DRAFT REPORT

TO
THOMPSON BERRILL LANDSCAPE DESIGN PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED MANLY DAM LINK TRAIL

AT
MANLY DAM, KING STREET, MANLY VALE, NSW

24 October 2017
Ref: 30911ZHrpt

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GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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For and on behalf of
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1 INTRODUCTION

This report presents the results of a limited scope geotechnical investigation for the proposed Manly Dam Link Trail (‘the trail’), located adjacent to Manly Dam, King Street, Manly Vale, NSW. A site location plan is presented as Figure 1. The investigation was commissioned by Mr Andrew Zouroudis of Thompson Berrill Landscape Design Pty Ltd (TBLD) in an email sent on 25 September 2017. The commission was on the basis of our proposal (Ref P45027ZH Manly Vale) dated 29 May 2017.

We have been provided with the following:

1. A survey plan along the proposed trail alignment (Drawing Name: 11881Adetail, Issue 1, dated 29 June 2017) prepared by C.M.S. Surveyors Pty Ltd;
2. Final Draft ‘Path Concept Plans and Sections’ drawings, dated 23 October 2015, prepared by Phillips Marler;
3. Plans showing the proposed trail alignment overlaid onto the survey plan (Plan Nos. MDLT-ADDS-A to MDLT-ADDS-D, dated September 2017) prepared by TBLD. On this plan, the nominated TBLD borehole and DCP test locations are shown; and
4. A Project Brief for Geotechnical Engineering services dated May 2017, prepared by TBLD.

Based on the provided information, we understand that an approximate 420m long trail is proposed on the western side of Manly dam. The trail will be between 1.5m and 2m wide and will comprise a combination of ‘on-grade’ sections along with elevated timber boardwalks. Where the proposed trail will be on-grade and due to the sloping ground surface, some areas along the proposed trail length will need to be slightly raised (estimated to be no higher than 0.5m), with the raised areas supported by retaining walls. We do not expect any significant excavation will be required. The proposed works will also include the reconfiguration of Picnic Area 2 (located at the southern end of the proposed trail) which include new seating, picnic tables, bins, pathway and retaining walls. The approximate proposed trail alignment is shown on the attached Figure 2.

The investigation was limited by access constraints to the use of portable manually operated equipment to assess the subsurface conditions, and based on the results obtained, to present our comments and recommendations on site stability, earthworks, retaining walls and footings.

2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 3 October 2017 and was limited to the drilling of four boreholes (BH1, BH2, BH7 and BH8) using a hand auger to depths of either 0.6m (BH2 and BH7) or 0.8m (BH1 and BH7). A Dynamic Cone Penetration (DCP) test was completed at each borehole location to refusal depths of 0.56m (DCP1), 1.64m (DCP2), 2.52m (DCP7) and 0.73m (DCP8). Five additional DCP tests (DCP3 to DCP6 and DCP9) were completed to refusal depths between 0.45m (DCP6) and 0.81m (DCP4).

The test locations were completed as close as practical to the proposed trail alignment. BH7, however, was drilled to the east off the proposed trail alignment close to the dam, as access to the proposed test location further upslope was not possible due to dense bushland.

The test locations were set out by hand held GPS following an overlay of the design sketches and proposed borehole locations onto available Google Earth imagery and are shown approximately on the attached Figure 2. Figure 2 is based on a satellite image of the site.

The nature and composition of the subsoils were assessed by logging the materials recovered during drilling. The density and strength of the subsoil profile were assessed by interpretation of the DCP test results and tactile examination. We note that refusal of the DCP equipment often indicates the depth to the underlying bedrock, however, due to the equipment's limitations, it may also refuse on obstructions within fill, tree roots, 'floaters', or other 'hard' layers within the soil profile and not necessarily on bedrock. The DCP tests do not provide sample recovery. Groundwater observations were made in each borehole during the fieldwork. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineer (David Fisher) was present on a full-time basis during the fieldwork to set out the test locations, nominate the testing and sampling and prepare the attached borehole logs and DCP test results sheets. The approximate chainage of each borehole location is shown on the respective borehole logs. The Report Explanation Notes define the logging terms and symbols used.

