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PALING YARDS WIND FARM

APPENDIX 13

GEOTECHNICAL IMPACT ASSESSMENT

Report

Paling Yards Wind Farm

Geotechnical Exploration, Review and Advice

5/9/2011

Prepared for

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1

Introduction

1.1 General

URS Australia Pty Ltd (URS) has undertaken a geotechnical assessment for the proposed Paling Yards Wind Farm, NSW. The assessment was commissioned by Union Fenosa Wind Australia Pty Ltd (UFWA), and was carried out in general accordance with the URS fee proposal referenced 3091144/01/02, revision B, dated 2 March 2011.

The subject site is located on the western extent of the Great Diving Range, 60km south of Oberon, 60km north of Goulburn in NSW and about 140km west of Sydney. The surrounding area consists predominantly of large rural properties and National Park with the eastern edge of the site in the proximity of Kanangra Boyd National Park and Abercrombie National Park to the west and south. The site is situated in the Oberon Local Government Area (LGA).

The site is approximately 40km to the northeast of the existing Crookwell 1 Wind Farm and the approved Crookwell 2 Wind Farm.

The proposed Paling Yards Wind Farm Project will comprise up to 59 wind turbine generators (WTGs) associated with a new cable network, a temporary concrete batching plant, upgrading the local road infrastructure, new control buildings, a new electrical substation, and other associated infrastructure. The proposed WTGs have a maximum height of up to 175m to blade tip and up to 4.5MW capacity each.

The report presents findings on a number of geotechnical aspects relevant to the proposed wind farm. These include the following:

- Details of the investigation
- Subsurface conditions and geotechnical considerations for the proposed wind turbine sites.
- Groundwater issues
- Potential slope stability considerations
- Construction considerations
- Recommendations for future investigations

1.2 Safety on Site

Prior to the commencement of the geotechnical investigation, URS prepared a Safe Work Method Statement (SWMS) that included a Health, Environmental & Safety Plan (HESP)

Prior to conducting fieldwork, URS carried out a “Dial Before You Dig (DBYD)” services search for existing services at all turbine/test pit locations. In addition to DBYD, the proposed test pit locations were checked on site for any services that may not have picked up on DBYD plans by an experienced URS Geotechnical Engineer with cross reference from the land owners and signed off that all locations are clear of services.

Prior to commencing work, all personnel working on site were given a Health & safety talk and required to sign off an “induction register” ensuring that each person was aware of their responsibilities and safety procedures. A daily toolbox meeting was conducted at the start of the day, which covered all activities and risks associated with the day’s work.

1.3 Scope of Work

Preliminary geotechnical investigations were carried out between 11 April 2011 and 21 July 2011 to identify and characterise the main geologic units at the site. All the geotechnical investigation work was carried out by an experienced URS geotechnical engineer. The following works were carried out to characterise the soil and rock properties of the main geologic units across the site.

- A walk over inspection of the site and surroundings.
- Drilling of two (2) geotechnical boreholes up to a depth of 20m.
- Excavation of sixty (60) test pits.
- A total of sixty (60) Dynamic Cone Penetrometer (DCP) tests were carried out, ensuring a DCP test adjacent to each test pit
- Collection of representative soils samples for laboratory testing

The Test Pit and DCP locations were shown on Figure 1, Appendix A.

1.4 General Site Geology and Topography

1.4.1 Topography

The site is located on the western extent of the Great Diving Range, 60km south of the town of Oberon, 60km north of the city of Goulburn and comprises two separate land holdings totalling 3,900 hectares referred to as Mingary Park and Paling Yards. The majority of the site comprises farmland with farm houses and stock sheds present. The site is accessible via a network of unsealed farm roads and the existing Abercrombie road.

The site topography comprises plateau and hillcrest areas at an elevation of between 900m and 1065m surrounded by steeply sloping gullies and creek lines that flow to the Abercrombie River. The gently sloping plateau areas are generally cleared and used for grazing, while the more steeply sloping areas are generally uncleared and heavily vegetated.

1.4.2 Geology

Available geological information indicates that the plateau areas are underlain by Tertiary aged Volcanics which typically comprises residual clay, frequently with cobbles and boulders, overlying variably weathered basalt at relatively shallow depths. Tertiary aged alluvial deposits underlie the Tertiary Volcanics at depth, overlying Ordovician aged meta-siltstone basement.

Please see Figure 2, Appendix A for a site geological map.

Methodology

2.1 Test Pit Excavation

Test pit were excavated at each turbine location to provide an assessment of the likely subsurface materials and relevant geotechnical considerations. A total of sixty (60) test pits were excavated at/near along the proposed alignment of WTG across the site. The test pitting program was carried out between 11 April 2011 and 15 April 2011.

The test pits were excavated using a 5.5t small sized excavator which was operated by qualified personnel from Acclaimed Excavation Pty Ltd, fitted with an interchangeable 450mm wide toothed bucket. All test pits were terminated at effective refusal or targeted depth. Upon completion of test pit excavation, each test pit was made safe by backfilling with the excavated spoil and tamped with the excavator bucket.

The subsurface conditions encountered in the test pits, were logged and sampled by an experienced URS geotechnical engineer for visual assessment. The location of test pits are shown on Figure 1, Appendix A. The test pits were located using a handheld GPS unit to confirm the GPS co-ordinates provided by UFWA. The GPS co-ordinates of the test pit locations are recorded on the test pit logs. Test Pits TP1, TP10, TP11, and TP14 were offset from the proposed coordinates due to site accessibility issues.

Test Pit Logs and Photographs are attached in Appendix C together with notes regarding soil description and test methods.

2.2 Dynamic Cone Penetration (DCP) Testing

A total of sixty (60) Dynamic Cone Penetrometer (DCP) tests were performed along the proposed alignment of the WTG, ensuring a DCP test at/near each test pit location. The in-situ testing comprised the measurement of the consistency and in-situ strength of the subsurface materials to a steel rod driven into the ground by a dropped weight. The in-situ testing procedures are in accordance to AS 1289.F3.2. The equipment utilises a 9kg sliding weight with a drop height of 510mm and the rod is fitted with a conical tip. The test data are generally recorded as the number of blows (n) per 50mm of penetration. The test data are then processed by our in-house computer software.

DCP Logs are attached in Appendix D.

2.3 Borehole Drilling

The fieldwork for the geotechnical assessment included the drilling of two boreholes at WTG 9 and 38, as requested by UFWA. The selection of boreholes was based on UFWAs consultation with landowners, and it was perceived that these two locations may have significantly different subsurface conditions. The borehole drilling program was carried out between 18 July 2011 and 22 July 2011.

Drilling was carried out using a 2010 Model CME 55LC track mounted drilling rig which was operated by qualified personnel from Strategic Drilling Services Pty Ltd. The boreholes were drilled initially using a TC-bit attached to solid flight augers (150mm diameter) to refusal in bedrock, with standard penetration tests (SPTs) carried out in the soils at regular depth intervals (approximately 1.5m). The boreholes were subsequently cased then extended into the underlying bedrock to a depth of approximately 20m using NMLC diamond coring. Further details of the methods and procedures employed in the investigations are presented in Appendix B, Report Explanatory Notes.

The locations of the boreholes are shown in Figure 1, Appendix A. Borehole logs with core photographs are presented in Appendix E.

2.4 Electrical Resistivity Survey

The purpose of the Electrical Resistivity Survey (ERS) is to determine the electrical resistivity of the subsurface by means of ground measurements. The apparent ground resistivity is dependent on geological parameters such as mineral type, moisture content, porosity and degree of water saturation.

URS carried out an Electrical Resistivity Survey on the 18th of July 2011 at turbines WTG 9 and 38. The machine used for resistivity sounding was called an Automatic Resistivity System (ARES) made by GF instruments. To measure the resistivity of the subsurface soils at the site, a total of 40 stainless steel rods (in a straight line) with a spacing of 2m each were inserted to a depth of roughly 200mm into the ground. Upon completion of the set-up, ARES equipment estimated the electrical resistivity of the subsurface soils using Wenner Alpha, Schlumberger and Dipole-Dipole models.

The subsurface profile based on Wenner Alpha, Schlumberger and Dipole-Dipole models was estimated after processing the data collected at the site using software RES2DINV. The location of Electrical Resistivity testing was shown on Figures presented in Appendix G.

2.5 Laboratory Testing

Soil and rock testing were conducted on disturbed bulk soil and rock samples collected during the geotechnical field investigation. The results are summarised in the following section and attached in Appendix F.

Table 2-1 Lab Testing Schedule

Test	No. Tests
Moisture Content	20
Standard Compaction	10
California Bearing Ratio	10
Emerson Crumb	20
Soil thermal conductivity	10
Electrical Resistivity	6
Soil Aggressivity	10
Point Load Strength Index (Rock)	8

Geological Conditions

3.1 Test Pits Results

Based on the test pit investigations, two generalised soil profiles were inferred. Table 3-1 provides a summary of the Tertiary Volcanics encountered across the majority of the site. Table 3-2 provides a summary of Ordovician materials encountered across the site.

Table 3-1 Subsurface Conditions - Tertiary Volcanics Profile

Unit	Unit Description	Depth to Top of Unit (m)	Unit Thickness (m)
Tertiary Volcanics Profile	**Topsoil** : Silty SAND, fine grained, pale brown to dark brown, moist, medium dense to dense, few test pits encountered some gravel, cobble, and boulder basalt	0.0	0.2 to 0.4
	Residual Soils: Clayey SAND and Gravely SAND, fine grained, brown and pale brown, dry to moist, dense to very dense, with some fine to coarse grained sub-angular gravel, cobble, and boulder basalt or Sandy CLAY and CLAY, medium to high plasticity, brown, red, pale brown, and pale grey, dry to moist, friable/very stiff to hard, with some fine to coarse grained sub-angular gravel and cobble basalt, Residual	0.2 to 0.4 0.2 to 0.4	0.6 to 1.8 0.4 to 3.1
	Bedrock: BASALT, medium to high strength, distinctly to extremely weathered, grey, dark grey, and greenish grey, Bedrock	0.4 to 3.2	NOT PENETRATED

Table 3-1 is based on investigations TP4, TP12, TP15-TP45, TP47, and TP49-TP60. Variations to the above-generalised sequence were encountered in TP38, TP45, TP54 and TP60, where the Basalt bedrock stratum was deeper and not encountered within the investigation depths.

In-situ testing the Dynamic Cone Penetrometer (DCP) was carried out adjacent to each test pit location. The results of testing indicated that the strength of the subsurface residual soils profile to be of stiff to very stiff consistency, hence becoming hard with depth, underlain by weathered basalt bedrock.

3 Geological Conditions

Table 3-2 Subsurface Conditions - Ordovician Materials

Unit	Unit Description	Depth to Top of Unit (m)	Unit Thickness (m)
Ordovician Materials	**Topsoil:** Silty SAND, fine grained, pale brown, brown, and dark brown, moist, medium dense to dense, few test pits encountered some cobble basalt, Topsoil	0.0	0.2 to 0.3
	Residual Soil: Sandy CLAY, medium to high plasticity, brown, pale brown, orange, dry to moist, very stiff to hard, with a trace of fine to medium grained sub-rounded gravel basalt or Clayey SAND, fine grained, pale brown, dry to moist, dense to very dense, with a trace of fine to medium grained subrounded gravel basalt	0.2 to 0.3 0.2 to 0.8	0.6 to 1.4 0.2 to 0.6
	Bedrock: SILTSTONE, low to medium strength, distinctly to extremely weathered, pale grey and pale brown, Bedrock	0.2 to 1.7	NOT PENETRATED

Table 3-2 is based on investigations TP1-TP3, TP5-TP11, TP13-TP14, TP46, and TP48. The subject test pits are generally with relative lower elevation level and located closer to the Abercrombie River.

