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REPORT

TO

CROWN INTERNATIONAL HOLDINGS GROUP

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT

AT

EVAN AVENUE, EASTLAKES, NSW

31 May 2004 **Ref: 18602VBrpt**

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed BKK Shopping Centre redevelopment at Evans Avenue, Eastlakes, NSW. The investigation was commissioned by Mr R J Strahle of Crown International Holdings Group and was carried out in accordance with our proposal, Ref: P10200VB.

The redevelopment of the BKK Shopping Centre is currently at the feasibility stage and at this stage, only a brief concept of the redevelopment is known. The redevelopment site comprises the existing shopping centre located between Evans Avenue and Barber Avenue and the area on the northern side of Evans Avenue bounded by Gardeners Road and Racecourse Place (as shown on Figure 1). It is envisaged that the development may involve 2 to 3 basement levels for car parking and as such excavations may be required to depths of about 6m to 9m. At this feasibility stage, structural loads had not been determined.

The scope of the investigation was limited to obtaining geotechnical information on subsurface conditions at four locations as a basis for comments and preliminary recommendations on the proposed development concept, including excavations, retention, groundwater and footings.

2 INVESTIGATION PROCEDURE

Boreholes (BHs) 1 to 4 were auger drilled using our truck mounted JK450 rig to depths between 15.01m and 15.05m below the existing pavement surface. Prior to drilling, the borehole locations were scanned for underground services using electronic service detection equipment. The borehole locations, as shown on Figure 1, were set our by taped measurements from existing surface features and inferred site boundaries.

The approximate surface levels of BHs 1 and 2, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plan by Dunlop Thorpe & Co Pty Ltd (Ref: 10116-2, dated 18/2/00). The datum of the levels is Australian Height Datum (AHD). Estimation of the surface levels of BHs 3 and 4 was not possible as the detailed survey did not extend onto the northern side of Evans Avenue.

The relative density of the subsurface sands was assessed with reference to Standard Penetration Test (SPT) 'N' values and Solid Cone Penetration 'N_c' values, together with static auger pushes of the auger string by the hydraulics of the rig without rotation of the augers.

Groundwater observations were made both during drilling and following completion of the boreholes for the two days that our field crew was on site. Slotted PVC standpipes were installed in BHs 1 and 4 on completion of drilling to allow future measurements of groundwater levels. A groundwater measurement was made within the standpipe installed in BH1, at 24 hours following completion of drilling. Longer term monitoring of groundwater levels was not carried out.

Our geotechnical engineer, Mr N Smith, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, which include field test results and groundwater observations, are attached to this report together with a set of explanatory notes, which describe the investigation techniques and their limitations and define the logging terms and symbols used. Laboratory testing of the site soils, including for soil contamination, was outside the agreed scope of this preliminary investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within gently sloping topography and generally slopes down towards the south-west at about 2° to 3°. The site comprises essentially three properties on either side of Evans Avenue, which divides the site.

The main portion of the site is located on the southern side of Evans Avenue and is bounded by Barber Avenue to the east and south and Eastlakes reserve to the west. At the time of the fieldwork, this area of the site was occupied by a single storey brick and concrete shopping centre, with a single partial basement level towards the south-western corner of the site. A separate, single storey, brick, McDonald's building was located in the north-western corner of this portion of the site.

On the northern side of Evans Avenue, occupying the eastern portion, was an asphaltic concrete (AC) paved car park with a single storey brick building, containing a row of shops, along the northern boundary with Gardeners Road. Within the north-western corner of the site, bounded by Gardeners Road, Racecourse Place and Evans Avenue, were two, four storey, brick apartment buildings, surrounded by concrete pathways and grassed areas.

All buildings on the site generally appeared to be in good external condition. The site was surrounded by residential developments of up to 4 storeys in height, which also appeared to be in good external condition when briefly viewed from within the subject site.

3.2 Subsurface Conditions

In summary, the boreholes encountered AC pavements and fill covering a deep natural sand profile. Bedrock was not encountered within the maximum borehole depth of about 15m. Further comments on the pertinent details of the subsurface conditions are provided below. A graphical summary of the borehole information is presented as Figure 2. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered.

Pavements and Fill

All boreholes penetrated AC pavements with AC of 50mm thickness. Fill was encountered below the AC to depths between 0.3m and 1.2m. The fill comprised sandy gravel and sand. Based on the SPT N values the fill was assessed to be moderately to well compacted.

Natural Sands

Natural sands were encountered below the fill to the limit of the boreholes at depths of about 15m. Initially, in BHs 1, 2 and 4, the sands were of medium dense relative density and in BH3, of very loose relative density. In BH3, sands of medium dense relative density were encountered at a depth of 2.5m. The relative density of the sands generally increased with depth, with sands of dense or very dense relative density encountered at depths between 4.5m and 7.5m. However, bands of lower relative density ranging from very loose to loose to medium dense were encountered at depths between 10.5m and 12m, which extended to depths between 12.3m and 13.5m. Below these depths of 12.3m to 13.5m, the sands were of at least medium dense to dense relative density.

