Final Report

Preliminary Geotechnical Site Assessment: 17-21 Parramatta Road, Lidcombe

30 SEPTEMBER

Prepared for

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Introduction

1.1 General

URS Australia Pty Ltd (URS) was commissioned by Costco Wholesale (Australia) Pty Ltd (Costco) to conduct a Phase 1 and Limited Phase 2 Environmental Site Assessment (ESA) for the proposed bulk goods outlet at 17-21 Parramatta Road, Lidcombe, NSW. A preliminary geotechnical investigation was included as a part of the Environmental Site Assessments (ESAs) carried out at the site. The subject of this geotechnical investigation is limited to areas outside the existing warehouse building.

It is understood that the objective of the geotechnical investigations is to provide sufficient information for the foundation design of the proposed warehouse building. It is proposed that the new warehouse building would comprise two levels of car parking and another floor of sales and office area. Figure 1-1 shows the latest available concept plan information provided by Group GSA dated 3 September 2009. The option being considered for the car park is a Basement carpark with the finished floor level located at RL 2.34m or approximately up to 3.5m below the existing ground surface.

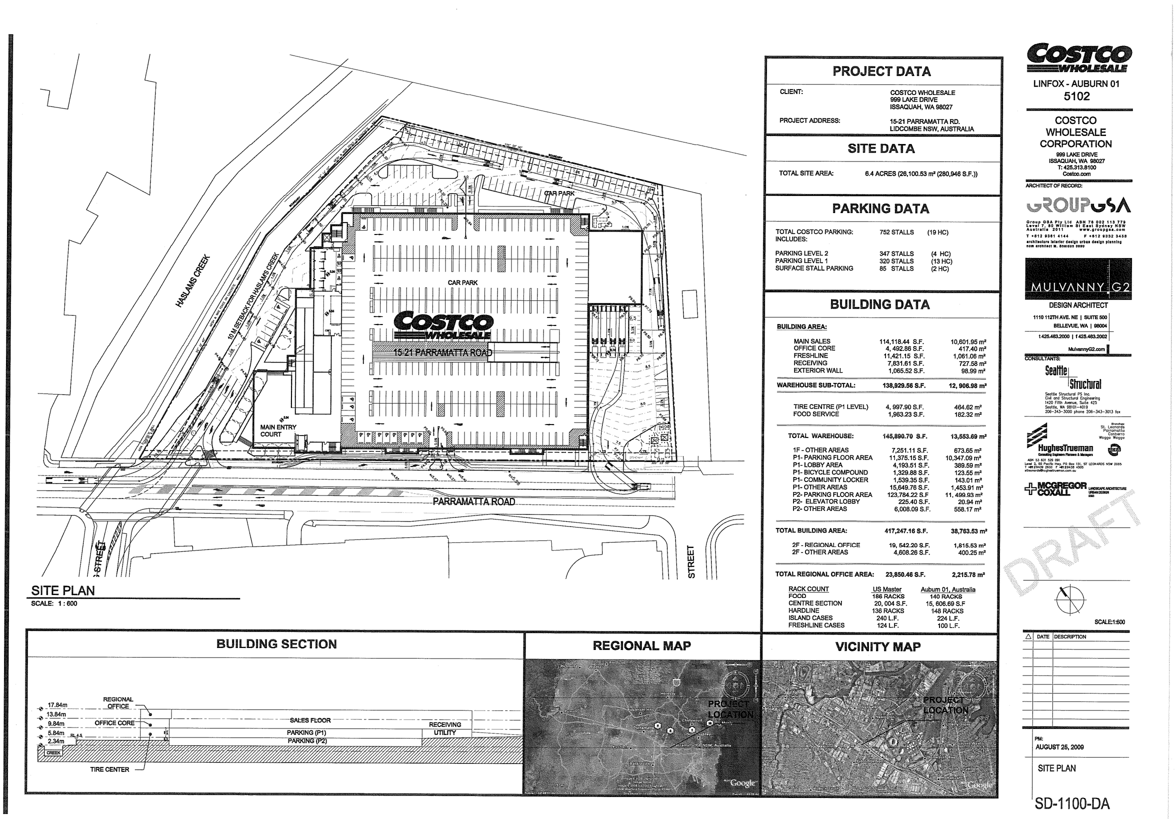

Figure 1-1 Concept Site Plan

This report presents and focuses only on the geotechnical investigation and assessment of the work. The results of Phase 1 and Limited Phase 2 ESA are presented in a separate report. The present report describes the scope, methods and results of the geotechnical investigations performed at the Site to date. The report also provides our preliminary geotechnical assessment of the site conditions and makes recommendations with regards to groundwater considerations, basement design, excavation support and general foundation design for the proposed work, site preparation works and the existing contiguous pile wall along Haslam's Creek.

1.2 Site Description

The site is located at 17-21 Parramatta Road, Lidcombe, NSW and is approximately 15 kilometres west of Sydney Central Business District (CBD). The site comprises an area of approximately 2.6 ha, described as Lot 1 in Deposited Plan (DP) 214452. Figure 1-2 shows the site layout (Frankham Engineering Surveys Pty Ltd, Site Plan, 209077, dated 9 April 2009).

The surrounding land use is as follows: Directly adjacent to the northern boundary is Haslams Creek. The M4 Western Motorway is located to the north-eastern corner of the site. Parramatta Road is located directly to the south of the site. Directly adjacent to the south eastern boundary is Hertz building.

The site is relatively flat with cross fall across the site of less than 1m towards Haslam's Creek.