In-situ testing of the Dynamic Cone Penetrometer (DCP) was carried out adjacent to each test pit location. The results of testing indicated that the consistency of the subsurface residual soils varied from stiff to very stiff, underlain by weathered basalt bedrock.

3.2 Boreholes Results

Based on the findings of the geotechnical borehole drilling, two generalised profiles were inferred. Table 3-3 provides a summary of the Ordovician Aged Alluvial Deposits encountered in BH1 (WTG location 8), and Table 5 provides a summary of Tertiary Aged Volcanics encountered in BH2 (WTG location 38).

Table 3-3 Subsurface Conditions in BH1 - Ordovician Materials

Unit	Unit Description	Depth to Top of Unit (m)	Unit Thickness (m)
Ordovician Materials	**Topsoil:** Clayey SAND, fine grained, dark brown, with some crushed sandstone and gravel with organics	0	0.4
	Bedrock: SILTSTONE, low to high strength, distinctly to slightly weathered, with some extremely weathered zones, pale brown to brown, with some fine to coarse grained sand, with some medium to gravel size quartz , with some clay infilling joints	0.4	BH1 terminated at 20m, targeted depth reached, no further penetrated

Table 3-4 Subsurface Condition in BH2 - Teriary Volcanics Profile

Unit	Unit Description	Depth to Top of Unit (m)	Unit Thickness (m)
Tertiary Volcanics Profile	**Topsoil:** Silty SAND, fine grained, pale brown, with organics	0	0.3
	Residual Soil: Sandy Silty CLAY, medium to high plasticity, pale brown and brown, with a trace of gravel	0.3	5.1
	Bedrock: BASALT, medium to high strength, slightly weathered to fresh rock, with some extremely weathered zones grey, dark grey to grey, massive, with a trace of iron staining and clay infilling along joints	5.1	BH2 terminated at 19.72m, targeted depth reached, no further penetrated

3.3 Groundwater Conditions

Groundwater was not observed in the test pits or boreholes during drilling. It should be noted that these observations were made at the time of the field investigation and actual groundwater levels may fluctuate significantly in response to seasonal effects, regional rainfall, and other factors that are not related to this investigation.

Based on past experience it is anticipated that the fractured Basalt and the underlying Tertiary sediments are typically water bearing and can form perched water tables on weathered Ordovician basement. The regional water table in fractured Ordovician bedrock is anticipated to be at a considerable depth.

3.4 Materials Properties of Geotechnical Soil Units

The soil unit distribution within this study area generally comprises the Tertiary Volcanics profiles and the Ordovician materials. The units are summarised and described based on analysis of the study area using a number of methods including field observation, test pits, borehole data and topographic analysis.

The soil unit distribution is listed in Table 3-5. Observations from field test pits along with laboratory results is summarised for each geotechnical soil unit in the following sections.

Table 3-5 Soil Unit Distribution

Soil Unit Description	Test Location Number	Sample Depth Range (m) below ground level
Tertiary Volcanics	TP4, TP12, TP15-TP45, TP47, and TP49-TP60	0.4m to 1.7m
Ordovician Materials	TP1-TP3, TP5-TP11, TP13-TP14, TP46, and TP48	0.4m to 0.7m

3.4.1 Tertiary Volcanics Profiles

The Tertiary Volcanics profiles generally comprise residual soils and cover most of the valley floor areas within the subject site. Soils identified as residual soil unit in these areas are generally relatively shallow, typically less than 2.5m. However exceptions to this would be expected, for example test pit TP39 excavated in the central portion of the site, encountered 3.3m clay residual soils overlying basalt bedrock.

Residual soils observed at the site were predominantly high plasticity clays, with gravelly sandy clays usually encountered before underlying Basalt bedrock. Table 3-6 presents lab testing results for this unit.

Table 3-6 Tertiary Volcanics Results Summary

Properties	Moisture Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (t/m^3)	CBR Value (%)	Chloride (mg/kg)	Sulphate (mg/kg)	pH	Emerson Class Number
Max Value	40	41	1.75	10	56	47	7	6
Min Value	14.4	17	1.25	1.5	2.4	0.5	5.7	3
No. of tests	18	9	9	9	9	9	9	18
Average	26.11	27.56	1.53	4.75	13.81	13.81	6.4	5

3.4.2 Ordovician Materials

The Ordovician materials encountered within the study area comprise clay dominated soils, with exceptions such as sands and gravels. The clays soils were characteristically medium to high plasticity, brown-pale brown, and orange. The underlying siltstone bedrock is relative shallow, typically less than 1m. Table 3-7 presents lab testing results for this unit.

Table 3-7 Ordovician Aged Alluvial Deposits Results Summary

Properties	Moisture Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (t/m^3)	CBR Value (%)	Chloride (mg/kg)	Sulphate (mg/kg)	pH	Emerson Class Number
Max Value	26.1	34	1.67	2.5	22	6.4	7	5
Min Value	17.6	18	1.38	2	22	6.4	7	5
No. of tests	2	2	2	2	1	1	1	2
Average	21.85	26	1.53	2.25	22	6.4	7	5

Geotechnical Comments and Recommendations

4.1 Geotechnical Comments

4.1.1 Subsoil Class for Earthquake Design

In accordance with AS 1170.4 – 2007, site's specific class parameters are as follows:

- Hazard factor (Z) of <0.09
- Sub-soil class of B_e – Rock

4.1.2 Geomorphology, Tectonics and Fracturing

The site geomorphology comprises a dissected upland plateau at an elevation of between 900m and 1065m surrounded by steeply sloping gullies and creek lines that fall to the Abercrombie River. The plateau is covered by Tertiary Basaltic Volcanics that erupted onto a plateau formed in Ordovician Siltstones. Uplift occurred post Tertiary and has resulted in the weathering and erosion of both Basalt and Siltstone.

No major faults of shear zones cross the site and the boundaries between the rock units are erosional.

Both the Basalt and Siltstone are fractured on a regional scale, the Basalt due to cooling and the Siltstone due to folding and low grade metamorphism.

4.2 Geotechnical Recommendations

4.2.1 Bedrock Characteristics

Selected rock core samples recovered from boreholes were sent to a NATA accredited laboratory, SGS Australia Pty Ltd for Point Load Strength Index Testing. The point Load Strength indices of the rock cores and the estimated rock strength, in accordance with the Australian Standards (AS4133.4.1 2007), are summarised in the following Table 4-1.

Table 4-1 Bedrock Point Load Strength Index Summary

Sample ID	Sample Source (m)	Lithology	Standard Deviation Point Load Strength Index I_s50 (MPa)		Rock Strength
			Diametric	Axial	
BH1-1	5.67 to 5.75	Siltstone, slightly weathered, pale brown and pale grey	0.39	0.77	Medium
BH1-2	9.23 to 9.34	Siltstone, slightly weathered, pale brown and pale grey	0.58	N/A	Medium
BH1-3	12.79 to 13	Siltstone, slightly weathered, pale brown and pale grey	1.68	1.46	High
BH1-4	15.6 to 17	Siltstone, slightly weathered, pale brown and pale grey	0.41	0.88	Medium

4 Geotechnical Comments and Recommendations

Sample ID	Sample Source (m)	Lithology	Standard Deviation Point Load Strength Index I_s50 (MPa)		Rock Strength
			Diametric	Axial	
BH2-1	6.83 to 6.97	Basalt, fresh rock, dark grey to black	1.92	3.83	Medium to High
BH2-2	8.83 to 8.91	Claystone, extremely weathered, brown and red	0.18	0.21	Low
BH2-3	13.56 to 13.68	Basalt, distinctly weathered, grey to dark grey	0.6	0.69	Medium
BH2-4	18.68 to 18.8	Basalt, distinctly weathered, grey to dark grey	0.92	N/A	Medium

4.2.2 Wind Turbine Generators (WTGs) Foundation Design - General

The conventional WTGs foundations are reinforced concrete gravity footings founded 1.5m to 3m below the existing ground surface. The critical loading for this foundation system are lateral loads from a combination of wind and earthquake events. The footings are sized such that the maximum allowable bearing pressure is not exceeded on one side of the footing while the other side of the footing experiences uplift loads.

An alternative foundation system is to reduce the size of the footing and resist the uplift loads by installing anchors or piles below foundation level. As the footings are smaller, bearing pressures are greater, and this system is only suitable where sound rock extends from foundation level to the depth of the anchors.

Based on the current geotechnical investigation the potential foundation systems suitable for each WTG site has been summarised in Table 4-2:

Table 4-2 Potential Foundation Systems for WTGs

WTG	Test Pit	Founding Conditions	Potential Foundation System
1	TP-1 (50m offset)*	Basalt/Siltstone – Strength unknown	Anchored Footings/Gravity Footings
2	TP2	Siltstone- Low to medium strength	Gravity Footings
3	TP3	Siltstone- Low to medium strength	
4	TP4	Basalt – Medium to High Strength	Anchored Footings
5	TP5	Siltstone- Low to medium strength	Gravity Footings
6	TP6	Siltstone- Low to medium strength	
7	TP7	Siltstone- Low to medium strength	
8	TP8	Siltstone- Low to medium strength	
9	TP9	Siltstone- Low to medium strength	
10	TP10	Siltstone- Low to medium strength	
11	TP11	Siltstone- Low to medium strength	
12	TP12	Basalt – Medium to High Strength	Anchored Footings
13	TP13	Siltstone- Low to medium strength	Gravity Footings
14	TP14	Siltstone- Low to medium strength	

4 Geotechnical Comments and Recommendations

WTG	Test Pit	Founding Conditions	Potential Foundation System
15	TP15	Basalt – Medium to High Strength	
16	TP16	Basalt – Medium to High Strength	
17	TP17	Basalt – Medium to High Strength	
18	TP18	Basalt – Medium to High Strength	
19	TP19	Basalt – Medium to High Strength	
20	TP20	Basalt – Medium to High Strength	
21	TP21	Basalt – Medium to High Strength	
22	TP22	Basalt – Medium to High Strength	Anchored Footings
23	TP23	Basalt – Medium to High Strength	
24	TP24	Basalt – Medium to High Strength	
25	TP25	Basalt – Medium to High Strength	
26	TP26	Basalt – Medium to High Strength	
27	TP27	Basalt – Medium to High Strength	
28	TP28	Basalt – Medium to High Strength	
29	TP29	Basalt – Medium to High Strength	
30	TP30	Basalt – Medium to High Strength	
31	TP31	Basalt – Medium to High Strength	
32	TP32	Basalt – Medium to High Strength	
33	TP33	Basalt – Medium to High Strength	
34	TP34	Basalt – Medium to High Strength	Anchored Footings
35	TP35	Basalt – Medium to High Strength	
36	TP36	Basalt – Medium to High Strength	
37	TP37	Basalt – Medium to High Strength	
38	TP38	Clay –Soil depth 5m	Gravity Footing
39	TP39	Basalt – Medium to High Strength	
40	TP40	Basalt – Medium to High Strength	
41	TP41	Basalt – Medium to High Strength	Anchored Footings
42	TP42	Basalt – Medium to High Strength	
43	TP43	Basalt – Medium to High Strength	
44	TP44	Basalt – Medium to High Strength	
45	TP45	Gravelly Sand – Soil depth >2.0m	Gravity Footings
46	TP46	Siltstone- Low to medium strength	
47	TP47	Basalt – Medium to High Strength	Anchored Footings
48	TP48	Siltstone- Low to medium strength	Gravity Footing
49	TP49	Basalt – Medium to High Strength	
50	TP50	Basalt – Medium to High Strength	
51	TP51	Basalt – Medium to High Strength	Anchored Footings
52	TP52	Basalt – Medium to High Strength	
53	TP53	Basalt – Medium to High Strength	
54	TP54	Gravelly Sand – Soil depth >1.5m	Gravity Footing
55	TP55	Basalt – Medium to High Strength	Anchored Footings
56	TP56	Basalt – Medium to High Strength	