Groundwater

Groundwater seepage was encountered during drilling at depths between 3m and 4m. Boreholes 2 and 3 collapsed on completion at depths of 1.0m and 1.4m and further measurements of groundwater levels were not possible. Groundwater measurements were taken within the PVC standpipes installed in BHs 1 and 4. In BH1, groundwater was measured at a depth of 4.0m on completion of drilling and at a depth of 2.5m, at 24 hours following completion. In BH4, groundwater was measured at a depth of 3.9m on completion of drilling.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

Based on the results of this preliminary geotechnical investigation the following geotechnical issues have been identified, which will need to be considered in design and construction of the proposed redevelopment. Further comments on these issues are provided within the subsequent sections of this report.

- The site is underlain by a deep profile of sands and any excavations within the sands will not be self supporting and a full depth retention system will be required. Such a retention system will need to take into consideration the relatively shallow groundwater levels and any dewatering required and may be require a 'tanked' basement or deep cut-off walls to reduce groundwater flow into the excavation.
- Groundwater was encountered at depths of 2.5m and any excavations for basement construction will encounter the groundwater within the highly permeable sands. Dewatering may be required to drawdown the groundwater levels and allow construction of the basement and the basement will need to be designed as a tanked basement to resist the hydrostatic uplift pressures. However, any dewatering must be carried out with care to reduce the impact on the adjoining properties.
- The sands were generally of medium dense to very dense relative density, but bands of very loose to loose relative density sands were encountered at depths of 10.5m to 12m. If basement excavations are undertaken to depths of about 6m to 9m, these lower density bands will be present at shallow depths below the bulk excavation level. Therefore, the use of shallow footings would not be

feasible as only very low bearing pressures would be appropriate and settlements of such footings may be excessive. As such, piled footings will be required, founded below the lower density bands and within the better quality sands.

Consideration could be given to limiting the development to only 1 basement level to reduce the difficulties detailed above from the shallow groundwater level and loose relative density bands. This may then not require extensive dewatering and design of a tanked basement and allow the use of shallow footings as the lower density bands will be a sufficient depths to not affect the bearing pressures and settlements of shallow footings.

The comments provided in this report are based on the results of the four boreholes drilled as part of this preliminary investigation, which only provide a very broad coverage of the site. In addition, the project is currently at feasibility stage and the exact details of the development have not been determined. Detailed geotechnical investigations will be required as part of the final design and the comments provided in this report should be reviewed, and probably amplified, following the detailed investigation once the exact details of the proposed development are known.

We believe that a meeting of the design team would be of benefit, once the concept design is further advanced, in order to discuss the geotechnical problems and solutions in more detail and the scope of the detailed geotechnical investigations. Further comments on additional geotechnical investigations are provided in Section 4.7 below.

4.2 Excavations

Excavations for basement levels may be required to depths of about 6m to 9m. Such excavations are anticipated to encounter surface fill and natural sands and excavation of these soils should be achievable using conventional earthmoving equipment, such as hydraulic excavators. However, the sands will not be self supporting and will need to be formed at appropriate batters or a retention system installed prior to the start of excavations, as detailed in Section 4.4 below.

Prior to the start of excavations, it is recommended that detailed dilapidation reports be prepared on adjoining buildings located within a distance of 2H of the excavation perimeter (where H is the depth in metres of the proposed excavations). Due to the roadways that surround the site and the reserve to the west, dilapidation reports may only be required on the buildings to the east of the portion of the site on the northern side of Evans Avenue. The respective owners of the buildings should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. We can complete these dilapidation reports if you wish to commission us.

Excavation recommendations provided in this report should be complemented by reference to the Code of Practice Excavation Work, Cat. No. 312 dated 31 March 2000, by WorkCover NSW.

4.3 <u>Groundwater</u>

Groundwater seepage was encountered during drilling at depths between 3m and 4m and was measured within the standpipe in BH1 at a depth of 2.5m, 24 hours following drilling. Therefore, it is expected that groundwater will be encountered within the excavations and will be above the lower basement level.

Temporary dewatering could be carried out to lower the groundwater level during construction using a ring of well points. However, protracted dewatering may cause settlement of the overlying sands due to the drop in groundwater levels, which may cause problems to adjoining buildings supported on high level footings. Impermeable cut-off walls could be installed around the excavation perimeter to reduce the dewatering required outside the site, but such walls would need to penetrate through the permeable sands and into the underlying lower permeability material, such as the sandstone bedrock. The depth to the sandstone bedrock is in excess of 15m as it was not encountered within our boreholes.

