvial or residual soils the total and differential settlements are expected to be within 25mm provided that the allowable bearing capacities are not exceeded. For deep foundations bearing on the underlying bedrock the total and differential settlements are expected to be within 25mm provided that the allowable bearing capacities are not exceeded.

5.3.2 Shallow Foundations

If it is proposed to use shallow foundations on fill material, then the existing ground should be excavated to remove any soft, organic or moisture affected materials. The exposed subgrade should then be compacted to a minimum dry density ratio of 98% relative to standard compaction at a moisture ratio of 60 - 90% of the optimum moisture content. The prepared subgrade shall then be proof rolled to identify any soft spots to remedy it. Fill material can then be placed and compacted to 98% relative to standard compaction at a moisture ratio of 60 - 90% of the optimum moisture content in maximum 250mm loose thickness layers up to design level. An allowable bearing capacity of 150kPa can be assumed for the compacted fill material.

5.3.3 Deep Foundations

Piles should be bored to found in the underlying mudstone. It is likely that a rock auger or coring bucket will be required for piles that are designed to be socketed into the underlying bedrock. Provision should be made for temporary casing of bored piles below groundwater level.

5.4 Excavation and Stability

Excavation of the alluvial and residual soils is expected to be straightforward using traditional excavation equipment. For temporary work conditions above groundwater level, benching in the cohesive soils or slope angles of 1V:1H in the non-cohesive soils is considered appropriate for the materials. For temporary work conditions below groundwater level excavation support will be required. For permanent conditions slope angles of 1V:2H is considered appropriate, subject to a slope stability assessment.

5.5 Aggressive Soils

We refer to Table 6.4.2 (c) Exposure Classification for Concrete Piles AS2159 – 2009 'Piling – Design and Installation'.

The soil condition is classified as 'Condition – A' and 'Condition – B'. The test results indicate very low levels of Sulphates (<10 - 20 ppm), Chlorides (<10 ppm) and a pH (7.9 – 8.0). Therefore, the soils at this site are classified as Mild due to the presence of groundwater.

6 CONCLUSION

The findings of our report were based on our fieldwork, in-situ testing, laboratory testing and technical assessment for this site.

We trust the foregoing is sufficient for your present purposes, and if you have any questions please contact the undersigned.

Sam Greene	**David Clarkson**	**John Boyle**
Engineering Geologist	Senior Geotechnical Engineer	Geotechnical Manager
BSc (Hons)	BEng MSc MIEAust	BSc (Hons) ME (Geotechnical) Affil MIEAust

Attached: Limitations of Geotechnical Site Investigation.
References: Australian Standard 1726 – 2017 Geotechnical Site Investigations

LIMITATIONS OF GEOTECHNICAL SITE INVESTIGATION

Scope of Services

This report has been prepared for the Client in accordance with the Services Engagement Form (SEF), between the Client and Macquarie Geotechnical.

Reliance on Data

Macquarie Geotechnical has relied upon data and other information provided by the Client and other individuals. Macquarie Geotechnical has not verified the accuracy or completeness of the data, except as otherwise stated in the report. Recommendations in the report are based on the data.

Macquarie Geotechnical will not be liable in relation to incorrect recommendations should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed.

Geotechnical Investigation

Findings of Geotechnical Investigations are based extensively on judgment and experience. Geotechnical reports are prepared to meet the specific needs of individual clients. This report was prepared expressly for the Client and expressly for the Clients purposes.

This report is based on a subsurface investigation, which was designed for project-specific factors. Unless further geotechnical advice is obtained this report cannot be applied to an adjacent site nor can it be used when the nature of any proposed development is changed.

Limitations of Site investigation

As a result of the limited number of sub-surface excavations or boreholes there is the possibility that variations may occur between test locations. The investigation undertaken is an estimate of the general profile of the subsurface conditions. The data derived from the investigation and laboratory testing are extrapolated across the site to form a geological model. This geological model infers the subsurface conditions and their likely behavior with regard to the proposed development.

The actual conditions at the site might differ from those inferred to exist.

No subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

Time Dependence

This report is based on conditions, which existed at the time of subsurface exploration. Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report.

Macquarie Geotechnical should be kept appraised of any such events, and should be consulted for further geotechnical advice if any changes are noted.

Avoid Misinterpretation

A geotechnical engineer or engineering geologist should be retained to work with other design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their plans and specifications relative to geotechnical issues.

No part of this report should be separated from the Final Report.

Sub-surface Logs

Sub-surface logs are developed by geoscientific professionals based upon their interpretation of field logs and laboratory evaluation of field samples. These logs should not under any circumstances be redrawn for inclusion in any drawings.

Geotechnical Involvement During Construction

During construction, excavation frequently exposes subsurface conditions. Geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed.

Report for Benefit of Client

The report has been prepared for the benefit of the Client and no other party. Other parties should not rely upon the report or the accuracy or completeness of any recommendations and should make their own enquiries and obtain independent advice in relation to such matters

Macquarie Geotechnical assumes no responsibility and will not be liable to any other person or organisations for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisations arising from matters dealt with or conclusions expressed in the report.

Other limitations

Macquarie Geotechnical will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.

Other Information

For further information reference should be made to "Guidelines for the Provision of Geotechnical Information in Construction Contracts" published by the Institution of Engineers Australia, 1987.