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WTG	Test Pit	Founding Conditions	Potential Foundation System
57	TP57	Basalt – Medium to High Strength	Anchored Footings
58	TP58	Basalt – Medium to High Strength	
59	TP59	Basalt – Medium to High Strength	
60	TP60	Sandy Clay –Soil depth >2.1m	Gravity Footing

*TP-1 was offset by 50m due to accessibility issues and foundation conditions at WTG1 cannot be assessed from current geotechnical investigations

It is not clear at this stage of the design process if anchored foundations represent a major cost saving over gravity foundations. It is recommended that a number of preliminary foundation designs for a range of tower heights be costed so that the most cost effective foundation system can be selected for each site and tower combination.

4.2.3 Wind Turbine Generators (WTGs) Foundation Design – Gravity Footings

Based on the current geotechnical investigation, distinctly to extremely weathered basalt and siltstone may be anticipated at the depth of about 1.5m to 3m. Gravity Footings may be designed based on the parameters given in Table 4-3:

Table 4-3 Foundation Design Parameters

Material	Allowable Bearing	Ultimate Bearing	Ult. Bond Stress
Medium Strength Siltstone or Basalt	1.0MPa	8.0MPa	500kPa
High Strength Basalt	3.5MPa	30MPa	2000kPa

It should be noted that at ultimate bearing capacity settlement values can exceed 5% of footing dimension and this needs to be taken into account in the design. Settlement values under allowable loading are not anticipated to exceed 1% of footing dimension.

It is possible that weaker materials (low strength rock) may be encountered locally within this depth range and all footings must be inspected by an experienced Geotechnical Engineer or Engineering Geologist to confirm appropriate founding materials and achievement of design socket lengths, that the recommended serviceability bearing pressures could be met and to ensure that all soft and wet materials have been removed from the foundation footprint prior to concrete placement.

4.2.4 Wind Turbine Generators (WTGs) Foundation Design – Anchored Footings

Anchored footing may be designed using the parameters for high strength Basalt in Table 4-3. The capacity of the anchors in uplift need to satisfy both the bond stress requirements and cone pull out assuming a 60 degree cone with its apex at the centre of the anchor bond zone. The impact of interfering cones may also need to be taken into account.

WTG sites with anchored footings require additional geotechnical investigation to confirm the anchor can be installed into sound rock. This generally comprises one bore within the foundation footprint to 1m below the maximum anchor depth.

4 Geotechnical Comments and Recommendations

4.2.5 Proposed Foundations for Turbines

Based on borehole drilling significantly different subsurface conditions were encountered at WTG38 in comparison to WTG9. URS understands that the preferred location for the substation is WTG38. However, recommendations on foundations at both the locations (WTG38 & WTG9) were provided in this section. The ground conditions at WTG38 and WTG9 are summarised in Table 4-4

Table 4-4 Ground Conditions At WTG38 & WTG9

Location	Test Pits	Bores	Subsurface Conditions
1	TP9	BH 1	Low to medium strength siltstone from shallow depth
2	TP38	BH 2	Stiff to very stiff clays over high strength Basalt at 5m depth

At Location 1, relevant infrastructure may generally be supported by shallow footings (pad or strip footings) founded in medium strength siltstone bedrock. The appropriate foundation parameters in Table 4-3 may be used for footing design.

At location 2 lightly loaded structures may be founded on Stiff Clays with an allowable bearing capacity of not less than 100kPa. For heavily loaded or settlement sensitive structures it is recommended that the loads be transferred to the high strength basalt bedrock using bored piles.

All footings must be inspected by an experienced Geotechnical Engineer or Engineering Geologist to confirm appropriate founding materials and achievement of recommended serviceability bearing pressures could be met and to ensure that all soft and wet materials have been removed from the foundation footprint prior to concrete placement.

With regards to shallow footings supported on the deep clay soils, it should be noted that such clays encountered in the study area are of high plasticity and are generally considered to have a high potential for expansion and swelling as a result of variation in moisture condition. The requirements of AS 2870 should be included in the design of shallow footings supported on the natural high plasticity clays.

4.2.6 Elastic Properties of Soils

Based on current geotechnical investigation, indicative preliminary values of geotechnical parameters that may be used for preliminary design purposes are provided in this section. The parameters estimated based on geotechnical investigations and our experience with similar materials are presented in Table 4-5 below.

Table 4-5 Geotechnical Design Parameters

Material	Undrained Shear Strength (kPa)	Elastic Modulus (MPa)	Friction Angle (Degree)	Bulk Density (kN/m^3)
Topsoil Silty Sand or Clayey Sand, medium dense	n/a	20 to 30	27 to 30	17 to 19
Residual Sandy Clay, Clayey Sand, very stiff to high, with gravel	150 to 250	25 to 50	n/a	20
Siltstone, low to medium strength	n/a	500	n/a	22

Material	Undrained Shear Strength (kPa)	Elastic Modulus (MPa)	Friction Angle (Degree)	Bulk Density (kN/m^3)
Basalt, medium to high strength	n/a	1000	n/a	24

The range of parameter in Table 4-5 reflects the variation and localised differences encountered at all the sixty test pit locations.

4.2.7 Soil Thermal Conductivity

Thermal resistivity testing was carried out on selected soil samples recovered from test pits by Chadwick T&T Pty Ltd. Summary of testing results are presented in Table 4-6. Full results are attached in Appendix F.

Table 4-6 Thermal Conductivity Testing Results

Sample ID	Sample Source (m)	Lithology	Moisture (%)	Compacted Density (t/m^3)	Thermal Conductivity* (W/mK)
TP8	0.5 – 0.8	Sandy Clay, brown and pale brown	27	1.582	0.76
TP15	0.5 – 0.8	Sandy Clay, pale grey and pale brown	29	1.546	0.68
TP17	0.4 – 0.7	Sandy Clay, brown and pale brown	32.3	1.392	0.75
TP21	0.4 – 0.7	Sandy Clay, brown	32.3	1.529	0.95
TP25	0.5 – 0.8	Sandy Clay, brown and red	19.2	1.947	2.51
TP30	0.5 – 0.8	Sandy Clay, brown and pale brown	17.1	1.6	0.55
TP39	0.4 – 0.7	Sandy Clay, brown and red	13.7	1.82	1.36
TP41	0.5 – 0.8	Sandy Clay, brown	31	1.642	0.68
TP48	0.4 – 0.7	Sandy Clay and Siltstone, pale brown and orange	No Result received**	No Result received**	No Result received**
TP57	0.4 – 0.7	Sandy Clay, brown	32.3	1.596	0.86

** The subjected samples were tested in 100% compaction standard at the received moisture content.*
*** No result was received on TP48 sample as siltstone component.*

4.2.8 Electrical Resistivity Survey

URS undertook a total of three resistivity surveys at each of the two proposed locations (near WTG 9 and 38). The purpose of this survey was to provide information about the existing ground resistivity for the design of the earthing grid at the proposed substation locations. The results and figures are available in Appendix G. These tests include the Wenner Alpha array which is reliable for determining

4 Geotechnical Comments and Recommendations

depth variations in 1-D earth, while Schlumberger Array is more sensitive to lateral variation in Earth and Dipole-Dipole array is reliable in estimating sensitivity to lateral variation at depth.

The first proposed substation location surveyed was at borehole 1 near WTG9. Due to the sloping area and out cropping rock in the way, the survey line had to be offset approximately 50 meters away from the borehole. The resistivity survey indicates areas of low resistivity within the first few meters of the ground subsurface. All the three tests indicate a consistent pocket of high resistivity near the north eastern region of the survey line (refer to figures in appendix G). The siltstone in this region is highly fractured, as a result water is able to seep through the voids and create pockets of low resistivity.

The second proposed substation location surveyed was at borehole 2 near WTG38. This site was relatively flat and the survey line was laid immediately adjacent to the borehole.

The electrical resistivity results at Borehole 2 are similar to the electrical resistivity results obtained at Borehole 1. In both locations areas of low resistivity exist within the first few meters of the strata.

At borehole 2 all three tests indicate a pocket of high resistive material around the borehole location.

The Wenner Alpha results of borehole 2 indicate a large continuous zone of low resistivity past a depth of approximately 2.5 meters. A possible explanation for this is the substantial amount of rain the area has received in the weeks leading up to our testing. Given that the first few meters of the strata is residual soil, the water would have soaked through the ground and settled on the top layers and the faults and defects of the basalt. This soaking of the ground could be a possible explanation for the anomalously low resistivity of the deeper strata.

The results of the electrical resistivity tests are presented in Table 4-7.

Table 4-7 Electrical Resistivity Results

Location	Description of Soil/Rock Layer	Lowest (Ohm.m)	Highest (Ohm.m)	Average (Ohm.m)	Anomaly (Ohm.m)
BH1	Siltstone and Sandstone, medium strength, distinctly weathered, slightly fractured	5.13	750	280	+ 15000
BH2	Sandy Clay and Silty Clay, medium plasticity	100	350	175	+ 2000

Construction Consideration

5.1 Excavation Conditions

Based on the subsurface conditions assessed from the test pits, excavations for access roads, construction platform and foundations for the proposed WTGs would likely encounter a variable thickness of sandy clay/clayey sand with some basalt cobble and boulder, weathered basalt and siltstone bedrock.

Excavations within soil materials may be carried out using tracked excavators or bulldozers. Some basalt boulders may be encountered when excavating within first few meters, which may require larger plant and some over excavation to remove.

Bulk excavation in the extremely to distinctly weathered basalt or siltstone may be generally carried out using large excavation plant such as a heavy bulldozer or a heavy hydraulic excavator.

5.2 Cut Batter Slope Stability

For unsupported cuts, up to a height of 3m, the recommended batter slopes are presented in the following Table 5-1.

Table 5-1 Recommended Batter Slopes for Unsupported Cuts

Materials	Temporary (Horizontal : Vertical)		Permanent (Horizontal : Vertical)	
	Exposed	Protected	Exposed	Protected
Topsoil, Residual and Alluvial Soils	1.5H: 1.0V (34°)	1.0H : 1.0V (45°)	2.0H : 1.0V (27°)	1.5H : 1.0V (34°)
Weathered Basalt and Siltstone	1.0V: 1.0V (45°)	1.0H : 1.5V (56°)	1.0H : 1.5V (56°)	1.0H : 2.0V (63°)

Subjected to the frequency of rainfall at site during construction, temporary surface protection may be provided for temporary cuts. All batter slopes will need to be assessed and confirmed on site as construction work proceeds.