Laboratory geotechnical testing was not carried out as it was not deemed appropriate. A contamination screen of site soils and groundwater was outside the agreed scope of this investigation.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The proposed trail alignment is located just above the toe of a gently to moderately sloping east facing hillside, which generally grades between 5° and 10°, but with locally steeper slopes to a maximum of 30°. Many Dam is located along the toe of the hillside. Sir Roden Cutler VC Memorial Drive was located within approximately 70m to the west of the proposed trail alignment.

At the time of the fieldwork, the hillside was undeveloped and comprised a bushland setting characterised by outcrops of sandstone bedrock, as well as numerous detached sandstone boulders, particularly along, and adjacent to, the central portion of the proposed trail. The sandstone bedrock was assessed to be at least medium strength. Leaf litter often covered the ground surface where the vegetation cover was sparse.

Picnic Area 2 located off the southern end of the proposed trail alignment was relatively flat and sandstone bedrock frequently outcropped in the area.

Where the underlying surface soils were exposed, these were mostly sand or gravel. Trafficability under foot was generally quite good, apart from areas located immediately adjacent to the dam at water level.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone.

The boreholes have disclosed a variable subsurface profile comprising surficial sand fill (BH1 and BH2 only) overlying alluvial and/or residual sand, sandy clay, silty sand and clayey sand with sandstone bedrock encountered, or inferred, at mostly shallow depth. We note the presence of numerous sandstone outcrops and detached boulders along, and adjacent to, the proposed trail alignment. Reference should be made to the attached borehole logs and DCP test results for detailed subsurface conditions at specific locations. A summary of the subsurface conditions as encountered is presented below:

- Fill comprising silty sand was encountered at the surface of BH1 and BH2 and extended down to depths of 0.2m (BH1) and 0.1m (BH2). Inclusions of igneous gravel were present within the fill.

- Alluvial and/or residual soils comprising sand, sandy clay, silty sand and clayey sand were encountered below the fill in BH1 and BH2 and from the ground surface in BH7 and BH8 and extended down to depths of 0.6m in BH1 and the borehole termination depths in BH2, BH7 and BH8. The sand, silty sand and clayey sand were very loose and medium dense. The sandy clay was stiff, very stiff and hard. We note that BH7 was drilled on a small beach located adjacent to the dam, whilst the other boreholes were located further upslope away from the dam. With the exception of DCP7, the other DCP test results infer medium dense to dense conditions (for a sand, silty sand or clayey sand) and very stiff to hard strengths (for a clay). Assuming DCP7 extended through similar alluvial sands, very loose conditions were indicated down to 2.3m depth, with medium dense conditions down to the DCP refusal depth.
- Extremely weathered sandstone bedrock of extremely low strength was encountered in BH1 at 0.6m. The hand auger in BH1 refused within the bedrock profile at 0.8m depth. Sandstone bedrock was inferred at the DCP refusal depths which were between 0.45m (DCP6) and 2.52m (DCP7).
- BH1, BH2 and BH8 were 'dry' during and on completion of drilling. In BH7, groundwater seepage was observed during drilling at 0.2m depth. On completion of drilling, groundwater was also measured at a depth of 0.2m in BH7. We note that groundwater levels may not have stabilised within the short observation period.

4 COMMENTS AND RECOMMENDATIONS

4.1 Site Stability

The site is located on a gently to moderately sloping hillside slope, which appears to be well drained. Sandstone bedrock outcrops on the site or is otherwise inferred at relatively shallow depth. Our observations indicated no recent deep seated hillside instability, nor is such instability expected due to the shallow bedrock. Further, there was no indication of any shallow creep movements.

Based on the investigation results, we consider that construction of the proposed trail feasible from a geotechnical perspective, provided the comments and recommendations below are adopted in their entirety

4.2 Earthworks

All earthworks recommendations should be complemented by reference to AS3798-2007 ("Guidelines on Earthworks for Commercial and Residential Developments").

4.2.1 Site Preparation

For the proposed on-grade trail areas, we recommend that all vegetation including leaf litter, topsoil (if present), root affected soils and any deleterious or contaminated existing fill be stripped from the surface. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may, however, be reused for landscaping purposes. Reference should be made to Section 5 below for guidance on the off-site disposal of soil.

Excavation down to design subgrade levels, if any, will extend through the fill and natural soil profile, in areas where the rock does not outcrop, and may be completed using a bucket fitted to a hydraulic excavator. Should bedrock need to be excavated, then use a rock hammer will be required.