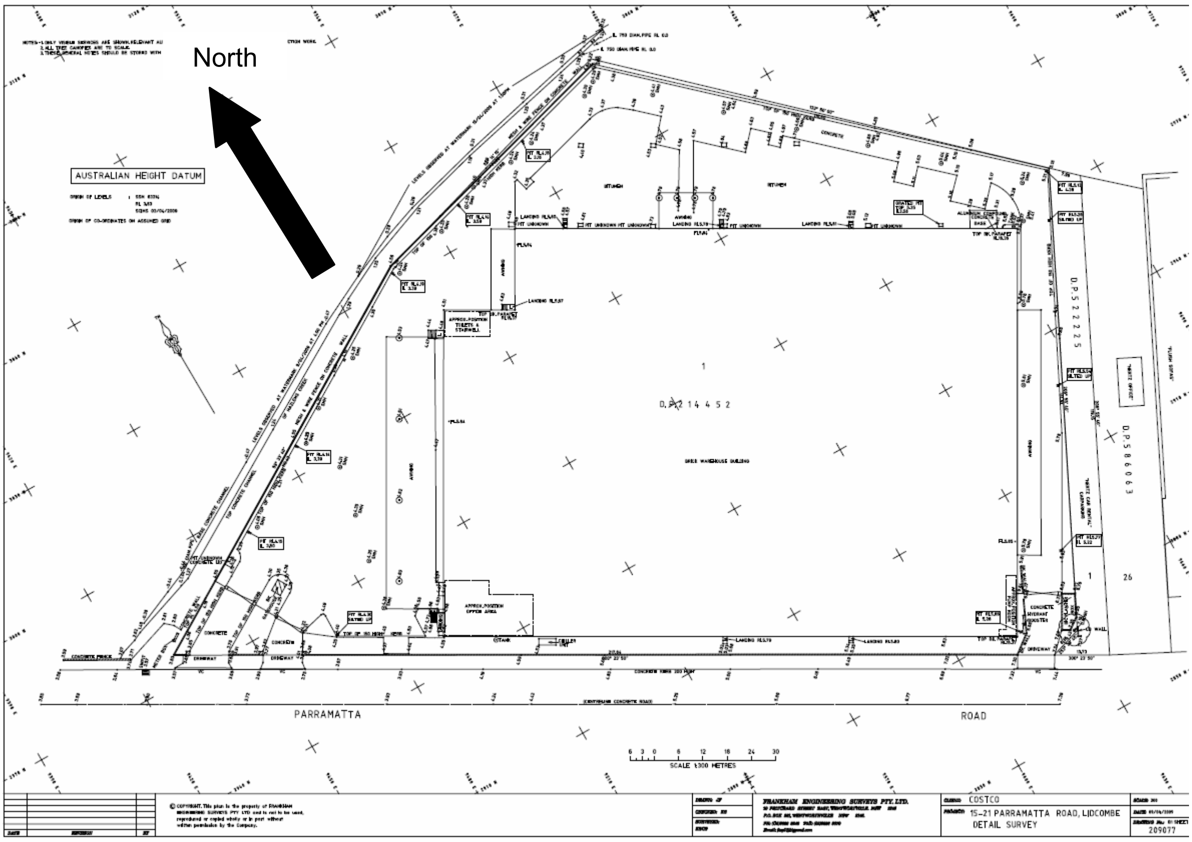

Figure 1-2 Site Layout

(Source: Frankham Engineering Surveys Pty Ltd)

1.3 Site Geology

The 1:100,000 Geological Sheet for Sydney (9130, 1983) indicates that the site is close to a geological border between Ashfield Shale and Quarternary Alluvium associated with Haslam's Creek. The Ashfield Shale comprises black and dark gray shale and laminate of the Triassic Wianamatta Group. It is expected that this Shale is underlain by the medium to coarse-grained quartz sandstone, very minor shale, and laminite interbeds. The alluvium comprises silty to peaty quartz sand, silt and clay, with ferruginous and humic cementation in places and shell layers.

Geotechnical Investigations

2.1 General

The geotechnical investigation comprised 12 boreholes of which four (4) deep boreholes (MWD series) constructed as groundwater monitoring wells for groundwater level monitoring. In addition, four additional shallow monitoring wells (MWS series) were also constructed. These shallow wells were located approximately 0.5 m from the corresponding deep wells.

Locations of all boreholes were set out in the field prior to commencing fieldwork by URS and the final locations were recorded using a handheld GPS. The locations of geotechnical and environmental boreholes are shown on Drawing 1 in Appendix A. The geotechnical holes comprised SB06, SB07, SB10, SB12, SB14, SB15, SB18, SB22, MW1D, MW2D, MW3D and MW4D. Bores MW1D to MW4D were also constructed as deep monitoring wells. Adjacent shallow holes were labelled MW1S to MW4S.

In addition to geotechnical boreholes, there were 19 other environmental boreholes (soil bores) from which soil samples including quality control samples were submitted to the environmental laboratory for analysis. The complete discussions of the laboratory analysis results of these environmental samples are discussed separately in URS Report "Phase 1 and Limited Phase 2 Environmental Site Assessment (2009)".

As the geotechnical drilling works were not permitted inside the existing building, all boreholes were situated around the exterior of the existing building. The locations of these boreholes were chosen such that broad information on the ground conditions at the site could be obtained. Drawing No. 1 shows the locations of the geotechnical boreholes.

All field works were conducted in the full time presence of a URS Geotechnical Engineer who was responsible for sampling and logging the encountered strata. Field investigations were carried out between 29 May 2009 and 2 August 2009.

2.2 Investigations and Sampling

2.2.1 Borehole Drilling

Four geotechnical boreholes (SB06, SB12, SB15 and SB18) were drilled through soil to V-bit refusal within bedrock to between 7 and 8m depth. Eight boreholes (SB07, SB10, SB14, SB22, MW1D, MW2D, MW3D and MW4D) were continued through rock to target depths between 8.4 to 15m.

Drilling was undertaken using a truck mounted drill rig using solid flight augering techniques with the bores extended into bedrock using NMLC diamond coring (52mm diameter core). Initially augering was carried out using a V-Bit, with auger refusal depths noted on logs. Coring was commenced within each borehole from the depth of auger refusal to completion of the borehole.

Standard penetration tests (SPT's) were carried out during borehole drilling at regular intervals within the soil and weathered rock horizons, to assess in-situ strength/relative density of materials and to recover representative samples. Selected undisturbed samples were also recovered from boreholes by pushing 50 mm thin walled steel tubes into the soil and these soil samples were sent to the soil laboratory for characteristics, strength and consolidation testing.

Rock core recovered from the drilling was packed in core trays, geotechnically logged and photographed. Borehole logs and core photographs are provided in Appendix C.

2.2.2 Monitor Well Construction

Construction details for the standpipe piezometers are shown on the borehole logs. Two types of monitoring wells were constructed. The shallow monitoring wells (MWS series) refer to the wells constructed with screened interval within overburden alluvial soils. The deep monitoring wells (MWD series) refer to the wells constructed with screened interval within shale bedrock.

The piezometers were constructed using 50 mm diameter uPVC pipe with machine slotted screens. The annular spacing of the wells was packed with washed 2 mm quartz sand around the well screen and sealed with a layer of bentonite pellets, with the remainder of the holes filled with concrete-grout mix.

2.3 Geotechnical Laboratory Testing

The undisturbed soil samples and 21 rock core samples collected during the borehole drilling were submitted to Australian Soil Testing, which is a NATA accredited for a range of soil and rock testing including:

- Basic Index property testing including insitu moisture content and Atterberg Limits;
- Unconfined Compressive Strength
- Consolidated Undrained (CU) Triaxial Test
- One-dimensional Oedometer Test
- Rock Point Load Tests to assess the intact rock strength mass characteristics.

The results of these tests are presented in Tables 3-3 to 3-5 in Section 3.3. Point load rock strength tests are presented on the core logs, with a summary of the results provided in Table 3.6.

2.4 Groundwater Level Measurement

Following completion of the field investigations, the monitoring wells were developed by purging and allowing the groundwater levels to recover and reach equilibrium. Typically, groundwater levels were recovered 24 hours following purging. The monitoring wells were purged using poly tubing with a foot valve attachment or a disposable bailer. Water level readings taken are presented in Table 3-2. The recovery of the monitoring well after purging was also monitored to provide an indication of the permeability of the formation.

Site Geotechnical Conditions

3.1 Subsurface Conditions

Based on the review of available geotechnical information and results of the investigation, a geotechnical model has been developed for the site to assess excavation and foundation conditions across the site. A brief description of each of the identified geotechnical units forming part of the geotechnical model is provided in Table 3-1 in order of increasing depth. The inferred boundaries between the various units are presented on geotechnical cross-sections A, B and C showing the inferred extent, depths of these units and groundwater levels. (refer to Drawings Nos. 2, 3 and 4 in Appendix A). The soil types and strengths have been inferred based on SPT testing for the soil units, with point load testing results and field assessment used for the rock units.

The SPT results indicate that clayey soils in the area north of the realigned creek (old creek) were relatively weaker than the clayey soils to the south of the old creek corridor. This was also confirmed during the geotechnical investigation where undisturbed samples could not be recovered within boreholes MW1D and MW2D because of soft condition within the old creek corridor. The old creek corridor is shown on Drawing No. 1 in Appendix A.