The stability of batter slopes within the basalt and siltstone rock will depend on the orientation and spacing of joints and defects, which should be assessed during construction phase. For preliminary design purposes batter slopes within weathered basalt and siltstone may be adopted based on the recommended parameters presented in Table 5-1 above.

5.3 Fill Batter Stability

Fill batters up to 10m high may be supported by battering at 2H:1V. On sloping ground they shall be keyed into the slope using terraces not less than 1.0m high and 1.0m wide.

The footprint of embankments shall be inspected and proof rolled as per Section 5.5 to ensure they are founded on sound material and unsuitable material is not present.

5.4 Re-use of In-Situ Materials

The following comments are provided on the potential re-use of excavated materials for engineered fill:

- The performance of the residual sandy clay and clayey sand soils is likely to be sensitive to changes in moisture content and there is potential to heave or fail to compact under high moisture conditions. Careful moisture conditioning and compaction will be required to compact these materials effectively, all as indicated in Section 5.5 below.
- The extremely to distinctly weathered basalt and siltstone rock may be re-used as engineered fill if, during excavation, handling and re-compaction, the rock breaks down to fragments in the order of 100mm or less. Generally zones of rock fragments that are larger than 100mm, may only be used as rock fill. Alternatively, these materials may be used as engineered fill following processing of rock into an aggregate of particle size 100mm or less.

5.5 Sub-grade Preparation and Fill Placement

It is recommended that the following site preparation be carried out for pavement sub-grade and fill placement beneath structures and footings using predominantly residual sandy clay and clayey sand soils and broken up basalt and siltstone rock.

5.5.1 Bulk Earth Filling (Residual Soils and Extremely Low to Low Strength Rock)

- Remove any soft, wet, and highly compressible material or topsoil material and organics.
- Assess moisture contents of the bulk excavated soils and weathered rock. For compaction of any materials other than free draining sands, the moisture content should be in range OMC +/-2% (wet/dry), where OMC is the optimum moisture content at Standard Compaction.
- Test roll the complete surface of the sub-grade in order to detect the presence of any soft or loose zones, which should be excavated out and replaced with approved filling. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 8 tonne.
- For pavements, compact the natural foundation soil to a minimum dry density ratio of 98% Standard for clay soils or a minimum density index of 75% for sand soils.
- For pavements, approved filling excavated from site, should be placed in layers not exceeding 250mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% Standard or a minimum density index of 75% for filling greater than 0.5m below top of finished sub-grade level. It is recommended that the final upper 0.5m of filling sub-grade be compacted to a minimum dry density ratio of 100% Standard or 80% density index. Where filling has a clay content, moisture content within the filling should be maintained within OMC -2% (dry) to OMC +2% (wet) during and after compaction.
- All filling beneath structures and footings should be compacted to a dry density ratio of at least 100% Standard or relative density index of at least 80%. This compaction should apply to all filling extending from a nominal horizontal distance of 2m at the edge of each structure with a nominal zone of influence of 1H:1V down and away from the proposed sub-grade level.
- Any compaction of silty or sandy clay foundation soils at or close to footing formation level should be sealed or covered as soon as practicable, to reduce the opportunity for occurrence of desiccation and cracking.

- Level 1 testing and supervision of filling, in accordance with AS3798, is recommended where the filling is to be used for support of structural loads, within the 2m horizontal distance and spread from structures as outlined above.
- All weathered rock, excavated from site for re-use beneath structures and as pavement sub-grade filling, should be processed so that individual particles are in the order of 100mm or less.

5.5.2 Bulk Rock Filling (Medium to High Strength Rock)

For general bulk rock filling placed outside the area of influence of the various structures (refer Section 5.5.1 above), it is recommended that the following site preparation be carried out for sub-grade preparation and rock fill placement:

- Remove any soft, wet, and highly compressible material or topsoil material rich in organics or root matter.
- Assess moisture contents of the bulk excavated soils and weathered rock. For compaction of any materials other than free draining sands, the moisture content should be in range OMC -2% (dry) to OMC +2% (wet), where OMC is the optimum moisture content at Standard Compaction.
- Test roll the complete surface of the sub-grade in order to detect the presence of any soft or loose zones, which should be excavated out and replaced with approved filling. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 8-tonne.
- All weathered rock, excavated from site for re-use beneath structures and as pavement sub-grade filling, should be processed so that individual particles are in the order of 100mm or less.
- Approved rock filling excavated from site should be placed in layers not exceeding 300mm loose thickness with care taken to minimise the occurrence of voids. Fine sands and dispersive clays should not be included in the fill due to the susceptibility to erosion.

Difficulty to measure the density of bulk rock fill layer using conventional earthworks testing equipment (ie. nuclear densometer and laboratory compaction testing) must be recognised and it may be necessary to establish a suitable roller routine to achieve 'acceptable' compaction level. It follows that, where strict settlement criteria are imposed on the proposed structure, there is a higher risk of settlement under bulk rock filling due to the potential of void creation during placement and due to the lack of conventional earthworks testing to confirm density levels.

5.5.3 Pavements over Bulk Rock Filling

- Where pavements are proposed over bulk rock filling placed in accordance with Section 5.5.2 above, it is recommended that the rock fill be covered with a non-woven, needle punched, continuous filament polyester geofabric of sufficient strength to avoid punching failure.
- Place a minimum 0.5m thick cover of granular bridging on the geofabric in two layers of 250mm loose thickness, to provide sub-grade support for the pavement. The bridging layers should be compacted to a minimum dry density ratio of 100% Standard or 80% density index.
- Granular bridging or sub-grade filling should comprise engineered fill material supplied and placed in accordance with Section 5.5.1 above.

5.6 Pavement Sub-grade

The results of limited soaked CBR tests conducted on selected sub-grade samples of residual sandy clay, sandy or gravelly sand, indicated CBR values of between 1.5% and 10%.

Based on the findings of investigations, it is recommended that a CBR value of 2% to be adopted for sub-grade materials with a high clay content (such as where the Basalt outcrops), and a CBR value of 10% adopted for predominantly weathered siltstone bedrock in the design of flexible sealed or unsealed granular pavement.

These values are estimated to be close to a lower bound value of these materials and are based on the assumption that the topsoil will be stripped prior to pavement construction. It is also contingent upon adequate site preparation by proof rolling (to detect any unsuitable soft or loose materials) and sub-grade compaction procedures as recommended in Section 5.5 above.

Different values may be found where clay or rock fill is imported from elsewhere on the site and used in the road embankment. Such values can only be determined after a representative sample comprising similar plasticity content and particle size, as proposed to be used, is subjected to additional CBR testing.

The above recommendations are based on the provision and maintenance of adequate surface and subsurface drainage.

5.7 Slope Stability Assessment and Erosion

Slope instability issues have been found along the Abercrombie Road, adjacent to the southern central boundary of the site. The subject area and its hilly surrounds support mature, healthy native forest vegetation. Numerous mature trees surrounding and down and up slope of the Abercrombie Road have curved and leaning trunks, showing continued down slope soil creep. Small slope failure has occurred during the investigation period (refer to site photographs attached in Appendix C). No evidence of major slope instability was observed.

Slope instability issues are likely to be confined to steeply sloping land at the head of a gully. In generally the access roads should be designed to stay on the ridge crests and remain clear of potential land slips.

If crossing a potential land slip is required then the road formation should be designed to remove any potentially unstable material and found on stable bedrock.

The results of a limited number of laboratory Emerson Class dispersivity tests on selected near surface samples of residual soils indicate there is a low dispersion potential under acidic conditions.

It should be recognised, however, that there is a relatively high proportion of silty sands across the site, which can potentially scour under concentrated water flows. It is therefore recommended that site works, including excavation and filling, be planned accordingly to reduce the risk of high concentrated surface water runoff.

URS understands a Soil Erosion Management Plan will be prepared as part of the Construction Environmental Management Plan.

Further Geotechnical Investigations

The current study presents an appraisal of likely conditions across the Paling Yards Wind Farm site. Access at this relatively early stage in the project has been limited, to the extent that a fully representative sample of site conditions may not have been obtained. It is recommended that further detailed subsurface geotechnical investigation and analysis be conducted to provide information for the detailed design of footings, access road, slope stability, and other associated infrastructure.

7 Closure

This preliminary geotechnical investigation has provided a better understanding of the geological setting and its impacts on the proposed Paling Yards Wind Farm. It has revealed that from the investigations carried out, there are no major geological issues that would potential prevent the construction of the proposed development, provided the recommendations of this study are followed and further investigation is undertaken at a later stage where warranted.

The attached document titled “Appendix B - Report Explanatory Notes” presents additional information on the uses and limitations of this report.

References

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Limitations

URS Australia Pty Ltd (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Union Fenosa Wind Australia Pty Ltd and only those third parties who have been authorised in writing by URS to rely on the report. It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this report. It is prepared in accordance with the scope of work and for the purpose outlined in the Proposal dated 2nd March 2011.

The methodology adopted and sources of information used by URS are outlined in this report. URS has made no independent verification of this information beyond the agreed scope of works and URS assumes no responsibility for any inaccuracies or omissions. No indications were found during our investigations that information contained in this report as provided to URS was false.

This report was prepared between 22nd April 2011 and 18th August 2011, and is based on the site conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