Based on the investigation results and noting that the proposed trail alignment will be located well upslope from the edge of the dam, we do not expect any significant seepage into the proposed excavations. If seepage does occur, we expect it expect it will be localised, of limited volume and readily controlled by gravity drainage down to the dam.

4.2.2 Subgrade Preparation

Following stripping, the exposed soil subgrade where the on-grade trail is proposed, should be proof rolled with at least six passes of a small non-vibratory smooth drum roller of at least two tonnes deadweight. Proof rolling will assist in improving the near surface compaction of the soils and identifying any ‘soft’ or unstable subgrade. The final pass of proof rolling should be carried out under the direction of a geotechnical engineer.

Subgrade heaving during proof rolling is expected to occur where the underlying clay soils have become ‘saturated’. Heaving areas should be locally removed down to a stable base and replaced with engineered fill, as outlined in Section 4.1.3 below. Possible alternatives to stripping the full depth of the heaving areas must be provided by the geotechnical engineer during the proof rolling inspection, if appropriate.

Engineered fill must be used where site levels need to be raised.

4.2.3 Engineered Fill

Material suitable for engineered fill comprises a good quality well graded granular material such as imported crushed sandstone, on condition the material is 'clean' and free of organic matter and particles greater than 40mm size. Engineered fill should be placed and compacted in maximum 150mm loose thickness layers and compacted to achieve a minimum density ratio of 95% of Standard Maximum Dry Density (SMDD).

Density tests should be carried out on the engineered fill to confirm the above specification is achieved in accordance with Table 8.1 in AS3798-2007. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. Due to a potential conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by Council (or their representative) and not by the earthworks contractor or sub-contractors.

All engineered fill should be either retained or, alternatively, battered to a permanent slope no steeper than 1 Vertical (V) on 2 Horizontal (H). A flatter batter to 1V on 3H or even 1V on 4H may be preferred in order to facilitate maintenance. All permanent fill batter slopes must be protected from erosion by quickly establishing a vegetative cover etc.

4.3 Retaining Walls

The proposed retaining wall should be designed using the following parameters:

- As sandstone bedrock is expected at relatively shallow depth in areas where the rock does not outcrop, all retaining wall footings should be uniformly founded in the underlying sandstone bedrock. For allowable bearing pressure recommendations, refer to Section 4.4 below.
- For free-standing cantilever walls supporting the on-grade trail portions, a triangular lateral earth pressure distribution may be adopted with an 'active' earth pressure coefficient, K_a, of 0.35, for the soil profile, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.
- Any surcharge affecting the walls (e.g. construction loads, compaction stresses during backfilling, inclined backfill etc) should be allowed for in the design using the appropriate earth pressure coefficient from above.
- The retaining walls should be designed as drained and measures taken to induce complete and permanent drainage of the ground behind the wall. Subsurface drains should incorporate a non-woven geotextile filter such as Bidim A34 to control subsoil erosion.

- Lateral toe restraint for the retaining walls constructed in areas where the rock outcrops can be secured into the underlying bedrock using dowels. Permanent dowels must designed for corrosion resistance and long term durability. For a gravity wall, an effective friction angle of 35° may be adopted over the bedrock surface.

4.4 Footings

Based on the investigation results, we expect that along the proposed trail alignment and in the vicinity of Picnic Area 2, sandstone bedrock to outcrop, or otherwise be present at relatively shallow depth. Furthermore, the proposed trail is located on the side of a gently to moderately sloping hillside. Therefore, for uniformity of support and for stability reasons, we recommend that all footings be founded within the underlying sandstone bedrock.

Pad/strip footings founded in sandstone bedrock should be designed for a maximum allowable bearing pressure of 600kPa.

Due to the sloping site and presence of detached boulders, we recommend that all footing excavations be inspected by a geotechnical engineer, prior to pouring, to confirm that a satisfactory bearing stratum has been achieved.

Footings located directly behind the crest of a ‘step’ down in the bedrock surface should be inspected by a geotechnical engineer to identify any adverse defects or presence of poor quality bedrock that may require stabilisation (eg. shotcrete, rock bolts etc).