It is recommended that a detailed groundwater model be determined to assess the extent of dewatering required, estimates of final drawdown and subsequent settlements below the footings of adjoining buildings. Details of the footings of the adjoining buildings will be required as part of this assessment.

It is recommended that further measurements of the groundwater levels within the standpipes be made to assess the long term groundwater levels. Research into any other nearby basements may also provide information on likely groundwater levels, long term fluctuations, seepage flows, etc.

4.4 <u>Retention</u>

It is expected that given the anticipated depths of the basement excavations, insufficient space will be available for the formation of temporary batters and the excavations will need to be supported by a full depth retention system installed prior to the start of bulk excavations. Nevertheless, where space permits, temporary batters to depths of 2m to 3m may be formed at slopes not steeper than 1 Vertical in 1.75 Horizontal (1V:1.75H), provided no groundwater is encountered. Much flatter batters and additional support would be required where groundwater was encountered. These batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well away from the crest of the batters.

Any necessary retention systems will need to be installed prior to the start of bulk excavations. It should be appreciated that no matter what method of excavation and support system is used, some adjacent ground displacement will occur within the area of influence of the excavation. The objective of properly engineered shoring and retention systems is to keep the movements within tolerable limits. The actual wall movements are highly dependant on the construction sequence, detailing and quality of installation, and should be closely monitored in critical areas. The extent of significant influence can be defined as extending a horizontal distance from the excavation perimeter equal to at least twice the depth of the excavation. Hence, any adjoining structures or buried services that fall within this area of influence should be assessed for risks of damage due to the excavation induced movements.

Similarly, during excavations care must be taken to ensure that the excavations do not undermine or render unstable the footings of any adjoining structures. If such structures have high level footing within the influence zone of the excavation, then these may have to be underpinned prior to proceeding with the excavation. The levels of the footings and basement levels of any adjoining buildings should be investigated to assess the need for underpinning.

The magnitude of lateral movements is directly related to the stiffness of the retention system. To reduce yielding and settlement of the adjacent ground surface or property, we recommend that the retaining walls be made as stiff as possible by providing lateral support by anchors or bracing. Temporary lateral support may be provided by temporary anchors, with permanent lateral support provided by the floor slabs within the excavation.

The following retention systems may be considered for the vertical excavations:

- Diaphragm walls in the form of concrete cast-in-place in a trench supported by bentonite slurry. Such walls are practically impermeable if properly constructed and may be used as cut-off walls to reduce the amount of dewatering required.
- Contiguous or secant pile retaining walls formed by grout injected piles. The use of bored piles is not recommended due to construction difficulties of caving conditions and groundwater inflow. Secant pile walls are more impermeable than contiguous pile walls as they are formed by overlapping piles. All gaps in contiguous walls would need to be filled with mortar or shotcrete to prevent soil loss. This would be more permeable than diaphragm walls and will, therefore, result in the greatest drawdown of the water table and hence, potentially greater adjoining ground settlement.
- Low energy installation, low vibrations, interlocking steel sheet pile walls (e.g. the McDonald Sheet Pile system). The use of conventional sheet piling is not recommended as the installation of sheet piles may cause damage to adjoining structures or services due to ground vibrations. If this method is considered, it is recommended that the specialist contractor be contacted to obtain their assurance that their system can satisfactorily be used in the site conditions without damaging nearby structures or services.

To prevent boiling or lifting of the sands at the base of the proposed excavations, the retaining walls should be embedded at least 3m below the base of the excavations, including excavations for footings or services.

Propped or anchored walls may be designed using a uniform rectangular earth pressure of magnitude 6H kPa (where H equals the retained height in metres) where some adjacent ground movements are tolerable and neighbouring buildings or movement sensitive services are located beyond a distance of 2H of the wall. Where buildings or services are located within 2H of the wall, the uniform rectangular earth pressure distribution should be increased to 9H kPa. These nominated lateral earth pressures are for horizontal backfill surfaces and where inclined backfill is proposed the lateral earth pressures would need to be increased or the inclined backfill taken as a surcharge load.

All surcharge loads should be allowed for in the design. Full hydrostatic pressures should be considered unless measures are undertaken to provide complete and permanent drainage of the ground behind the wall.

The passive toe resistance of the walls may be estimated using a passive earth pressure coefficient, K_p, of 3 for loose sands, 3.5 for medium dense sands or 4 for dense or very dense sands (but with a Factor of Safety of at least 2). These resistance values assume that excavation is not carried out within the zone of influence of the wall toe. The passive resistances should be disregarded for the upper 0.5m below the excavation level (including footing and service excavations).