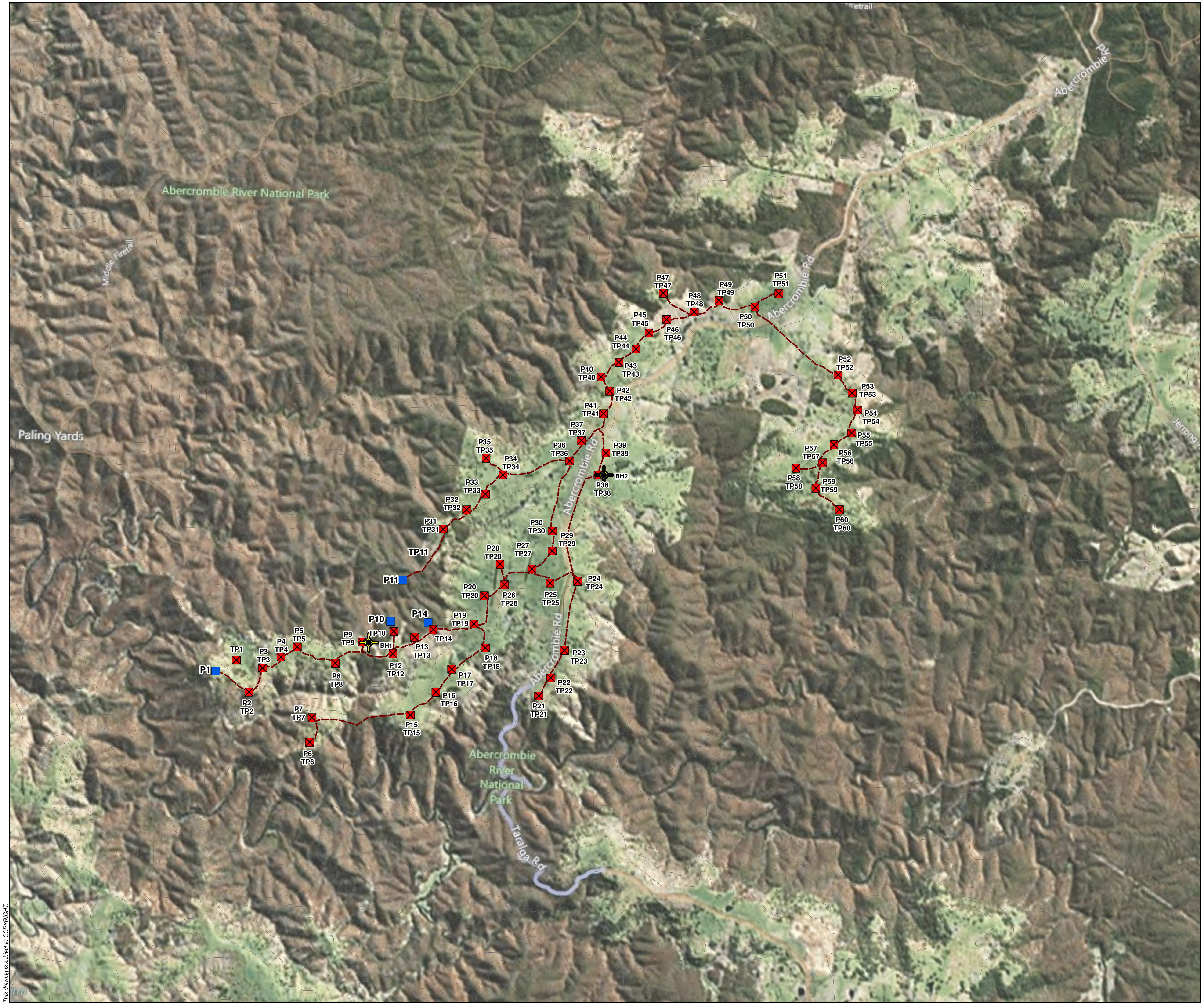
This report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

This report contains information obtained by inspection, sampling, testing or other means of investigation. This information is directly relevant only to the points in the ground where they were obtained at the time of the assessment. The borehole logs indicate the inferred ground conditions only at the specific locations tested. The precision with which conditions are indicated depends largely on the frequency and method of sampling, and the uniformity of conditions as constrained by the project budget limitations. The behaviour of groundwater and some aspects of contaminants in soil and groundwater are complex. Our conclusions are based upon the analytical data presented in this report and our experience. Future advances in regard to the understanding of chemicals and their behaviour, and changes in regulations affecting their management, could impact on our conclusions and recommendations regarding their potential presence on this site.

Where conditions encountered at the site are subsequently found to differ significantly from those anticipated in this report, URS must be notified of any such findings and be provided with an opportunity to review the recommendations of this report.

Whilst to the best of our knowledge information contained in this report is accurate at the date of issue, subsurface conditions, including groundwater levels can change in a limited time. Therefore this document and the information contained herein should only be regarded as valid at the time of the investigation unless otherwise explicitly stated in this report.

Appendix A Figures



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LEGEND

TP1 TEST PIT

P1 TURBINES

BOREHOLES

33kV UNDERGROUND CABLE

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W E
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0 0.5 1 2

Km

Datum: GDA94

Source: Aerial Image from Bing Maps Copyright © 2010 Microsoft Corporation and/or its suppliers

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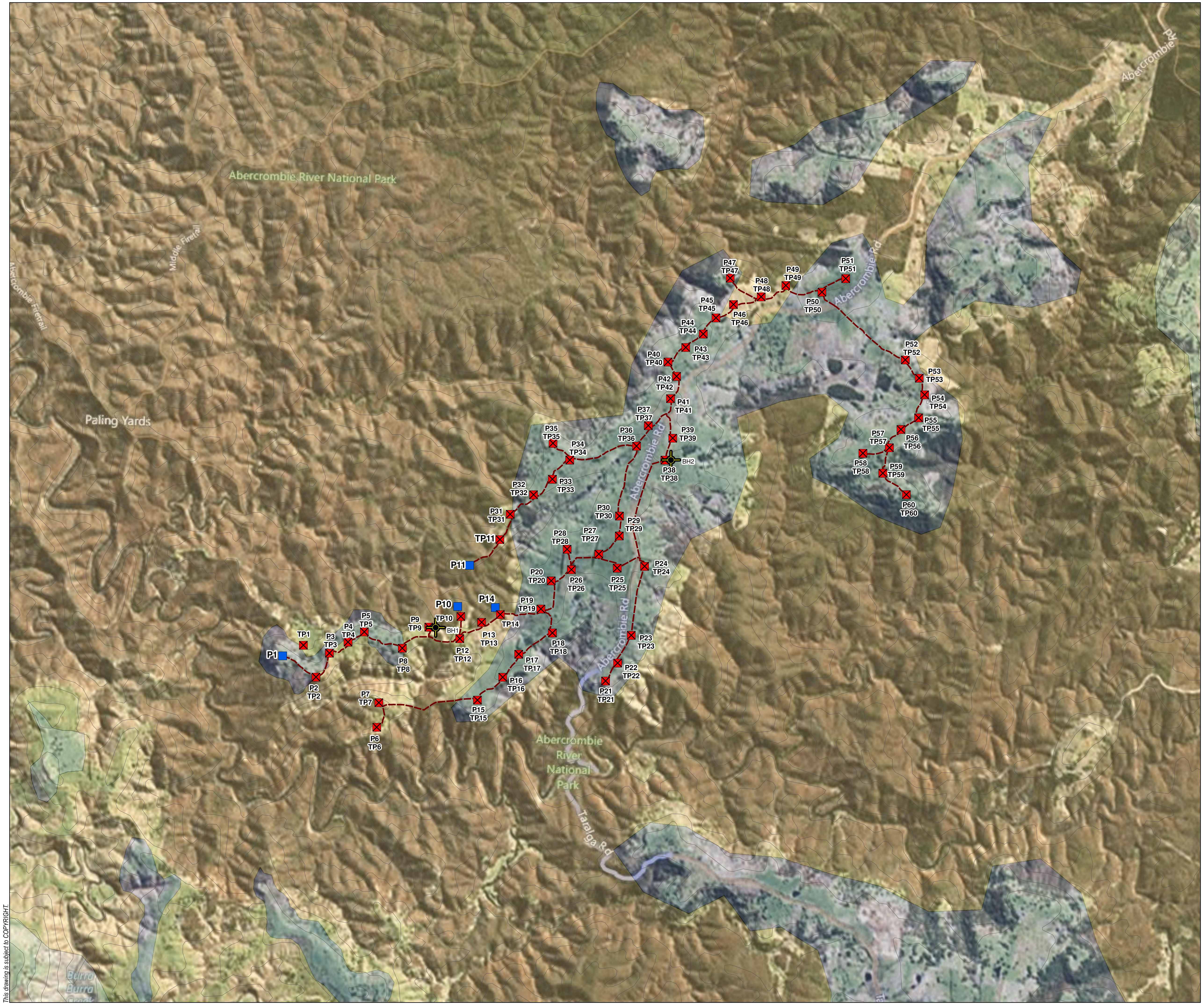
UNION FENOSA
WIND AUSTRALIA

PALING YARDS
WIND FARM

TEST PIT LOCATION
SITE PLAN

URS Figure: 1
Rev. A A3



LEGEND

TP1 TEST PIT

P1 TURBINES

BOREHOLES

33kV UNDERGROUND CABLE

GEOLOGY*:

Cainozoic (Tb), Basalt, dolerite

Ordovician (Ɵs), Silty sandstone, micaceous siltstone, phyllite, shale, slate quartzite and minor amount of porphyry

N W E S

0 0.5 1 2

Km

Datum: GDA94

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*NSW1500K_UnitBoundaries_GDA94_Lamberts - Geoscience Australia

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UNION FENOSA
WIND AUSTRALIA

PALING YARDS
WIND FARM

GEOLOGICAL MAP

URS Figure: 2

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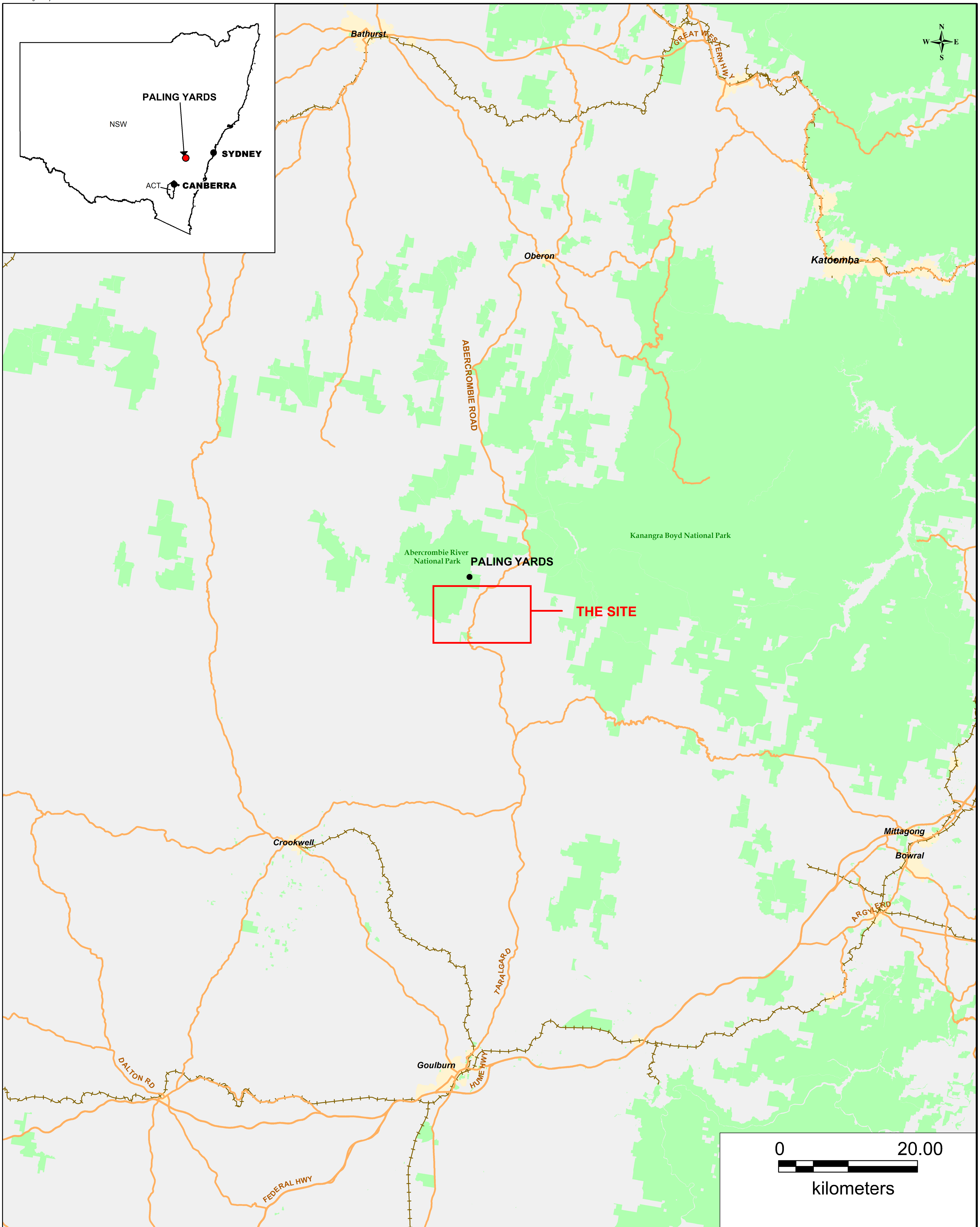

UNION FENOSA
WIND AUSTRALIA

PALING YARDS WIND FARM

Site Locality Map

URS

Figure: 3

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Appendix B Report Explanatory Notes

URS

REPORT EXPLANATORY NOTES

INTRODUCTION

These notes have been provided to amplify this Geotechnical Report in regard to investigation methodology, classification methods, field and laboratory procedures, the interpretation of the ground characteristics and the comments and recommendations based therein. Not all these notes are necessarily relevant to all reports.