All footings should be excavated, cleaned out, inspected, ’dry’ and poured with minimal delay.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed test locations may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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BOREHOLE LOG

Borehole No. **1**

1/1

APPROXIMATE CHAINAGE: 0m

Client: THOMPSON BERRILL LANDSCAPE DESIGN PTY LTD

Project: PROPOSED MANLY DAM LINK TRAIL

Location: MANLY DAM, KING STREET, MANLY VALE, NSW

Job No. 30911ZH **Method:** HAND AUGER **R.L. Surface:** ≈ 36.0m

Date: 3-10-17 **Datum:**

Logged/Checked by: D.A.F./A.J.H.

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION		REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown and light grey, trace of roots and fine to medium grained igneous gravel.	M			
			0.5		CL	SANDY CLAY: low plasticity, light orange brown, with fine to medium grained ironstone gravel.	MC<PL	(H)		RESIDUAL TOO FRIABLE FOR HP TESTING
					-	SANDSTONE: fine to medium grained, light grey and light orange brown.	XW	EL		
			1			END OF BOREHOLE AT 0.8m				HAND AUGER REFUSAL
			1.5							
			2							
			2.5							
			3							
			3.5							

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BOREHOLE LOG

Borehole No. **2**
1/1

APPROXIMATE CHAINAGE: 50m

Client: THOMPSON BERRILL LANDSCAPE DESIGN PTY LTD
Project: PROPOSED MANLY DAM LINK TRAIL
Location: MANLY DAM, KING STREET, MANLY VALE, NSW

Job No. 30911ZH **Method:** HAND AUGER **R.L. Surface:** ≈ 37.5m
Date: 3-10-17 **Datum:** AHD
Logged/Checked by: D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0			FILL: Silty sand, fine to medium grained, dark brown and light grey, trace of fine to medium grained igneous gravel.	M			
								CL	SANDY CLAY: low plasticity, light orange brown, with fine to medium grained ironstone gravel.	MC<PL	H		RESIDUAL TOO FRIABLE FOR HP TESTING
						0.5							
									END OF BOREHOLE AT 0.6m				HAND AUGER REFUSAL ON IRONSTONE GRAVEL
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No.
7
1/1

APPROXIMATE CHAINAGE: 360m

Client: THOMPSON BERRILL LANDSCAPE DESIGN PTY LTD
Project: PROPOSED MANLY DAM LINK TRAIL
Location: MANLY DAM, KING STREET, MANLY VALE, NSW

Job No. 30911ZH **Method:** HAND AUGER **R.L. Surface:** ≈ 34.4m
Date: 3-10-17 **Datum:** AHD
Logged/Checked by: D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
ON COMPLETION					REFER TO DCP TEST RESULTS	0		SP	SAND: fine to coarse grained, light grey and grey, trace of silt fines.	M	VL		ALLUVIAL
										W			
						0.5							
									END OF BOREHOLE AT 0.6m				BOREHOLE TERMINATED DUE TO SIDES CONTINUALLY COLLAPSING
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No. **8**
1/1

APPROXIMATE CHAINAGE: 385m

Client: THOMPSON BERRILL LANDSCAPE DESIGN PTY LTD
Project: PROPOSED MANLY DAM LINK TRAIL
Location: MANLY DAM, KING STREET, MANLY VALE, NSW