Lateral stability of the retaining walls may be provided by sand anchors, props or by adopting a staged top-down type construction with temporary lateral support provided by the passive resistance of the unexcavated material and permanent support provided by the building slabs. An effective friction angle of 30° may be adopted for the design of anchors with a bond into medium dense sands and 34° for anchors with a bond into dense sands. Uncased anchor holes drilled into the sands will almost certainly collapse and temporary casing would be required or the use of anchors that do not require open holes. Permission will need to be obtained from owners of adjoining properties before installation of anchors below their properties.

The anchor design and construction should also satisfy the following conditions:

- Anchor bond length of not less than 3m beyond the 'active' zone of the retention system.

- Overall stability, including anchor group interaction, is satisfied.
- All anchors are respectively proof loaded to at least 1.3 times the design working load before locking off at working load.

4.5 Footings

Following bulk excavations to depths of about 6m to 9m, it is expected that sands ranging from medium dense to very dense would be encountered. However, given the expected loads of the structure with 2 to 3 basement levels and at least 1 or 2 above ground levels, shallow footings founded within the sands are expected to result in large footing sizes being required and excessive differential settlements. In addition, bands of very loose to loose sands may be present at shallow depths below the excavation level (encountered at depths between 10.5m and 12m) which would limit the applicable allowable bearing pressures for shallow footings. Therefore, it is recommended that the proposed structure be supported on piles founded within sands of at least medium dense relative density.

If the use of shallow footings is desired and their feasibility is to be fully assessed, further analysis would be required to estimate bearing pressures, footing sizes and settlements based on the footing layout and column loads once these have been determined.

Conventional bored piles are not recommended due to the collapsing nature of the sands and grout injected, displacement type piles or steel screw piles are recommended. Steel screw piles will need to be designed for adequate corrosion resistance.

Piles would need to penetrate to depths below the lower density bands (encountered to depths between 12.3m and 13.5m) in order to found within sands of medium dense to dense relative density. Otherwise, only very limited bearing pressures could be adopted due to these lower density layers. Therefore, the piles would need to be founded at depths of at least 4m to 5m below the bulk excavation level.

The allowable end bearing pressure of piles is dependant on the depth of pile embedment and the pile diameter. As a guide, for a 0.5m diameter pile founded within medium dense to dense sands at an embedment depth of at least 4m below the bulk excavation level, an allowable end bearing pressure of 800kPa would be appropriate. Allowable shaft frictions of 5kPa and 10kPa may be adopted through loose and medium dense sands, respectively. The final allowable bearing pressures for the design of piles will depend on the pile diameters and embedment depths adopted.

Alternatively, if the designer wishes to adopt the limit state design methods of the piling code, AS2159-1995, then the ultimate values of end bearing and shaft frictions may be estimated by multiplying the above recommended allowable values by Factors of Safety of 3 and 2, respectively. We recommend that ultimate bearing values be multiplied by a geotechnical reduction factor, Φ_g, of 0.45 and an appropriate load factor be applied in the design.

Further testing with an Electric Friction Cone Penetrometer (EFCP) may justify the use of higher end bearing and shaft friction values, which are based on SPT values. However, EFCP testing may be limited due to the very dense sand bands, as discussed in Section 4.6 below.

Alternatively, the proposed building may be supported on piles founded within the underlying sandstone bedrock. The depth and the quality of the bedrock are not known as boreholes have not been drilled to sufficient depths to encounter the bedrock. Therefore, further boreholes would be required to reach the sandstone bedrock, with coring of the bedrock required once encountered to assess the rock strength and quality.

4.6 Basement Floor Slabs

Since the basement will be constructed below the groundwater table, it is recommended that the basement be designed as a 'tanked' basement, where the floor slab is designed to resist hydrostatic uplift pressures. An underfloor drainage system is not recommended as it is likely to result in continual, heavy pumping that may cause permanent and significant depression of the groundwater table in surrounding properties, which may result in significant ground settlements.

4.7 Further Geotechnical Investigations

As detailed above, this preliminary geotechnical investigation only provides a broad coverage of the site and a detailed geotechnical investigations will be required as part of the final design. Since the subsurface profile comprises sands, it is preferable that EFCP testing be carried out as part of the detailed investigation.

EFCP testing involves pushing a 35mm diameter rod with a cone tip into the soil using the hydraulic rams of a purpose built rig. This provides a near continuous profile of the resistance of the soils on the cone tip and friction on a sleeve mounted behind the tip. The subsurface material identification, including material strength/relative density, is assessed by interpretation of the test results based on past experience, empirical correlations and comparison with the borehole logs. Further details of the test method are provided within the attached Report Explanation Notes.

Such EFCP testing would provide greater detail of the variations in the relative density of the sands and in particular the thickness and extent of the lower density layers encountered within the boreholes at depths between 10.5m and 12m. However, since bands of very dense sands were encountered within the boreholes above these lower relative density bands, refusal of the tests may occur within the very dense sands and only limited information may be obtained. Nevertheless, it is recommended that some EFCP testing be attempted as it would provide greater detail on the variations within the subsurface profile.