A classification of the rock mass has also been provided which has generally been undertaken in accordance with the guidelines presented for foundations on sandstone and shales in the Sydney Basin (Pells et al, 1998). Table 3-1 shows the summary description of geotechnical units encountered at the site.

Table 3-1 Summary Description of Geotechnical Units

Geotechnical Unit	Approx. Thickness (m)	Summary Description
Unit 1: Pavement	0 – 0.15	**(Fill) – Concrete or Bitumen**.
Unit 2: Fill	0.15 to 3.25	**(Fill) – SAND,** uniformly graded, angular, brown, fine to medium grained moist.
Unit 3a: Alluvial Soil	0.3 to 4.8	**Silty CLAY,** typically soft to firm, medium to high plasticity, with trace of sand, brown.
Unit 3b: Residual Soil	0.5 to 5.3	**CLAY,** typically stiff to very stiff, medium plasticity , reddish, greyish brown
Unit 4: Bedrock		
Unit 4a	0.5 to 3.5	**(CLASS V/ CLASS IV) Shale–** extremely low to low strength, residual soil to distinctly weathered, grey, fragmented to highly fractured
Unit 4b	>1.5	**(CLASS III/ CLASS II) Shale/laminite –** typically low to high strength, distinctly to slightly weathered, grey, slightly to highly fractured

The alluvial soils comprising clay and silty clay materials were found within the upper soils in the north-western part of the site. The alluvial soils were usually medium plasticity and soft to firm. The residual clays found in the south-eastern part of the site were medium plasticity and typically stiff to hard. Ironstone bands were encountered within a few boreholes in the transition zone to weathered shale.

3.2 Groundwater Levels

Following the recent well installations, the results of static groundwater level measurements undertaken across the site on 10 August 2009 are presented in Table 3-2 below. This water level is higher than expected but it is relatively consistent across the site. Based on these water levels, it is apparent that the flow of groundwater is toward the North (Haslam's Creek).

Table 3-2 Groundwater Levels

Location	Date	Screened Interval of Piezometer (m)	Depth to Groundwater (m below top of casing)	Groundwater Level (m)
MW1S	10/8/09	3.0 – 6.0	2.83	RL 2.11
MW1D	10/8/09	7.5 – 15.0	2.90	RL 2.04
MW2S	10/8/09	3.0 – 6.1	3.53	RL 0.82
MW2D	10/8/09	7.5 – 15.0	2.84	RL 1.51
MW3S	10/8/09	2.5 – 5.5	2.86	RL 2.92
MW3D	10/8/09	5.8 – 13.0	3.07	RL 2.71
MW4S	10/8/09	3.0 – 6.1	2.30	RL 2.95
MW4D	10/8/09	7.0 – 13.9	2.47	RL 2.78

Based on the results of groundwater measurements, groundwater was encountered across the site between 2.3 to 3.5 m below existing ground surface. Based on the current architectural plans for Option 1, it is anticipated that basement levels will be mostly near or slightly below the groundwater level, within the soil Unit 2 or Unit 3a.

3.3 Laboratory Soil and Rock Testing Results

The soil and rock samples collected were subject to a range of laboratory tests (as outlined in Section 2) with a summary presented in the following tables. Copies of Laboratory Test Certificates are provided in Appendix C. Tables 3-3 to 3-5 present the results of soil characteristics, soil strength and consolidation testing, respectively.

Table 3-3 Soil Characteristics Testing Results

Location	Depth (m)	Description	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)
SB7	3.5-3.9	Silty CLAY: mottled yellow-brown & grey, medium plasticity, with fine to coarse sand	19.5	36	18
SB12	3.4-3.8	Silty CLAY: mottled grey & dark grey, low plasticity, with fine sand	18.0	26	15

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Location	Depth (m)	Description	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)
SB14	5.0-5.4	Silty CLAY: mottled yellow-brown & grey, medium plasticity, with fine to coarse sand	18.8	38	21
MW2D	3.0-3.5	Silty CLAY: mottled grey and yellow brown, low plasticity, with fine to coarse sand (pockets of sandy clay), roots present	-	30	18
MW3D	3.0-3.45	Clayey SILT: grey, low to medium plasticity, with fine sand	14.8	-	-
MW4D	3.0-3.45	Sandy CLAY: brown/grey, medium plasticity, with fine to coarse sand	17.8	-	-

Table 3-4 Unconfined Compressive Strength and CU Triaxial Testing Results

Location	Depth (m)	Description	Dry Density (t/m³)	Unconfined Compressive Strength (kPa)	Effective Cohesion (kPa)	Effective Angle of Friction (°)
SB7	3.5-3.9	Silty CLAY: mottled yellow-brown & grey, medium plasticity, with fine sand	1.74	122	-	-
SB12	3.4-3.8	Silty CLAY: mottled grey & dark grey, low plasticity, with fine sand	1.81	-	32	18
SB14	5.0-5.4	Silty CLAY: mottled yellow-brown & grey, medium plasticity, with fine to coarse sand	1.88	67	-	-
MW2D	3.0 – 3.5	Silty CLAY: mottled grey and yellow brown, low plasticity, with fine to coarse sand (pockets of sandy clay), roots present	2.12	-	8	25

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Table 3-5 One-Dimensional Consolidation Testing Results

Location	Depth (m)	Description	Recompression Index, Ca (x 10^{-3})	Compression Index, Cc	Coefficient of Consolidation t_{90} (kPa)	Coefficient of Volume Change, Mv (kPa^{-1} x 10^{-3})
SB12	3.4-3.8	Silty CLAY: mottled grey & dark grey, low plasticity, with fine sand	1.09	0.053	2.31	0.110
MW2D	3.0 – 3.5	Silty CLAY: mottled grey and yellow brown, low plasticity, with fine to coarse sand (pockets of sandy clay), roots present	3.21	0.130	1.25	0.238

[1] Based on pressure between 100 and 200 kPa

The laboratory test results indicate that the alluvial soils are generally classified as a low to medium plasticity silty Clay with fine to coarse sand.

Table 3-6 presents the statistical analysis results of 21 point load rock strength tests from the investigation boreholes within Units 4A and 4B. The tests were undertaken along both the axial and diametral core orientation and are expressed in terms of Is(50).

Table 3-6 Summary of Point Load Strength Test Results

Geotechnical Unit	Mean Point Load Strength I_s50 (MPa)		Standard Deviation Point Load Strength I_s50 (MPa)		Mean Inferred Axial UCS (MPa)
	Diametral	Axial	Diametral	Axial	Axial
Unit 4a	0.05	0.09	0.04	0.01	1.8 (extremely low to very low)
Unit 4b	0.48	0.58	0.24	0.28	11.6 (medium strength)

Inferred values of Unconfined Compressive Strength (UCS) using the empirical correlation of UCS equals 20 times $I_s(50)$ have also been included in the table. In this instance the axial point load tests are considered to be the more representative data for correlation purposes. Based on this correlation Unit 4B can be classified as typically ‘medium“ strength rock and, using terms presented in Appendix B.

3.4 Potential Acid Sulphate Soils (PASS)

In view of the present evidence of old creek within the site and also the requirements by the Director General’s Office, Department of Planning, six soil samples were collected from MW1D and MW2D and submitted to and analysed in the laboratory for presence of PASS.