LIMITATIONS ON INTERPRETATION, USE AND LIABILITY

The ground is a product of continuing natural and man-made processes and thus exhibits a variety of characteristics and properties that vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand and predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, drilling, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where, and the time when the investigation was carried out.

Any interpretation or recommendation given in this report shall be understood to be based on judgement and experience, not on greater knowledge of facts other than those reported. The interpretation and recommendations are therefore opinions provided for the Clients sole use in accordance with a specific brief. As such they do not necessarily address all aspects of the ground behaviour on the subject site.

The environmental investigation addresses the likelihood of hazardous substance contamination resulting from past and current known uses of the subject site. As a result, certain conditions such as those listed below may not be revealed:

- naturally occurring toxins in the subsurface soils, rock, water or the toxicity of the on-site flora;
- toxicity of substances common in current habitable environments such as stored household products, building materials and consumables;
- subsurface contaminant concentrations that do not violate present regulatory standards but may violate such future standards; and
- unknown site contamination such as "midnight" dumping and/or accidental spillage which may occur following the site visit by URS.

There is no investigation which is thorough enough to preclude the presence of material which presently, or in the future, may be considered hazardous at the site. Because regulatory evaluation criteria are constantly changing, concentrations of contaminants presently considered low may, in the future, fall under different regulatory standards that require remediation.

Opinions and judgments expressed herein, which are based on our understanding and interpretation of current regulatory standards, should not be construed as legal opinions.

The responsibility of URS is solely to our client, as noted on the cover of the report. This report is not intended for, and should not be relied upon, by any third party. No liability is undertaken to any third party.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS1726-1993, "Geotechnical Site Investigations".

In general, these descriptions cover the following properties - soil or rock type, structure, colour, strength/consistency or density, and inclusions.

Field identification and classification of soil and rock involves judgment and URS implies accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominant particle size and material behaviour, qualified by the presence of other soil particles and materials (eg sandy clay).

Non-cohesive soils are classified on the basis of relative density, generally from the results of insitu tests or field classification.

Cohesive soils are classified on the basis of soil consistency and undrained shear strength, determined by insitu tests or field classification.

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, discontinuities, etc. Where relevant, further information regarding rock classification is given in the text of the report.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination and laboratory testing (where required) of the soil or rock.

Disturbed soil samples are taken during field investigations to provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed soil samples are usually taken by pushing a thin-walled sample tube, usually 50mm to 100mm diameter (known as U50, U60, U75 etc.), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of soil strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

In very stiff or hard cohesive soils the URS driven ring lined sampler may be used to obtain samples. In some instances a thin wall extension tube is employed to minimise soil disturbance. The ring sampler is generally pushed hydraulically through 0.45 metres although in hard clays and dense sands it may be driven with the S.P.T. hammer. Where the sampler has been driven, an "equivalent N" value is shown on the borehole records.

Details of the type and method of sampling used during the field investigation are given on the engineering field logs provided with this report.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by URS with some comments on their use and application. All methods, except test pits, hand auger drilling and portable dynamic cone penetrometers, require the use of a mechanical drilling rig.

EXCAVATION AND DRILLING

Test pits - These are normally excavated with a backhoe or a tracked excavator. They allow close examination of the soils insitu condition up to a depth of about 1.5m, if safe, and collection of disturbed bulk samples from greater depths. The depth of penetration is limited to about 4m for a backhoe and up to 6m for an excavator. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction (not generally possible) or locate the pit outside an area of possible influence or to design and construct the structure so that it is not adversely affected by poorly compacted backfill at the test pit location.

Hand Augers - Boreholes of 50mm to 100mm diameter may be advanced manually. Hand augers are generally used where only shallow soil profiles are required (ie. less than 1.5m) or in areas inexcessable to larger drilling or excavation equipment. Limited insitu testing can be carried out within hand auger boreholes.

Refusal during hand augering can occur in a variety of materials, such as hard clay or gravel, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers - Boreholes are advanced using a 75mm to 115mm diameter continuous spiral flight auger, which is withdrawn at intervals to allow sampling and in situ testing.

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This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected by other techniques after the withdrawal of the auger flights, but they can be very disturbed and may be cross-contaminated.

Information from the drilling (as distinct from specific sampling by S.P.T.'s or undisturbed sampling) is of relatively low reliability due to remoulding, cross-contamination or softening of samples by groundwater or uncertainties as to the original depth of the materials. Augering below the groundwater table is of less reliability than augering above the water table.

Use can be made of a Tungsten Carbide (T.C.) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments.

Wash bore drilling - Boreholes are usually advanced by a mechanical or hydraulic rotary bit, with water or mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings.

The water or mud is also used to provide support to the borehole in difficult soil conditions. The term mud encompasses a range of products from bentonite to polymers such as Revert, foam or Biogel.

Only major changes in stratification can be determined from the cuttings returned, together with some information from "feel" and rate of penetration. The use of mud support may mask the identification of some soils from cuttings.

Generally, the use of wash bore drilling is carried out in conjunction with insitu testing and sampling at regular intervals to provide more accurate identification of changes in stratification.

Continuous Core Drilling - Continuous rock core samples are obtained using a diamond tipped core barrel.
Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a reliable (but relatively expensive) method of field investigation.

In rocks, an N.M.L.C. triple tube core barrel, which gives a core of about 50 mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as core loss. The location of losses are determined on site by the inspecting engineer. Where the location is uncertain, the loss is indicated at the top end of the drill run.

The core recovery ratio (CRR) is the ratio of recovered core to length cored expressed as a percentage. The rock quality designation (RQD) is a modified core recovery ratio in which only pieces over 100mm long are summed and expressed as a percentage of the core length.

FIELD TESTS

Standard Penetration Tests

Standard Penetration Tests (S.P.T.) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard AS1289, "Methods of Testing Soils for Engineering Purposes" - Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the "N" value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued. An equivalent extrapolated value for 300mm of penetration may be given.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4,6 and 7 blows, as

  4,6,7
  N = 13

- In a case penetration is incomplete, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm, the distance penetrated is given as

  15, 30 / 40 mm
  N > 30,
  [or Nx=225]

The results of the test can be related empirically to the engineering properties of soil.

Occasionally the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the S.P.T. is where the same driving system is used with a solid 60 degree tipped steel cone of the same diameter as the S.P.T. hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the hollow sampler. The results of this Dynamic Penetration Test are shown as "Nc" on the borehole logs, together with the number of blows per 150 mm penetration.

Static Cone Penetrometer Testing

Cone penetrometer testing (CPT) (sometimes referred to as a Dutch Cone Test) is used mainly in low strength soils as a means of determining a continuous profile of soil characteristics. The test is described in Australian Standard 1289, Test F5.1., and ASTM D3441-79.

In the tests, a 35 mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specifically designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate sleeve, immediately behind the cone. Advanced CPT equipment may also measure soil piezometric pressures at the tip and variation in the inclination of the cone probe. Transducers in the tip of the assembly are electrically connected to recorder unit at the surface.

As penetration occurs, (at a rate of about 20 mm per second) the information is output onto continuous chart recorders or stored on computer.
The information provided from CPT tests usually comprises:

- Cone resistance - the actual end bearing force divided by the cross sectional area of the cone - expressed in MPa.

- Sleeve friction - the frictional force on the sleeve divided by the surface area - expressed in kPa.

- Friction ratio - the ratio of sleeve friction to cone resistance, expressed as a percentage.

In addition the following may be given:

- Piezometric pressure - the pore water pressure at the cone tip expressed as kPa.

- Cone inclination - some cones may provide a continuous recording of the cone inclination expressed in degrees from vertical to determine the exact location of the probe.

The test method provides a continuous profile of certain soil characteristics. Stratification can be inferred from the cone and friction traces, from experience and information from nearby boreholes etc.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats.

Where shown, soil profile information is presented for general guidance only. Soil descriptions based on friction ratios are only inferred and must be regarded as interpretive, not an exact profile. Where precise information on soil classification and engineering properties are required, direct sampling from drilling may be preferable.

Correlations between CPT and SPT values can be developed for both sands and clays but may only be site specific. Interpretation of CPT values can be made to empirically estimate modulus or compressibility values to allow calculation of foundation settlements.

Portable Dynamic Penetrometers - Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive increments of penetration. The aim of the tests are to empirically estimate soil consistency and relative density.

Typically, DCP tests consist of driving a cone by the free-fall of a 9kg hammer. The number of blows for each 150mm of penetration is recorded. It is possible to relate these values obtained to empirical charts developed for soil consistency and relative density.

Two similar DCP tests are described by Australian Standards, AS1289 - F3.2 & F3.3. The major variation between these tests is the use of either a pointed or rounded penetration cone.

Interpretation of DCP results requires care and knowledge of local site conditions.

FIELD RECORDS/LOGS

The field logs or records attached with this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation.

Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits carried out during a field investigation represent only a very small sample of the overall subsurface conditions.

The attached explanatory notes for soil logs and rock logs define the terms and symbols used in preparation of the borehole or test pit records.

Interpretation of the information shown on the logs, and its application to design and construction should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits (for example, in limestone). Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time the hole is left open.

- A localised perched water table may lead to an erroneous indication of the true water table.

- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.

- The use of water or mud as a drilling fluid may mask any groundwater inflow or outflow. Drilling water has to be removed from the hole and drilling mud must be washed out of the hole or "reverted" chemically if accurate water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilisation of water levels, which may take several days to perhaps weeks for low permeability soils.

Piezometers, sealed in a particular stratum, are advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL MATERIALS

The presence of fill materials can often be determined only by the inclusion of foreign objects (e.g. bricks, steel etc.) or by distinctly unusual colour, texture or fabric.

Identification of the extent of fill materials will also depend on investigation methods and sampling frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing for engineering projects is normally carried out in accordance with the relevant Australian Standards. Details of each test procedure used will be provided on the individual report forms.

In order to maintain a high degree of quality control and assurance, URS utilise independent laboratories registered by the National Association of Testing Authorities (NATA).

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the field information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (e.g. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (e.g. to a twenty storey building). If this situation occurs, URS would be pleased to review the report and the sufficiency of the field investigation work in relation to the proposed development.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, URS cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions. The potential for this will be partially dependent on borehole spacing, sampling frequency and investigation technique as well as the time elapsed between investigation and construction;

- changes in policy or interpretation of policy by statutory authorities; and

- the actions of persons or contractors responding to commercial pressures.