Since EFCP testing does not provide sample recovery and due to its possible limitations detailed above, further boreholes should be drilled as part of the detailed investigations. The depth of the boreholes will depend on the anticipated footing system of the proposed buildings and may be required to penetrate into the underlying sandstone bedrock (with coring) if piles founded within the sandstone are envisaged.

Any detailed investigations would need to be carried out following demolition of the existing buildings on site, once access to the whole of the site is possible for the truck mounted EFCP testing rig and drilling rig.

5 GENERAL COMMENTS

Occasionally, the subsurface soil conditions between the completed boreholes 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.

The offsite disposal of soil may require classification in accordance with the EPA guidelines as inert, solid, industrial or hazardous waste. We can complete the necessary classification and testing if you wish to commission us. As testing requires about seven days to complete, allowance should be made for such testing in the construction programme unless testing is completed prior to construction. If contamination is found to be present then substantial further testing and delays should be expected.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

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. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. 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.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

<u>QA Review by:</u>

D J Bliss
Senior Geotechnical Engineer

F Vega
Senior Associate

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **1**

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Date: 17-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: ≅ 16.7m
Datum: AHD
Logged/Checked by: N.E.S./[illegible]

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
			0		-	ASPHALTIC CONCRETE: 50mm.t	D	-	-	
						FILL: Sandy gravel, fine to medium grained igneous gravel, grey, with fine to coarse grained sand.				APPEARS MODERATELY COMPACTED
	DS	N = 9 4,5,4				FILL: Sand, fine to medium grained, grey brown.				
			1							
					SP	SAND: fine to medium grained, dark brown, with a trace of cemented nodules.	D	MD	-	
	DS	N = 17 10,8,9								
AFTER 24 HRS			2							
						SAND: fine to medium grained, light grey.	M			
	DS	N = 23 7,11,12	3				W			
ON COMPLETION			4							
	DS	N = 34 6,14,20						D		
			5							
		Nc = 21 32/ 100mm	6					(VD)		
			7							

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BOREHOLE LOG

Borehole No.
1

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Job No. 18602VB
Date: 17-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: ≅ 16.7m
Datum: AHD
Logged/Checked by: N.E.S./DM

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
					SP	SAND: fine to medium grained, light grey.	W	(VD)		
	DS	N > 30 19,30/ 150mm								
			8							
		Nc = 19 26 40	9					VD		
			10							
		Nc = 10 28 30 /100mm	11							
	DS									
		Nc = 3 3 1 1 2 1 2 2 9	12					VL-L		
			13							
								MD-D		30mm
			14							

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BOREHOLE LOG

Borehole No. 1

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Date: 17-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: ≅ 16.7m
Datum: AHD
Logged/Checked by: N.E.S./Dm

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
				■				SM	SILTY SAND: fine to medium grained, dark grey brown.	W	MD-D		ORGANIC ODOUR
						15			END OF BOREHOLE AT 15.01m				10mm PVC STANDPIPE INSTALLED ON COMPLETION TO 6m DEPTH
						16							
						17							
						18							
						19							
						20							

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BOREHOLE LOG

Borehole No. 2

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Date: 17-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: ≅ 18.2m
Datum: AHD
Logged/Checked by: N.E.S./ DM

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
			0		-	ASPHALTIC CONCRETE: 50mm.t				
						FILL: Sandy gravel, fine to medium grained igneous gravel, grey brown, fine to coarse grained sand.	D	-	-	
		N = 28 9,14,14			SP	FILL: Sand, fine to medium grained grey brown.	D	MD		
ON COMPLETION			1			SAND: fine to medium grained, light grey.				
		N = 13 6,5,8								
			2			SAND: fine to medium grained, brown.	M			
		N = 21 10,10,11	3			SAND: fine to medium grained, light grey.				
▶			4				W			
		N = 30 4,11,19						MD-D		
			5							
		N = 19 6,7,12	6					MD		
			7							

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Borehole No. 2

BOREHOLE LOG

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB **Method:** SPIRAL AUGER JK450 **R.L. Surface:** ≅ 18.2m
Date: 17-5-04 **Datum:** AHD
Logged/Checked by: N.E.S./

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
					SP	SAND: fine to medium grained, light grey.	W	MD		
	DS	N = 41 8,19,22						D		
			8							
	DS	N = 43 1,13,30	9							
			10							
		Nc = 1, 2, 3, 2, 1, 5, 12, 14, -						L		
			11					L-MD		
		Nc = 2, 3, 7, 10, 13, 20	12					MD -D		
			13							40mm
			14							

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BOREHOLE LOG

Borehole No. **2**

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB **Method:** SPIRAL AUGER JK450 **R.L. Surface:** ≅ 18.2m
Date: 17-5-04 **Datum:** AHD
Logged/Checked by: N.E.S./DM