3 Site Geotechnical Conditions

Soil samples for PASS were taken from 3.3 m to 6.5 m below ground level at MW1D and from 1.6 m to 5.2 m below ground level at MW2D. The soils comprised mainly dark grey to black, soft silty clay sediment with minor organic matter and a slight sulphide odour. The laboratory results indicate that PASS materials are present along the old channel at an average depth of 4 to 6 m below ground level.

Nevertheless, the likely presence of PASS will necessitate management of these soils, if disturbed. If future developments require the soils to be disturbed, URS recommend the preparation and implementation of an Acid Sulphate Soil Management Plan to ensure the material is managed in a way that is consistent with the relevant regulatory guidelines and is protective of the environment. The Acid Sulphate Soil Management Plan should be prepared in a manner which is consistent with the requirements and guidance outlined in the Acid Sulphate Soil Manual (ASSMAC, 1998).

Further investigations are required in areas to be excavated within the footprint of the new building to quantify volume of PASS to be generated during construction and to provide better estimates for soil treatment requirements.

Geotechnical Assessment

4.1 General

It is understood that the basement carpark being proposed for the structure of the proposed bulk goods outlet at 17-21 Parramatta Road Lidcombe at the time of preparing this report.

The building area will be allocated for Floor Sales, Office Core, Freshline, Receiving Room, Exterior Wall, Tire Centre and Food Service and there will be two floors car parks occupying 755 stalls.

There is no specific information on foundation loads and settlement tolerance provided. Thus, the information below has been given to assist the designers with foundation and excavation support requirements. It is recommended that once the final concepts have been developed further that additional geotechnical advice and investigation be undertaken to confirm assumptions and predictions to satisfy geotechnical and structural criteria.

To confirm the soil conditions underneath the existing building, it is recommended to carry out further geotechnical investigation inside the building once access to the building is permitted. As least four geotechnical boreholes are recommended.

4.2 Implications of the Proposed Basement Carpark

The following issues should be considered for the proposed basement carpark:

1. The measured groundwater levels are at between RL 0.8m and RL 3.0m. The finished floor level of the proposed basement (RL 2.3m) is below the groundwater level in some areas, especially in the eastern area of the site.
2. With consideration of long term water table rises during extended wet periods, the water level is expected to rise over short periods above the measured levels.
3. Consideration is to be given to either a drained or undrained basement. For the drained basement, the long term operating costs and risks will need to be compared to the initial capital cost for construction of undrained basement. The undrained basement will need to be designed to withstand long term uplift groundwater pressure taking into account seasonal fluctuations in groundwater levels.
4. For either option, construction dewatering will be required.
5. Calculation of groundwater inflows should be assessed separately for short term and long term inflows (If a drained basement is to be considered).
6. Methods of site dewatering during construction.
7. Predictions to be made for potential for groundwater drawdown induced settlements.
8. Temporary excavation support if required.
9. Presence of PASS materials may need an appropriate Acid Sulphate Soil Management Plan and consideration on the costs associated with soil treatments.
10. Traffic and construction issues related to working on soft subgrade.
11. Control of ground movements adjacent to excavations to ensure stability of other structures including the contiguous pile wall along Haslam's Creek.
12. Short and long term stability of the basement excavations with due consideration to soil stability and groundwater pressures.

4.3 Excavation Conditions and Support Requirements

4.3.1 Excavation Conditions

Excavation up to 3.5 metres below existing ground surface is likely to encounter only fill and top alluvial soils (within Units 1 to 3a). It is anticipated that the majority of this excavation could be carried out using conventional earth moving plant (i.e. excavators/dozer).

Where excavations are being untaken within Unit 2 fill and Unit 3 clay soils above the water table, temporary excavation batters should be excavated to a slope of 2.0(H):1(V) or flatter where space is available or otherwise temporary excavation support will be required.

Permanent batters, if any, should be formed not steeper than a slope of 3(H):1(V). Exposed surfaces of such permanent batters should be covered with an erosion control blanket, vegetative cover or other slope protection methods.

Excavated soils with no trace of contamination and meeting engineering fill requirements can be stockpiled and used as backfill materials. The slope for these stockpiled materials is recommended no steeper than 1.5(H):1(V).

4.3.2 Excavation Support

URS recommends that excavation support be provided for excavations of deeper than 1.5 m and where the consequence of failure could impact on any adjacent structures and underground services.

The structural wall support may be either temporary (short term) especially for soft areas or permanent depending on building construction requirements. Suitable wall types for the site could be conventional bored piles.

Such walls may be designed as fully cantilevered retaining walls socketed into the underlying soil/rock units or alternatively may rely on a combination of cantilevered and temporary ground anchors for support. Further geotechnical advice should be sought once the type of retaining wall construction is determined.

For preliminary design of soil retaining systems, either temporary or permanent, the soil properties given in Table 4-1 may be adopted. If the walls are to act as non-yielding members (i.e. restrained from lateral movement by other structural elements) a coefficient of “at rest” earth pressure Ko is recommended. If the walls are design to accommodate movement then the appropriate active (Ka) and passive (Kp) earth pressures can be adopted.

The wall design should also take into account of surcharge loads (eg. construction traffic, footing from adjoining buildings, etc) and short and long term groundwater pressures as appropriate.

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Table 4-1 Preliminary Soil and Rock Retaining Wall Design Properties

Geotechnical Units	Summary Description	Bulk Density kN/m^3	Effective Cohesion C' (kPa)	Effective Friction Angle ø' (deg)	Elastic Modulus E' (MPa)	Earth Pressure Coefficient at Rest		
						Ka	Ko	Kp
Unit 2	Fill	19	0	28	15	0.33	0.5	2.5
Unit 3a	Alluvial soil	20	0 - 5	25	10	0.40	0.6	2.8
Unit 3b	Residual Clay	20	5	28	30	0.30	0.5	3.0
Units 4A	Class IV/ V Shale	21	25	30	80	0.3	0.5	-
Unit 4B	Class III Shale	23	100	40	350	-	-	-

4.4 Foundations

4.4.1 General

Limited information is currently available on foundation loads and foundation layouts for the proposed warehouse building, however based on the current investigation data it is anticipated that the following foundation systems could be adopted:

- Shallow foundations (strip footings, pad footings, slab on grade) founded within engineered fill or alluvium/residual clay soils (Unit 2/3); and
- Piled foundation systems for heavily loaded structures (bored piers socketed into Class III shale bedrock (Unit 4b).

URS note that there are various footing options available for this site and it is recommended that the foundation design and foundation layouts be subjected to a geotechnical review once building loads and layouts are established.

4.4.2 High Level Foundations

Buildings

For lightly loaded structures not directly connected to the main buildings consideration could be given to the use of high level pad or strip footing. It is recommended that all shallow footing systems be founded a minimum of 0.6 metre deep below ground surface into the underlying Unit 2/3 clay soils (or engineered fill).

As mentioned previously, clayey soils in the area north of the realigned creek (old creek) were relatively weaker than the clayey soils to the south of the old creek corridor. Unless soil improvement or stabilisation is carried out especially in the northern area, the allowable bearing pressures for these two areas would likely to be different. At natural soil conditions, the foundation for lightly loaded structures can be designed for maximum allowable bearing pressures of 80 kPa and 150 kPa at the northern and southern areas of the site, respectively, subject to further geotechnical assessment.

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The allowable bearing capacity of the shallow foundations can be improved by means of compaction. For lightly loaded structures, a conventional approach by placing and compacting suitable engineered fill in layers to a certain depth can be chosen or for medium to heavily loaded structures, deep impact compaction and dynamic compaction can be considered.