Job No. 30911ZH
Method: HAND AUGER
R.L. Surface: ≈ 35.5m
Date: 3-10-17
Datum: AHD
Logged/Checked by: D.A.F./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS	0		SM	SILTY SAND: fine to medium grained, brown and light grey.	M	MD		ALLUVIAL
								SP	SAND: fine to coarse grained, light orange brown.				
								CL	SANDY CLAY: low plasticity, grey and brown.	MC>PL	(St-VSt)		
						0.5		SC	CLAYEY SAND: fine to coarse grained, light brown and orange brown.	M	MD		RESIDUAL
									END OF BOREHOLE AT 0.8m				HAND AUGER REFUSAL ON INFERRED SANDSTONE BEDROCK
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THOMPSON BERRILL LANDSCAPE DESIGN		
Project:	PROPOSED MANLY DAM TRAIL LINK		
Location:	MANLY DAM, KING STREET, MANLY VALE, NSW		
Job No.	30911ZH	Hammer Weight & Drop: 9kg/510mm	
Date:	3-10-17	Rod Diameter: 16mm	
Tested By:	D.A.F.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL≈36.0m	RL≈37.5m	RL≈36.4m	RL≈35.3m	RL≈35.4m	RL≈36.2m	RL≈34.4m
Depth (mm)	**1**	**2**	**3**	**4**	**5**	**6**	**7**
0 - 100	8	9	3	1	2	3	1
100 - 200	12	17	8	13	5	15	2
200 - 300	6	10	6	8	11	21	1
300 - 400	10	11	6	6	17	17	
400 - 500	26	9	8	11	17	24/50mm	1
500 - 600	24/60mm	8	10	21	14	REFUSAL	
600 - 700	REFUSAL	8	31	14	14		
700 - 800		9	20/50mm	20	5/10mm		
800 - 900		13	REFUSAL	6/10mm	REFUSAL		
900 - 1000		12		REFUSAL			
1000 - 1100		8					
1100 - 1200		11					
1200 - 1300		6					1
1300 - 1400		9					
1400 - 1500		9					
1500 - 1600		13					1
1600 - 1700		10/40mm					
1700 - 1800		REFUSAL					1
1800 - 1900							1
1900 - 2000							1
2000 - 2100							1
2100 - 2200							1
2200 - 2300							2
2300 - 2400							6
2400 - 2500							7
2500 - 2600							8/20mm
2600 - 2700							REFUSAL
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client:	THOMPSON BERRILL LANDSCAPE DESIGN		
Project:	PROPOSED MANLY DAM TRAIL LINK		
Location:	MANLY DAM, KING STREET, MANLY VALE, NSW		
Job No.	30911ZH	Hammer Weight & Drop: 9kg/510mm	
Date:	3-10-17	Rod Diameter: 16mm	
Tested By:	D.A.F.	Point Diameter: 20mm	

	Number of Blows per 100mm Penetration						
Test Location	RL≈35.5m	RL≈35.5m					
Depth (mm)	**8**	**9**					
0 - 100	12	6					
100 - 200	10	8					
200 - 300	6	8					
300 - 400	5	19					
400 - 500	7	14					
500 - 600	8	8					
600 - 700	15	12/50mm					
700 - 800	12/30mm	REFUSAL					
800 - 900	REFUSAL						
900 - 1000							
1000 - 1100							
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal
3. Survey datum is AHD

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

This plan should be read in conjunction with the JK Geotechnics report.

Title: **SITE LOCATION PLAN**

Location: MANLY DAM
KING STREET, MANLY VALE, NSW

Report No: 30911ZH

Figure No: 1

JK **Geotechnics**

PLOT DATE: 24/10/2017 2:47:49 PM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30911Z MANLY VALE\CAD\30911ZH.DWG

LEGEND

- HAND AUGER AND DCP TEST
- DCP TEST
- PROPOSED TRAIL ALIGNMENT

0 12 24 36 48 60

SCALE 1:1200 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	TEST LOCATION PLAN
Location:	MANLY DAM KING STREET, MANLY VALE, NSW
Report No: 30911ZH	Figure No: 2

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PLOT DATE: 24/10/2017 1:11:08 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30911Z MANLY VALE\CAD\30911ZH.DWG

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which 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 or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

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 samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), 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 shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

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

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu 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 after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm 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 supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) 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 1289, "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.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N>30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the 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 SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT 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 SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using a Cone Penetrometer Test (CPT). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- 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.

  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. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 represent only a very small sample of the total subsurface conditions.

The attached explanatory notes define the terms and symbols used in preparation of the logs.

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. 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 it 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 will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. 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 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 the 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 is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

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, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

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

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. 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 or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a 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 engineer.