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
	DS				SP	SAND: fine to medium grained, dark brown grey.	W	MD -D		ORGANIC ODOUR
			15							50mm
						END OF BOREHOLE AT 15.05m				
			16							
			17							
			18							
			19							
			20							

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BOREHOLE LOG

Borehole No. **3**

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Date: 18-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: N/A
Datum:
Logged/Checked by: N.E.S./

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
			0		-	ASPHALTIC CONCRETE: 50mm.t				
						FILL: Sand, fine to medium grained, brown.	-	-	-	
	DS	N = 19 4,9,10				FILL: Sand, fine to medium grained, yellow brown mottled grey and brown.	M			APPEARS WELL COMPACTED
			1		SP	SAND: fine to medium grained, yellow brown.	M	VL	-	
ON COMPLETION	DS	N = 2 3,2,0								
			2			SAND: fine to medium grained, light grey.		MD		
	DS	N = 29 7,13,16	3							
▶			4				W			
	DS	N = 53 6,26,27						VD		
			5							
	DS	N = 41 7,17,24	6					D		
			7							

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BOREHOLE LOG

Borehole No.
3

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB **Method:** SPIRAL AUGER JK450 **R.L. Surface:** N/A
Date: 18-5-04 **Datum:**
Logged/Checked by: N.E.S./

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
					SP	SAND: fine to medium grained, light grey.	W	D		
	DS	N > 52 10,32,20/ 80mm						VD		
			8							
	DS	N = 51 6,21,30	9							
			10							
		Nc = SUNK SUNK SUNK 3 5 6 9 15						VL		
			11					L-MD		
		Nc = 2 7 9 13 20 25 25	12					D		
			13							
										60mm
			14							

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Borehole No. 3

3/3

BOREHOLE LOG

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Date: 18-5-04
Method: SPIRAL AUGER JK450
R.L. Surface: N/A
Datum:
Logged/Checked by: N.E.S./

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
								SP	SAND: fine to medium grained, light grey.	M	D		
						15			END OF BOREHOLE AT 15.02m				20mm
						16							
						17							
						18							
						19							
						20							

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BOREHOLE LOG

Borehole No. **4**
1/3

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB **Method:** SPIRAL AUGER JK450 **R.L. Surface:** N/A
Date: 18-5-04 **Datum:**
Logged/Checked by: N.E.S./

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
			0		-	ASPHALTIC CONCRETE: 50mm.t	M			
						FILL: Sand, fine to medium grained, with a trace of fine to medium grained igneous gravel, grey brown.				
					SP	SAND: fine to medium grained, yellow brown.	M	MD	-	
	DS	N = 16 3,7,9								
			1							
	DS	N = 12 5,7,5								
			2							
						SAND: fine to medium grained, light grey.				
			3							
	DS	N = 20 9,9,11								
ON COMPLETION										
			4			SAND: fine to medium grained, grey.				
								D		
	DS	N = 34 7,17,17								
			5							
						SAND: fine to medium grained, light grey.				
	DS	N > 20 15,20/ 80mm	6							
			7							

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BOREHOLE LOG

Borehole No. **4**

2/3

Client: CROWN INTERNATIONAL HOLDINGS GROUP

Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT

Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB **Method:** SPIRAL AUGER JK450 **R.L. Surface:** N/A

Date: 18-5-04 **Datum:**

Logged/Checked by: N.E.S./

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
								SP	SAND: fine to medium grained, light grey.	W	D		
					N > 30 14,30/ 150mm						(VD)		
						8							
					N > 30 11,30/ 150mm	9							
						10							
					Nc = 13 40 -								
						11							
					Nc = SUNK SUNK 1 3 6 10 14 18	12					L-MD		
											MD -D		
						13							
													40mm
						14							