Besides the bearing capacity of the footings, settlement criteria should also be considered especially at the northern side of the site. Consolidation settlement can be estimated by adopting the parameters provided in Table 3-5 and also additional field and laboratory testing results.

4.4.3 Piled Foundation Systems

In general, URS anticipate that bored cast-in situ concrete piles would be necessary for heavily loaded foundations for the proposed warehouse buildings. Recommended preliminary geotechnical design parameters for pile foundations are provided below in Table 4-2.

URS has interpreted the underlying shale bedrock, based on the guidelines presented in Pells et al. *“Foundations on Sandstone and Shale in the Sydney Region”*, Australian Geomechanics Journal, 1998) and provided preliminary end bearing and shaft adhesion parameters.

It should be noted that the classification provided is for design of foundations and incorporates recommended allowances for rock defects such as fracture zones and clay seams. The actual intact rock strength in some cases may be higher than the rock classification suggested, and reference should be made to the bore log when assessing the excavation characteristics of these materials.

Table 4-2 Rock Classification and Preliminary Allowable Foundation Design Parameters

Material and Classification	Approx. Depth (m below Ground level)	Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion (kPa)	Allowable Uplift Shaft Adhesion (kPa)
Unit 4A – Class IV to V	4.7 – 7.2	700	50	25
Unit 4B – Class III	7.1 – 8.5	3500	180	60

The values in Table 4-2 assume that piles are socketed a minimum of 300 mm. Shaft adhesion in the fill and overburden soils should be neglected for design of rock end bearing and socketed piles.

4.4.4 Pile Inspection and Construction

For bored pile construction, it will be necessary to use a cleaning bucket to ensure that the base of the pile is clean of drilling debris. If pile capacities rely on shaft adhesion then it will also be necessary to use a sidewall roughing tool to ensure that the design shaft adhesion values can be achieved.

All foundation excavations (including those for high level footings and piles) should be kept free of ponded water to prevent softening of the founding strata. Excavations should not be left open overnight. All footings should be excavated, cleaned, and poured with minimal delay to avoid deterioration of the bearing surface. Where appropriate side wall support/pile casing should be provided to support unstable excavation conditions are encountered.

The base of all excavations should be inspected immediately prior to foundation construction to check that loose debris has been removed.

4.5 Site Preparation and General Earthworks

The following outlines the recommended sub-grade preparation works to be carried for all areas beneath building sub-grades, pavements and areas and/or surfaces which are to receive fill. It is assumed that all site preparation and earthworks (i.e. density testing and compaction) will comply with the appropriate requirements of AS 3798-1996.

General requirements are:

- All areas are to be stripped of all topsoil and organic matter (this material should be stockpiled separately and used for landscaping purposes) and deleterious material which may prevent subsequent layers of engineered fill achieving the specified level of compaction
- Under geotechnical supervision, compact and proof roll all exposed soil surfaces with a minimum of 8 passes of a roller of at least 5 tonnes per metre width static weight to detect any soft or compressible areas. If any unacceptable materials are found, then they should be excavated and replaced with a compacted engineered fill (as specified below). Once backfilling is completed, these areas should be proof rolled in the presence of a geotechnical engineer.
- Place and compact suitable engineered fill (to achieve design ground levels) in layers of no more than 250 mm loose thickness to 98% of the standard maximum dry density (SMDD), within -2% to +2% of the optimum moisture content (OMC). Engineered fill shall preferably comprise a well graded granular material such as crushed sandstone with a maximum particle size of 100 mm. Thinner layers may be required to achieve the density specification if light weight equipment is used such as within areas of limited working room. Where vibratory equipment is used, extreme care should be exercised to minimise the risk of vibration damage to adjacent structures.
- CBR testing should be carried out in future investigation for design of new pavement.

The existing sand/clayey sand fill may be reused provided that unsuitable components are removed from the fill such as concrete. organic matter, soft materials etc. For the purpose of reusing and disposal of material, acid sulphate soil assessment requirements, please refer to our contamination assessment report.

4.6 Contiguous Pile Wall along Haslam's Creek

Contiguous pile wall connected with a capping concrete beam is present at the northern site property boundary with Haslam's Creek's bank (refer to Figure 4-1). The condition of this wall is generally in a fair condition with slightly exposed spalling concrete under the capping beam. However, no design details are available and further investigations are required to determine the As-Built details of the wall.

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Figure 4-1 Contiguous Pile Wall along Haslam's Creek

The minimum distance of this wall to the proposed excavation area is approximately 8 m at the most northern corner of the proposed building. Considering this distance and the maximum excavation depth of 3.5 m below the existing ground surface for Option 1, excavation works with a slope of 2(H):1(V) will be unlikely to result in ground movement to the existing wall.

However, if in any changes from the present options occur such that the excavation area to the contiguous pile wall position is closer than 8m and deeper than 3.5 m below the exiting ground surface, excavation supports will be required.

Conclusions and Recommendations

A preliminary geotechnical site assessment of the site at 17-21 Parramatta road has been carried out. The conclusions and recommendations of the preliminary investigation are provided below.

5.1 Conclusions

The objective of the geotechnical investigations was to provide sufficient information for the preliminary design of foundations for the proposed warehouse building. It is understood that the option being considered for the car park is a Basement carpark with the finished floor level located at RL 2.34m or approximately up to 3.5m below the existing ground surface.

Various geotechnical and groundwater water conditions, monitoring wells were installed at four locations and laboratory testing was carried out for selected soil samples

Based on the geotechnical assessment for the site, the following conclusions have been made.

- The site is close to the geological boundary between Ashfield Shale and Quarternary Alluvium associated with Haslam's Creek. The Ashfield Shale comprises black and dark grey shale and laminite of the Triassic Wianamatta Group. The alluvium comprises silty to peaty quartz sand, silt and clay, with ferruginous and humic cementation in places and shell layers.
- Six main geotechnical units were identified from the surface with depth as summarised in the table below.

Geotechnical Unit	Approx. Thickness (m)	Summary Description
Unit 1: Pavement	0 – 0.15	**(Fill) – Concrete or Bitumen**.
Unit 2: Fill	0.15 to 3.25	**(Fill) – SAND,** uniformly graded, angular, brown, fine to medium grained moist.
Unit 3a: Alluvial Soil	0.3 to 4.8	**Silty CLAY,** typically soft to firm, medium to high plasticity, with trace of sand, brown.
Unit 3b: Residual Soil	0.5 to 5.3	**CLAY,** typically stiff to very stiff, medium plasticity , reddish, greyish brown
Unit 4: Bedrock		
Unit 4a	0.5 to 3.5	**(CLASS V/ CLASS IV) Shale–** extremely low to low strength, residual soil to distinctly weathered, grey, fragmented to highly fractured
Unit 4b	>1.5	**(CLASS III/ CLASS II) Shale/laminite –** typically low to high strength, distinctly to slightly weathered, grey, slightly to highly fractured

- Clayey soils in the area north of the realigned creek (old creek) were relatively weaker than the clayey soils to the south of the old creek corridor.
- The measured groundwater levels on 10 August 2009 are at between RL 0.8m and RL 3.0m. The finished floor level of the proposed basement (RL 2.3m) is below the groundwater level in some areas, especially in the eastern area of the site.