If these occur, URS will be pleased to assist with investigation or advice to resolve any problems or disputes occurring.

SITE ANOMALIES

Our report, plans and specifications are prepared contingent to inspection of the site works by an experienced geotechnical engineer familiar with the report and the assumptions adopted in the design.

Should the conditions encountered during construction appear to vary from those which were expected, URS requests that it is notified immediately. This will enable URS to judge whether the actual conditions vary in significant extent and whether changes to the adopted design are required. Most problems are much more readily resolved when conditions are exposed, than at some later stage.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion of comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. URS would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

REVIEW OF DESIGN

Designs based upon information and recommendations provided in our geotechnical report should be reviewed to ensure that the intent of our report is reflected in the proposed design.

Where major civil, mining or structural developments are proposed or where only limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical consultant.

We would be happy to assist in this regard as an extension of our investigation commission.

SITE INSPECTION

URS will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

- a site visit to confirm that conditions exposed are no worse than those interpreted; to
- a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths; or
- full-time engineering presence on site.

CORE DESCRIPTION SHEET

General

The intention of Core Log Sheets is to present FACTUAL information measured from the core or as recorded in the field. Some interpretative information is inevitable in the location of core loss, description of weathering and identification of drilling induced fractures. This should be noted in the use of Core Log Sheets and remembered in their utilisation.

Progress

Drilling and Casing

The types of drilling used to advance the drill hole are recorded for relevant intervals. The types of drilling may include: NMLC CORING, NQTT (NQ triple tube wire line), HW, HX, NW and NS casing, wash boring (tri-cone roller bit, TC drag bit, TC blade bit) or auger drilling (V-bit, TC drag bit).

Water

Water lost or water made during drilling is recorded and subsequent readings of water levels in the borehole or piezometers are recorded here with dates of observation.

Drill Depth

Drilling intervals are shown by depth increments and full horizontal marker lines.

Core Loss

Core loss is measured as a percentage of the drill run. If the location of the core loss is known or strongly suspected, it is shown in a region of the column bounded by horizontal lines. If unknown, core loss is assigned to the top of a coring run.

Samples and Field Tests

The location of samples taken for testing or the location of field tests are indicated by the appropriate symbol shown at the relevant location or over the relevant depth interval.

Reduced Level (RL)

Changes in rock types or the locations of piezometer tips, samples, test intervals, etc. are shown when information on the RL of the top of the hole is available.

Strata

Rock types are presented graphically using the symbols shown on the log.

Description

The rock type is described in accordance with AS1726, 1993.

Weathering

Weathering is described, by code letters, in accordance with the Standard Borehole Explanation Sheet (Rock). A weathering term or range of terms is usually assigned to various strata.

It is noted, however, that the assignment of a term of weathering is subjective and is normally used for identification and does NOT imply engineering behaviours (such behaviour being controlled principally by rock substance strength and defect frequency - collectively, rock mass strength). Consequently, boundaries are often not shown and weathering may even not be reported where potentially misleading.

Estimated Strength

The strength of the rock substance is estimated by a combination of Point Load testing and tactile appraisal in accordance with the Standard Borehole Explanation Sheet (Rock). The estimated strength is presented in a histogram form. Both axial and diametric point load test results can be presented on the logs by using symbols described below. The variation between axial and diametric is indicative of anisotropy of fissility of the rock unit.

Discontinuity Information

The identification of discontinuities requires an endeavour to exclude drilling induced breaks in the core and, as such, can be somewhat subjective. Natural fractures exist prior to coring the rock, whereas artificial fractures occur either during coring, during placing core in the core boxes, or during examination of core after being boxed.

The log of discontinuity description is presented as a combination of Discontinuity Spacing, Visual and Description. The spacing excludes bedding partings (unless there is evidence that separation of the partings was present prior to drilling) and is presented as a histogram. The creation of the histogram is also somewhat subjective. The visual log is presented using coding for brevity. Where fractures are suspected to be drilling induced, but this is not conclusive, the fracture is shown dashed in the visual log and noted accordingly.

GENERAL

Symbol	Description
D	Disturbed Sample
U	Undisturbed Sample (suffixed by sample size or tube diameter in mm if applicable)
SPT	Standard Penetration Test (blows per 0.15 m)
N	SPT Value
PP	Pocket Penetrometer (suffixed by value in kPa)
SV	Shear Vane Test (suffixed by value in kPa)
C	Core Sample (suffixed by diameter in mm)
CL	Core Loss: indicates interval of no core recovery
Tp	Tensional Pull apart structure
DI	Drilling induced break
NC	Not continuous
●	Point Load Test (axial)
O	Point Load Test (diametric)
PBT	Plate Bearing Test
IMP	Impression Device Test
PZ	Piezometer Installation
PK	Packer Test
PM	Pressure Meter Test
R	Rising Head Permeability test
F	Falling Head Test
∀	Final Water Level (and Date)
➤	Water Inflow
≺	Water Outflow

DISCONTINUITY DESCRIPTORS

a) Type:

FL - Fault
JN - Joint
FO - Foliation
VN - Vein

BP – Bedding Parting
SH – Shear
CZ – Crushed Zone
FZ – Fractured Zone
DZ – Decomposed Zone

W –Widely spaced	600mm - 2m
M – Moderately spaced	200 – 600mm
C – Closely spaced	60 – 200mm
Vc – Very closely spaced	20 – 60mm
EC – Extremely closely spaced	<20mm

b) Defect Inclination:
Measured as dip/dip direction in exposure; or measured in degrees from core normal in boreholes (90° is vertical)

c) Defect Shape:

Pl – Planar
Cu – Curved
Wa – Wavy
St – Stepped
Ir – Irregular

d) Defect Roughness:

Slk – Slickensided / polished
S – Smooth
Sr – Slightly rough
R – Rough
Vr – Very rough

e) Type of Infilling:

C – Clay
Ca – Calcite
Cb – Carbonaceous material
Ch – Chlorite
Fe – Iron Oxide
KL - Clean
Lm – Limonite
Qz - Quartz
No – None
Su – Sulphides
Rf – Rock fragments
RC – Rock/Clay mixture
Uk - Unknown

e) Amount of Infilling:

Measured in mm or use –

St – Stain (for limonite)
Vn – Veneer (for other infill types)

f) Spacing:

ORDER OF DESCRIPTION

Soils are described as follows:
A) MAIN SOIL TYPE & UNIFIED CLASSIFICATION SYMBOL (BLOCK LETTERS)
B) Plasticity if fine grained **or Particle Size Distribution and Grading** if coarse grained
C) Particle Shape
D) Colour.
E) Secondary and Minor component(s); name, estimated proportion, plasticity, particle size, colour.
F) Moisture condition.
G) Consistency or density.
H) Geological Origin (FILL, ALLUVIUM, COLLUVIUM, RESIDUAL etc.).
e.g. Silty SAND (SM); medium grained, poorly graded, rounded, yellow-brown, with a trace of fine grained subangular gravel, dry, loose. *Quaternary Alluvium*

A) Main Soil Type

(See over for additional details)

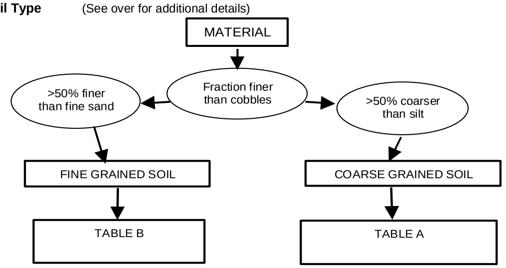


TABLE A: COARSE GRAINED SOILS: more than half of the material less than 60mm is larger than 0.06mm

			GRADATIONS	NATURE OF FINES	DRY STRENGTH	SYMBOL	TYPICAL NAME
>50% coarser than coarse sand	*Gravels*	Good	Wide range in grain size	Clean materials (not enough fines to bind coarse grains)	None	GW	Well graded gravels and sand - gravel mixtures, little or no fines
		Poor	Predominantly one size or range of sizes			GP	Poorly graded gravels and gravel - sand mixtures little or no fines
		Good to Fair	"Dirty" Materials (excess of Fines)	Fines are non-plastic	None to med	GM	Silty Gravels, gravel - sand - silt mixtures
				Fines are plastic	Med to high	GC	Clayey Gravels, gravel - sand - clay mixtures
>50% finer than fine gravel	*Sands*	Good	Wide range in grain size	Clean materials (not enough fines to bind coarse grains)	None	SW	Well graded sands and gravelly sands, little or no fines
		Poor	Predominantly one size or range of sizes			SP	Poorly graded sands and gravelly sands, little or no fines
		Good to Fair	"Dirty" Materials (excess of Fines)	Fines are non-plastic	None to med	SM	Silty Sand, sand-silt mixtures
				Fines are plastic	Med to high	SC	Clayey Sand, sand-clay mixtures

TABLE B: FINE GRAINED SOILS: more than half of the material less than 60mm is smaller than 0.06mm

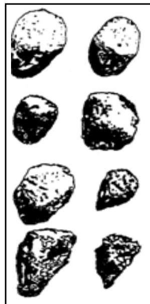
DRY STRENGTH	DILATANCY	TOUGHNESS	SYMBOL	TYPICAL NAME
None to Low	Quick to Slow	None	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
Medium to high	None to Very Slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays etc
Soil Low to medium	Slow	Low	OL	Organic silts and organic silt clays of low plasticity
Low to medium	Slow to none	Low to Medium	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
High to very high	None	High	CH	Inorganic clays of high plasticity
Medium to high	None to very slow	Low to Medium	OH	Organic clays of medium to high plasticity
Readily identified by colour, odour, spongy feel and generally by fibrous texture			Pt	Peat muck and other highly organic soils

Part Fresh	FR or staining	0.2	0.6	2.0	6	20	60	200	Size (mm)
Size		fine	medium	coarse	fine	medium	coarse		
Clay	Silt	Sand			Gravel			Cobbles	Boulders

B) Plasticity (fine grained soils) OR Grading (coarse grained soils)

DESCRIPTIVE TERM	LIQUID LIMIT (%)
Of low plasticity	<35
Of medium plasticity	>35; <50
Of high plasticity	>50

DESCRIPTIVE TERM	DEFINITION
Well graded	good representation of all particle size from largest to the smallest
Poorly graded	one or more intermediate size poorly represented
Gap Graded	one or more intermediate sizes absent
Uniform	Essentially one size



C) Particle Shape

D) Colour

Described in the moist condition, using simple terms (eg black, white, grey, red, brown, yellow etc, modified as necessary by 'pale', 'dark' or 'mottled'. Borderline colours may be described as a combination of these colours. (eg dark grey, red-brown)

E) Proportion of Secondary & Minor Components

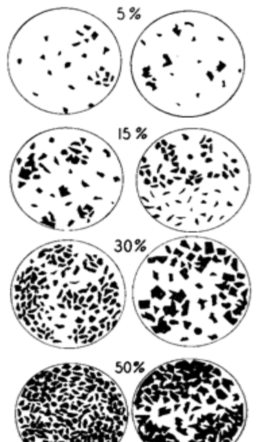
COARSE GRAINED SOILS		FINE GRAINED SOILS	
% fines	Modifier	% coarse	Modifier
<5	"with a trace of"	<15	"with a trace of"
>5 <15	"with some"	>15 <30	"with some"
>15	prefix soil with "silty, clayey, sandy, or gravelly"	>30	prefix soil with "sandy, gravelly silty, or clayey"