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BOREHOLE LOG

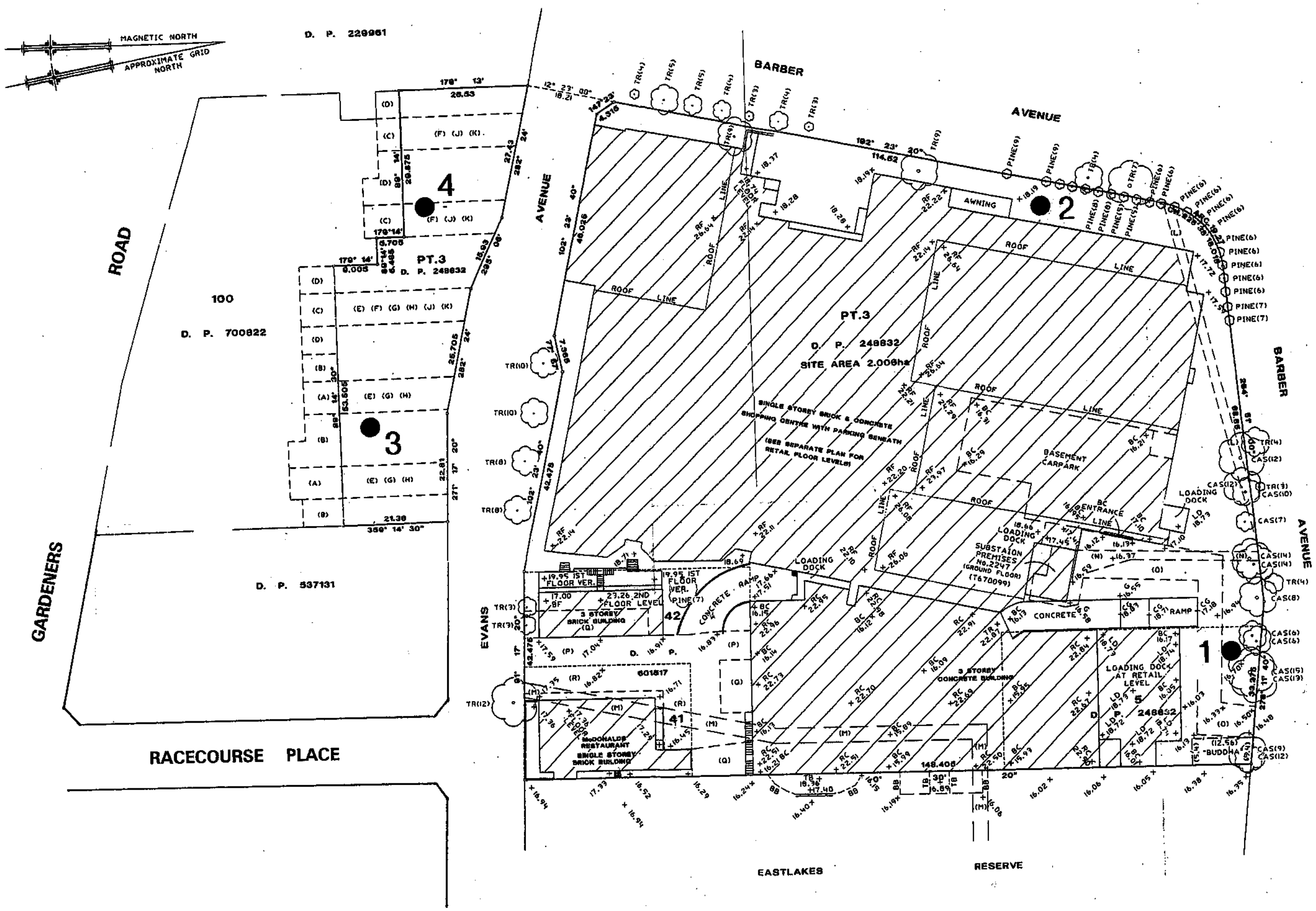
Borehole No. **4**

3/3

Client: CROWN INTERNATIONAL HOLDINGS GROUP
Project: PROPOSED BKK SHOPPING CENTRE REDEVELOPMENT
Location: EVANS AVENUE, EASTLAKES, NSW

Job No. 18602VB
Method: SPIRAL AUGER JK450
R.L. Surface: N/A
Date: 18-5-04
Datum:
Logged/Checked by: N.E.S./

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
	DS				SP	SAND: fine to medium grained, light grey.	W	MD -D		
			15			END OF BOREHOLE AT 15.05m				50mm SLOTTED PVC STANDPIPE INSTALLED ON COMPLETION TO 6m DEPTH
			16							
			17							
			18							
			19							
			20							