SITE INSPECTION

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

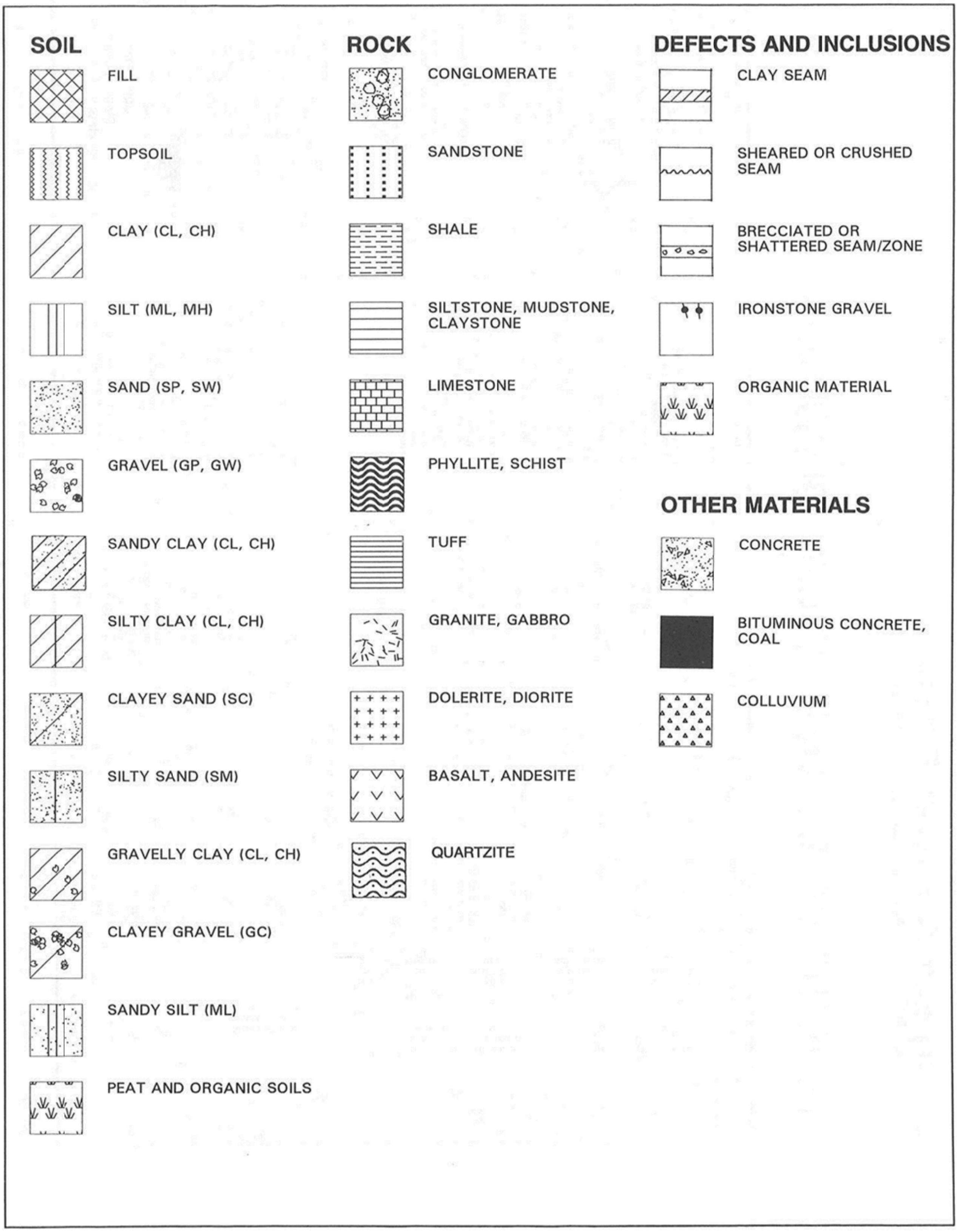
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) 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

iii) full time engineering presence on site.

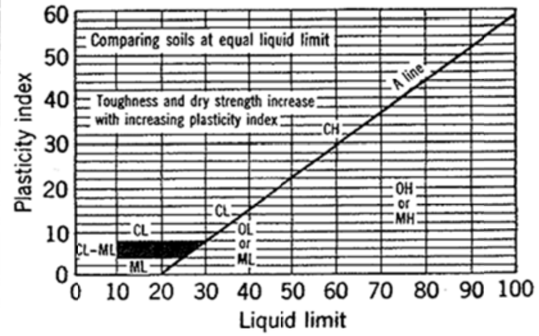
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



Major Division			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line, or *PI* less than 4. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures			Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines			$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or *PI* less than 5. Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits below "A" line with *PI* greater than 7

Identification Procedures on Fraction Smaller than 380 μm Sieve Size

Major Division		Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions. Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶—	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY – Runs freely through fingers.
	M	MOIST – Does not run freely but no free water visible on soil surface.
	W	WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – Unconfined compressive strength less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	HARD – Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	Very Dense >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	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 rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	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 rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
----------------------	-------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
----------------------	-------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
----------------------	-------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
----------------------	-------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
----------------------	--------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
----------------------	--------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	