- With consideration of long term water table rises during extended wet period, the water level is expected to rise over short periods above the measured levels.
- Bored cast-in-situ concrete piles are required for the proposed warehouse buildings.
- PASS materials were encountered at the site and an appropriate Acid Sulphate Soil Management Plan will likely be required.
- Excavations need to be battered and where required temporary excavation support is required.

5.2 Recommendations

The following general recommendations are provided for the site:

- To gain greater confidence on subsurface conditions and given access constrains, additional geotechnical investigations should be undertaken inside the existing building. Four additional deep boreholes combined with SPT are recommended.
- In designing the basement, consideration should be given to either a drained or undrained basement. For the drained basement, the long term operating costs and risks will need to be compared to the initial capital cost for construction of undrained basement. The undrained basement will need to be designed to withstand long term uplift groundwater pressure taking into account seasonal fluctuations in groundwater levels.
- Study the method of construction dewatering. This will apply for the above two options.
- Assessment on groundwater inflows should be carried out for short term and long term inflows if a drained basement is to be considered.
- Study and determine methods of site dewatering for construction. The use of sump pumps maybe feasible.
- Carry out analysis to predict groundwater drawdown induced settlements of the surrounding area. If a drained basement is considered such drawdown settlement may locally cause distress on adjacent structures particularly those founded on high level footings.

References

Geological Survey of NSW (1983), Department of Mineral Resources, Geological Series Map of Sydney, 1:100,000 Sheet 9130 (Edition 1).

Pells, P.J.N., Mostyn, G. and Walker, B.F. (1998) Foundations on Sandstone and Shale in The Sydney Region, Australian Geomechanics Dec 1998.

AS 2159 – 1995, Piling – Design and Installation.

AS 2870 – 1996, Residential Slabs and Footings - Construction

AS 3798 – 1990, Guidelines on Earthworks for Commercial and Residential Developments.

Limitations

7.1 Geotechnical Report

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 Costco Wholeshale 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 1 April 2009.

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 3 August and 30 September 2009 and is based on the conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

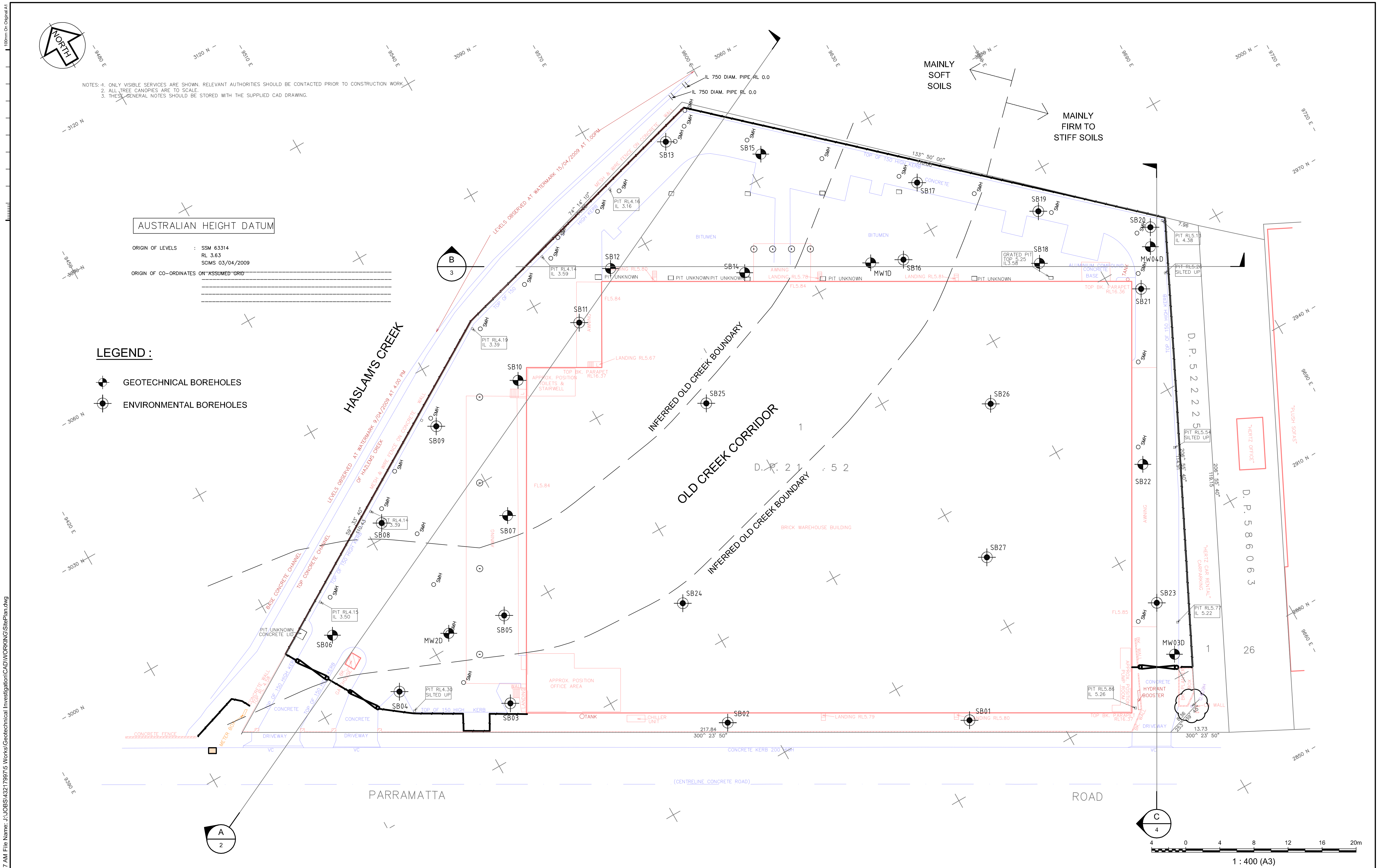
This report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

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 Drawings



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0	07.08.09	SD	VT	PRELIMINARY

Design	JK/RA	06/08/09
Drawn	SD	06/08/09
Design Check	VT/JS	06/08/09
Drafting Check	VT	06/08/09
Final Approval		

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Datum: AHD

URS

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Client: COSTCO WHOLESALE AUSTRALIA PTY. LTD.