F) Moisture Content

CONDITION	CRITERIA
Dry	Cohesive soils hard and friable, granular sands free running
Moist	Soils feel cool, darkened in colour. Cohesive soils can be moulded. Granular soils cohere
Wet	Soils feels cool, darkened in colour. Free water forms on hands when handling

Organic and Artificial Materials

PREFERRED TERMS	TYPE
Fibrous Peat Charcoal Wood Fragments Roots (greater than 2mm dia) Root fibres (less than 2mm dia)	Organic Matter
Oil, Bitumen Domestic Refuse Brickbats Concrete Rubble Fibrous Plaster Wood pieces, shavings, sawdust Iron filings, drums, steel scrap Bottles, broken glass Leather	Waste Fill



G) Consistency or Density

GRAVEL/SAND (GW, GP, GM, GC)

DESCRIPTION	FIELD TEST
Loosely Packed	Can be removed from exposure by hand or easily removed by shovel
Tightly Packed	Requires pick for removal, either as lumps or as disaggregated material

SAND (SW, SP, SM, SC)

DENSITY	FIELD TEST	PSP Blows/150mm	SPT(N-value)	RELATIVE DENSITY (%)	CPT q_c (Mpa)
Very Loose	Easily penetrated with 13mm reinforcing rod pushed by hand Can be excavated with a spade; 50mm wooden peg can be driven easily	0 - 1	<4	<15	0-2
Loose	Easily penetrated with 13mm reinforcing rod pushed by hand Can be excavated with a spade; 50mm wooden peg can be driven easily	1 - 3	4- 10	15 - 35	2 - 5
Medium Dense	Penetrated with 13mm reinforcing rod driven with 2kg hammer - hard shovelling	3 - 8	10 - 30	35 - 65	5 - 15
Dense	Penetrated 300mm with 13mm reinforcing rod driven with 2kg hammer, requires pick for excavation; 50mm wooden peg hard to drive	8 - 15	30 - 50	65 - 85	15 - 25
Very Dense	Penetrated only 25 - 50mm with 13mm reinforcing rod driven with 2kg hammer	>15	>50	>85	>25

SILT AND CLAY (ML, CL, OL, MH, CH, PH, Pt)

CONSISTENCY	FIELD TEST	DCP (blows/ 150mm)	SPT (N)	UNDRAINED SHEAR STRENGTH (kPa)	UCS (pocket penetrom.) (kPa)	CPT q_c (kPa)
Very Soft	Easily penetrated 40mm by thumb. Exudes between thumb and fingers when squeezed	<1	<2	<12	<25	0 - 180
Soft	Easily penetrated 10mm by thumb. Can be moulded by light finger pressure	1 - 1.5	2- 4	12 - 25	25 - 50	180 - 375
Firm	Impression made by thumb with moderate effort. Can be moulded by strong finger pressure	1.5 - 3	4 - 8	25 - 50	50 - 100	375 - 750
Stiff	Slight impression made by thumb, cannot be moulded by fingers	4 - 6	8 - 16	50 - 100	100 - 200	750 - 1500
Very Stiff	Very tough. Readily indented by thumbnail	7 - 12	16 - 32	100 - 200	200 - 400	1500 - 3000
Hard	Brittle. Indented with difficulty by thumbnail	>12	>32	>200	>400	>3000

DATA FOR DESCRIPTION AND CLASSIFICATION OF SOILS

URS

ORDER OF DESCRIPTION

Rock Material is described as follows:
A) MAIN ROCK TYPE (BLOCK LETTERS)
B) Strength
C) Weathering
D) Colour e.g. black, white, grey, red, brown, orange, yellow, green, or blue - using pale, dark or mottled.
E) Fabric (spacing and development)
F) Particle Size (if coarse grained)
G) Inclusions or minor components
H) Degree of Fracturing (drill core) or Defect spacing (outcrop)
Geological Name (optional)
eg. GRANODIORITE, very high strength, slightly weathered, light pink-grey, massive, coarse sand sized. Jointing widely spaced. *Mowamba Granodiorite*

A) Main Rock Type *SEE OVER PAGE*

B) Strength

Rock Strength is defined by the Point Load Strength Index (Is50) and refers to the strength of the rock substance in the direction normal to the fabric

STRENGTH	SYM-BOL	Is(50) (MPa)	UCS (approx)	FIELD GUIDE
Extremely Low	EL	<0.03	>0.7	Easily remoulded by hand to a material with soil properties
Very Low	VL	0.03 - 0.1	0.7 - 2.4	Material crumbles under firm blows with sharp end of pick; can be peeled by a knife. Pieces up to 30mm thick can be broken by finger pressure.
Low	L	>0.1 - 0.3	2.4 - 7	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long and 50mm diameter may be broken by hand.
Medium	M	>0.3 - 1.0	7 - 24	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty
High	H	>1 - 3	24 - 70	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken in one blow by a geological hammer
Very High	VH	>3 - 10	70 - 240	Hand specimen breaks with geological hammer after more than one blow; rock rings under hammer
Extremely High	EH	>10	>240	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer

C) Weathering

TERM	SYM-BOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely Weathered	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded, in water.
Distinctly Weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly Weathered	SW	Rock is slightly dicoloured but shows little or no change in strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition or staining.

E) Fabric

Typical rock fabrics include but are not limited to:
- bedding, cross bedding (sedimentary)
- flow banding (igneous)
- schistosity or foliation (metamorphic)

FABRIC SPACING

TERM		SEPARATION OF STRATIFICATION PLANES
Sedimentary	Igneous/Metamorphic	
Thinly laminated	Thinly foliated	< 6mm
Laminated	Foliated	6mm to 20mm
Very thinly bedded	Very thinly layered	20 to 60mm
Thinly bedded	Thinly layered	60mm to 0.2m
Medium bedded	Medium layered	0.2m to 0.6m
Thickly bedded	Thickly layered	0.6m to 2m
Very thickly bedded	Very thickly layered	>2m

FABRIC DEVELOPMENT

Massive	No obvious fabric - rock appears homogenous
Poorly developed	Fabric is barely obvious as faint mineralogical layering or grain size banding.
Well developed	Fabric is apparent as distinct layers or lines marked by mineralogical or grain size layering.
Very well developed	Fabric is often marked by a distinct colour banding as well as by mineralogical or grain size layering.

F) Particle Size

0.2	0.6	2.0	6	20	60	200	Size (mm)
fine	medium	coarse	fine	medium	coarse		
Sand			Gravel			Cobbles	Boulders

Sedimentary rocks:

Sandstone - Use sand terms
Conglomerate - Use gravel terms
Shale, Siltstone
Claystone - No description of grain size is necessary

Metamorphic and Igneous Rocks:

Either record the grain size in millimetres **or** use appropriate sedimentary term, for example, "fine sand sized crystals", medium gravel sized crystals"

G) Inclusions or Minor Components

Any isolated minor components within the rock material may be described using the appropriate terms. Some examples are given in the table below.

Sedimentary Rocks	Igneous Rocks
Concretions	Vesicles
Ironstone Band	Xenoliths
'Tea leaf' structure	Phenocrysts

H) Degree of Fracturing or Defect Spacing

Degree of Fracturing (borehole core)

TERM	DESCRIPTION
Fragmented	The core is composed primarily of fragments of length less than 20mm, and mostly of width less than the core diameter
Highly Fractured	Core lengths are generally less than 20mm - 40mm with occasional fragments
Fractured	Core lengths are mainly 30 - 100mm with occasional shorter and longer sections
Slightly Fractured	Core lengths are generally 300 - 1000mm with occasional longer sections and occasional sections between 100 to 300mm
Unbroken	The core does not contain any fractures

Defect Spacing (Outcrop)

TERM	SPACING (mm)
Extremely closely spaced	<20
Very closely spaced	20 - 60mm
Closely spaced	60 - 200m
Moderately spaced	200 - 600mm
Widely spaced	600mm - 2m

Rock Mass Defects Order of Description: Type; inclination, shape, roughness, infill type, infill thickness

1. Defect Type

Abbreviation	Map Symbol	Description
FL		Fault - fracture along which displacement is recognisable.
SH		Shear - a fracture along which movement has taken place but no displacement is recognisable. Evidence for movement may be slickensides, polishing and/or clay gouge.
SZ		Sheared Zone - zone of multiple closely spaced fracture planes with roughly parallel planar boundaries usually forming blocks of lenticular or wedge shaped intact material. Fractures are typically smooth, polished or slickensided; and curved.
BP		Bedding parting - arrangement in layers of mineral grains or crystals parallel to surface of deposition along which a continuous observable parting occurs.
BSH		Bedding plane shear - a shear formed along a bedding plane
JN		Joint - a single fracture across which rock has little or no tensile strength and is not obviously related to rock fabric.
CN		Contact - surface between two lithologies.
FO		Foliation – a planar arrangement of textural or structural features in any type of rock, especially the planar orientation of platy minerals.
CV		Cleavage - plane of mechanical fracture in a rock normally sufficiently closely spaced to form parallel-sided slices.
CZ		Crushed Zone - zone with roughly parallel, planar boundaries (commonly slickensided) containing disoriented usually angular rock fragments of variable size often in a soil matrix.
VN		Vein - fracture in which a tabular or sheet-like body of minerals have been intruded.
DZ		Decomposed Zone - zone of any shape but commonly with parallel boundaries containing moderately to extremely weathered rock, typically with gradational boundaries into fresher rock.
FZ		Fractured Zone - a zone of closely spaced defects (mainly joints, bedding, cleavage and/or schistosity) comprised of core lengths in the order of 50mm or less.

2. Defect Inclination
measured as dip/dipdirection in exposure
measured in degrees from core normal in boreholes (90° is vertical)

3. Defect Shape

Symbol	Term	Description
Pl	Planar	Forms a continuous plane without variation in orientation
Cu	Curved	Has a gradual change in orientation
Wa	Wavy	Has a wavy surface shape
St	Stepped	Has one or more well defined ridges
Ir	Irregular	Many changes of orientation

4. Defect Roughness

Sym-bol	Term	Description
Slk	Slickensided /polished	Visual evidence of striations or a smooth glassy finish
S	Smooth	Surface appears smooth and feels so to the touch
Sr	Slightly Rough	Asperities on the defect are distinguishable and can be felt
R	Rough	Some ridges and angle steps are evident; asperities are clearly visible and surface feels very abrasive
Vr	Very Rough	Near right angle steps and ridges occur on the surface

5. Defect Infill

Symbol	Description	Symbol	Description
KL	Clean	g	gravelly -
Ca	Calcite	s	sandy -
Cb	Carbonaceous material	z	silty -
Ch	Chlorite	c	clayey -
Lm	Limonite	G	Gravel
Qz	Quartz	S	Sand
Su	Sulphides	Z	Silt
Rf	Rock fragments	C	Clay
RC	Rock/Clay mixture	hp	high plasticity
		lp	low plasticity

6. Infill Thickness
measured in mm or use "St" (stain) - Limonite or "vn" (veneer) - other infill types

DATA FOR DESCRIPTION AND CLASSIFICATION OF ROCKS

URS