BOREHOLE LOCATION PLAN

SCALE (M)
0 50

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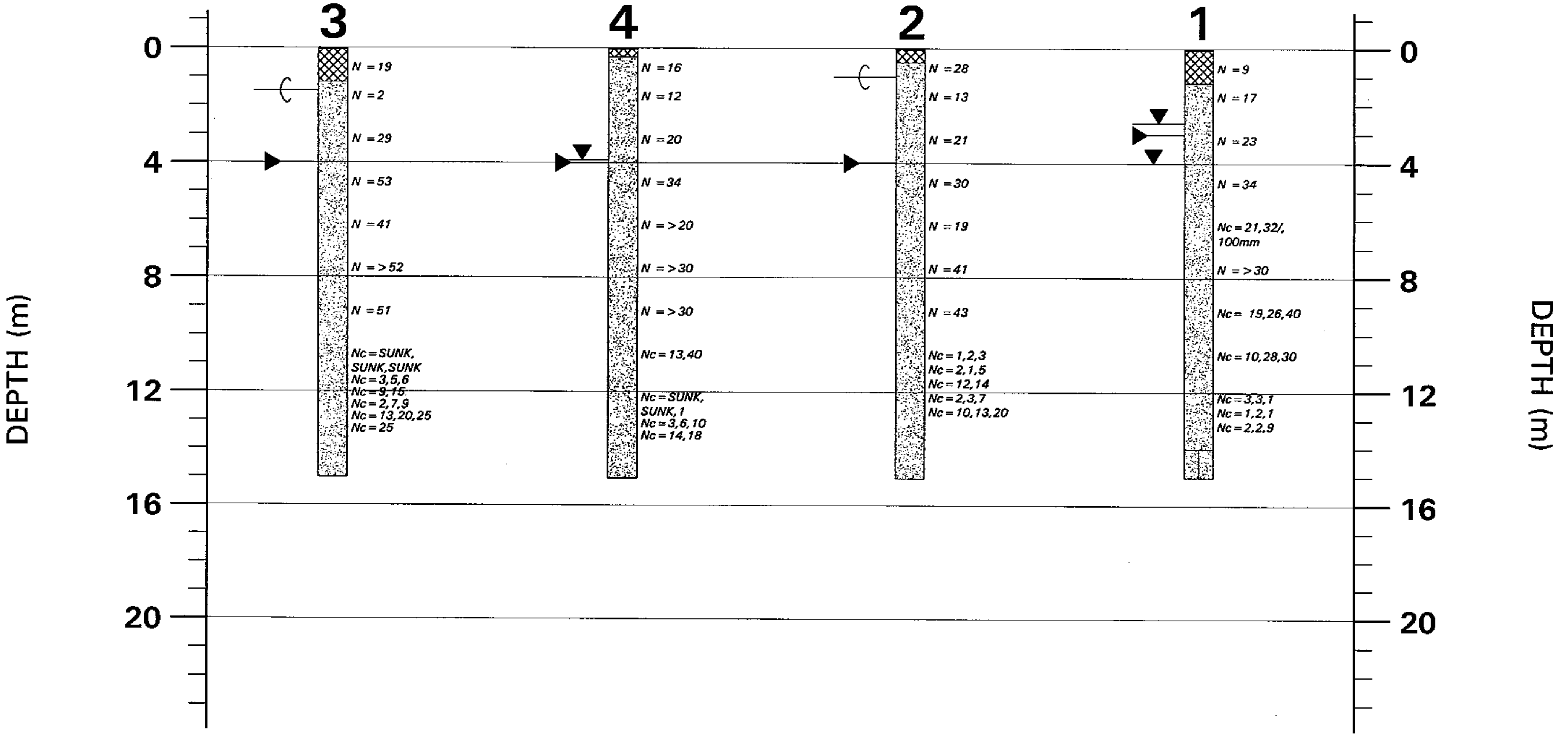
Report No. 18602VB Figure No. 1

GRAPHICAL BOREHOLE SUMMARY



Asphaltic/ Bituminous Paving or Coal	Silty Sand
Fill	Borehole Collapse Depth
Sand	Groundwater seepage level
Observed water level	
N	SPT "N" VALUE
Nc	SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

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Job No.: 18602VB

Figure No.: 2

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 containing 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

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

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

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding risk 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.

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 may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to remoulding, contamination 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. 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.

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 weak 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 540mm 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 same 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 an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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 on continuous chart recorders. The plotted results given in this report have been copies from the original records.

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.

There are two scales available for measurement of cone resistance. The lower (A) scale (0 to 5MPa) is used in softer soils where increased sensitivity is required. The main (B) scale has a range of 0 to 50MPa.

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 friction ratios are only inferred and must not be considered as exact.

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

Interpretation of EFCP 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 show, 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 tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scale 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 or 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 and the attached explanatory notes summarise important aspects of the Laboratory Test Procedures adopted.

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.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> 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.

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GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

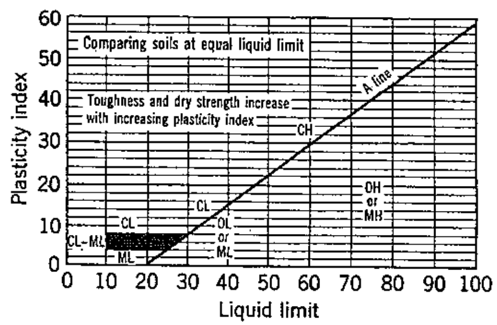
- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

UNIFIED SOIL CLASSIFICATION TABLE

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	$C_U = \frac{D_{60}}{D_{10}}$ 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	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	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	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*)	$C_U = \frac{D_{60}}{D_{10}}$ 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

Laboratory Classification Criteria (coarse-grained soils): 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.

Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols[a]	Typical Names	Information Required for Describing Soils
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
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
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 (e.g. 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.

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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.
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.

LOG COLUMN	SYMBOL	Density Index (I_D) Range (%)		SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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.		

LOG COLUMN	SYMBOL	DEFINITION
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.

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LOG SYMBOLS

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	