Project: PRELIMINARY ENVIRONMENTAL AND GEOTECHNICAL SITE ASSESSMENT
17-21 PARRAMATTA RD
LIDCOMBE

Drawing Title: SITE PLAN

Status: DRAFT

Project No. 43217997

Drawing No. 1

Scale (A3): AS SHOWN

Rev: 1

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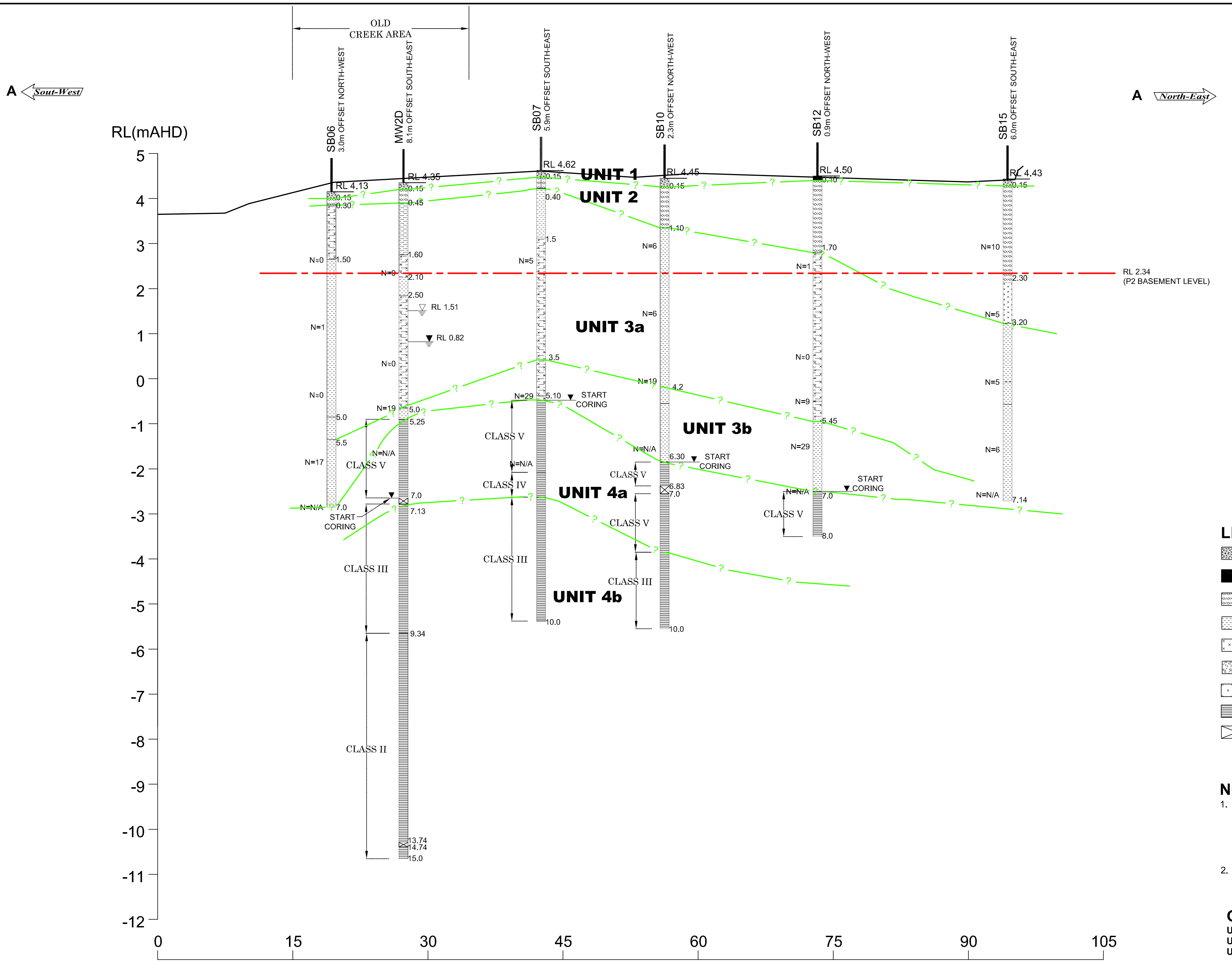

LEGEND

- CONCRETE
- BITUMEN
- FILL
- CLAY
- SILT
- SAND
- GRAVEL
- SHALE
- CORE LOSS
- GROUNDWATER LEVEL IN DEEP WELL (10 AUGUST 2009)
- GROUNDWATER LEVEL IN SHALLOW WELL (10 AUGUST 2009)

NOTES

1. THE SUBSURFACE CONDITIONS SHOWN ON THIS FIGURE ARE CONSIDERED ACCURATE ONLY AT BOREHOLE LOCATIONS. THE SUBSURFACE CONDITIONS BETWEEN THESE LOCATIONS REPRESENTS URS PRELIMINARY ASSESSMENT BASED ON AVAILABLE DATA. THE BOUNDARY BETWEEN THE VARIOUS UNITS HAS BEEN INFERRED AND SHOULD BE CONFIRMED IN FINAL DESIGN.
2. ROCK MASS CLASSIFICATION BASED ON PELLS, et al, 1998.

GEOTECHNICAL UNITS

UNIT 1 PAVEMENT SURFACE: Concrete/Bitumen road base.
UNIT 2 FILL: Sand, uniformly graded, angular, brown, moist.
UNIT 3a ALLUVIUM: Silty Clay with trace of sand, medium to high plasticity, brown, moist, soft to firm.
UNIT 3b RESIDUAL: Clay, medium plasticity, reddish, greyish brown, moist, stiff to very stiff.
UNIT 4a SHALE: Extremely low to low strength, residual soil to distinctly weathered, grey, fragmented to highly fractured (Class IV to V Shale) .
UNIT 4b SHALE: Low to high strength, distinctly to slightly weathered, grey, slightly to highly fractured (Class II to III Shale).

1 0 1 2 3 4 5m
1 : 100 VERTICAL (A3)
5 0 5 10 15 20 25m
1 : 500 HORIZONTAL (A3)

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Drawing Title: CROSS SECTION A

Project No. 43217997
Drawing No. 2
Status: DRAFT
Scale (A3): AS SHOWN
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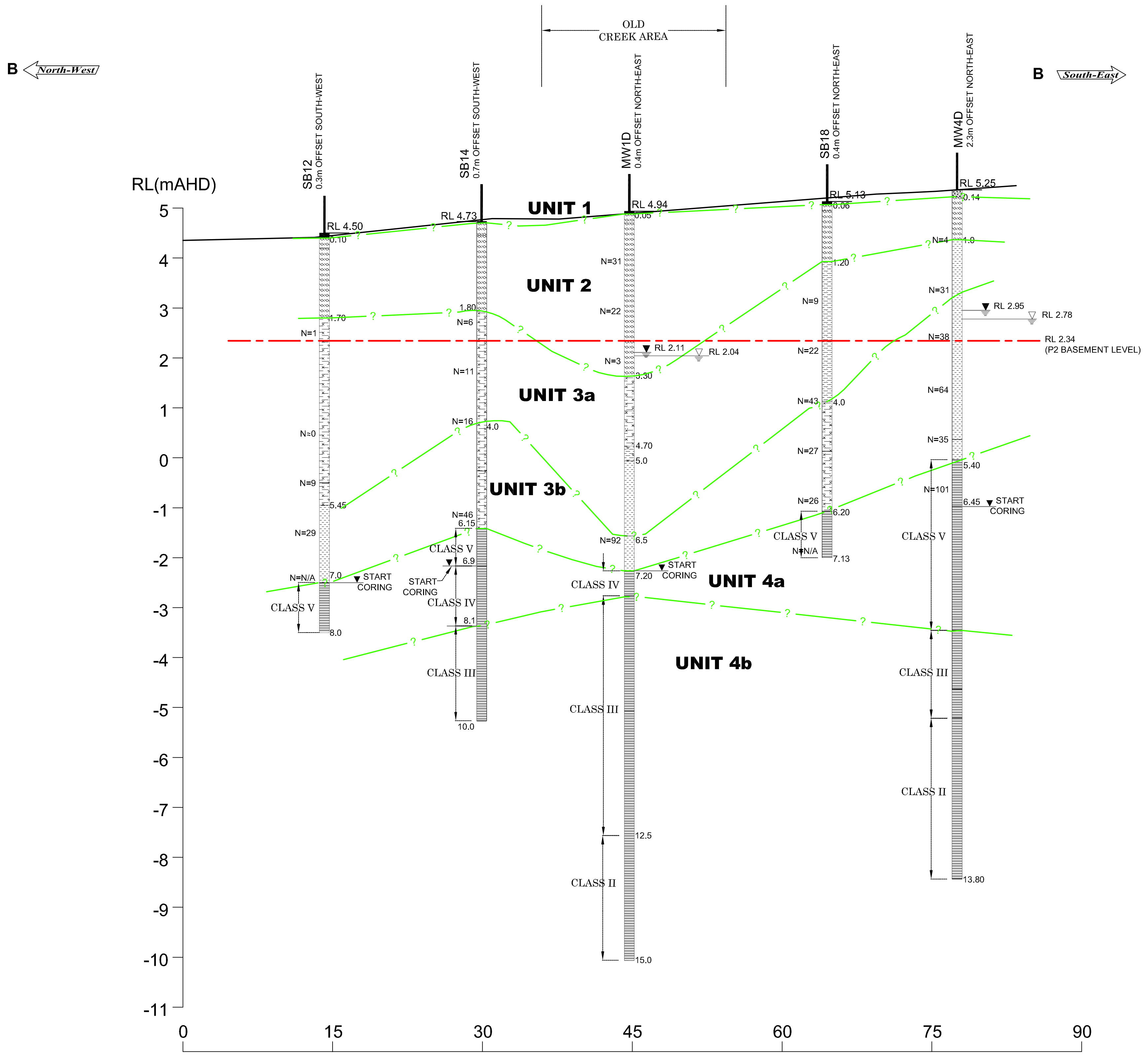

LEGEND

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1 : 100 VERTICAL (A3)

1 : 500 HORIZONTAL (A3)

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Drawing Title: CROSS SECTION B

Project No. 43217997

Drawing No. 3

Status: DRAFT

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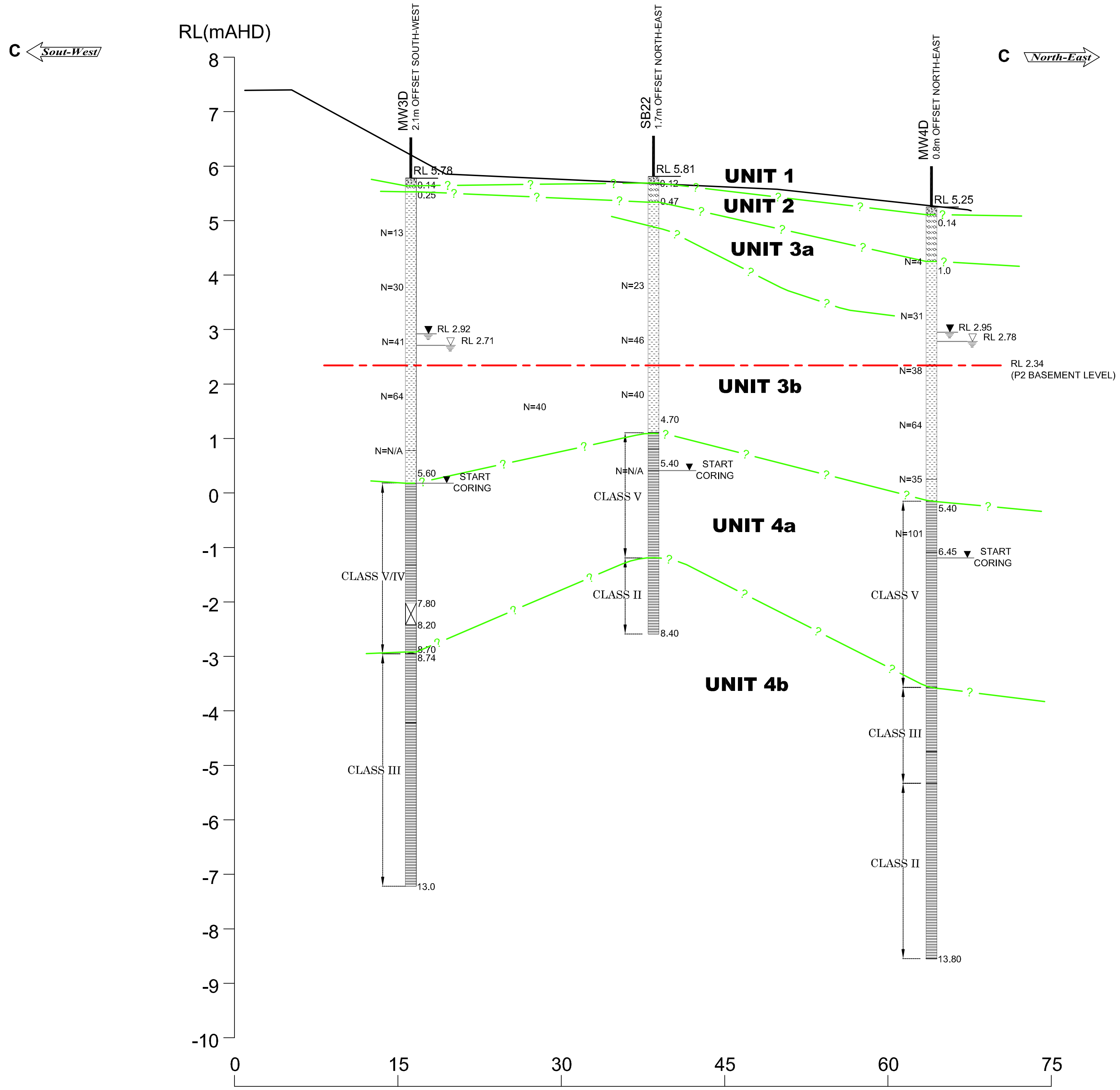

LEGEND

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- FILL
- CLAY
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Project: PRELIMINARY ENVIRONMENTAL AND GEOTECHNICAL SITE ASSESSMENT 17-21 PARRAMATTA RD LIDCOMBE

Drawing Title: CROSS SECTION C

Project No. 43217997
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Status: DRAFT
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Rev: 1

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.

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.

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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.

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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.

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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

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:

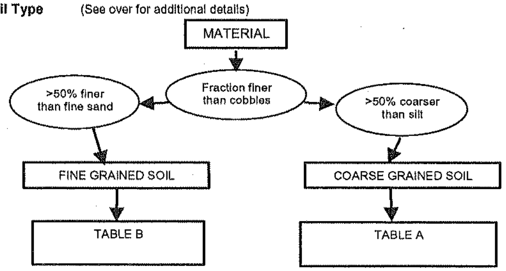
W –Widely spaced	600mm - 2m
M – Moderately spaced	200 – 600mm
C – Closely spaced	60 – 200mm
Vc – Very closely spaced	20 – 60mm
EC – Extremely closely spaced	<20mm

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

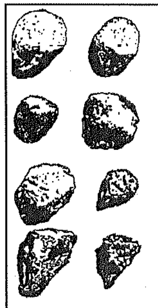
(See over for additional details)



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

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

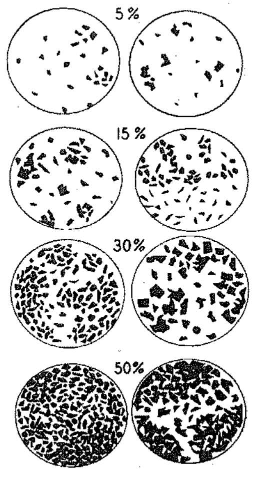
TABLE B: FINE GRAINED SOILS: more than half of the material less than 60mm is smaller than 0.06mm

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

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

MM CM 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

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