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REPORT

on

GEOTECHNICAL INVESTIGATION

PROPOSED TRINITY POINT MARINA AND
TOURIST DEVELOPMENT
49 LAKEVIEW ROAD
MORISSET PARK

Prepared for
JOHNSON PROPERTY GROUP PTY LTD

Project 39823
DECEMBER 2007

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4 December 2007

REPORT ON

GEOTECHNICAL INVESTIGATION

PROPOSED TRINITY POINT MARINA AND TOURIST DEVELOPMENT

49 LAKEVIEW ROAD, MORISSET PARK

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1. INTRODUCTION

This report presents the results of a geotechnical investigation at the site of the proposed Trinity Point Marina and Tourist Development, located at 49 Lakeview Drive, Morisset Park (Lot 31, Part Lot 32 and Part Lot 33, DP 1117408). The work was carried out for Johnson Property Group Pty Ltd.

The purpose of the investigation was to provide the following:

- subsurface conditions at the site;
- site classification with regard to foundation soil reactivity (shrink-swell), in accordance with AS 2870-1996 (Ref 1);
- comments on suitable footing types and soil parameters for footing design of the proposed on-land structures and the proposed marina;
- background groundwater quality data;
- comments on site preparation and earthworks;
- flexible pavement thickness design;
- material quality and compaction requirements for the proposed driveways and parking areas;

For the purpose of the investigation, the client supplied concept plans of the proposed development, along with site survey plans. The concept plans used in the preparation of this report are the Site Plans by HBO + EMTB Architects Pty Ltd (Ref No 202669, SK000, Issue I dated 29 October 2007 and SK000 Option 01, Revision J, dated 2 November 2007). The site survey plan was prepared by SurDevel Pty Ltd, (Ref 1320, dated 30 November 2006). A hydrographic survey of the proposed marina area had been undertaken by another consultant, however the contours were provided to DP on a plan by Patterson Britton & Partners (Ref 6759.10-GA, dated 17 September 2007).

The project is subject to other reports recently prepared by Douglas Partners Pty Ltd (DP) which includes an acid sulphate soil assessment (Ref 1), a geochemical analysis within the proposed marina (Ref 2), and a waste classification report for the northern part of the site (Ref 3).

2. PROPOSED DEVELOPMENT

2.1 General

The Trinity Point Marina and Tourist Resort comprises a number of components including the Marina, Marina Village and clusters of multi-storey accommodation buildings (Blocks A to G).

The Marina and Marina Village development will include an approximately 300 berth marina, along with an associated breakwater, boat maintenance facilities (travel lift, hardstand and workshop), and other related commercial infrastructure such as café, restaurant and function facilities.

Immediately south of the Marina Village is a cluster of multi storey buildings, up to six stories in height for short to medium term tourist accommodation. These areas are shown as Blocks A, B, C and D on the attached Drawing 2. These buildings will include under-croft car parking.

Another three clusters of multi-storey accommodation buildings are located further to the south (shown as Blocks E, F and G on attached Drawing 2). These three clusters comprise apartment style accommodation, in two to five storey buildings, associated car parking (underground parking), access roadways, footpaths, boardwalks, jetties and landscaping.

2.2 Proposed Marina Village Centre and Floating Marina Berths

The proposed marina and village centre will include a 308 berth marina consisting of up to four arms of floating pontoons, a floating helipad pontoon, marina administration offices, a breakwater, a travel lift with associated hardstand area for boat repairs and maintenance, and a workshop. It is understood that the marina has been configured to avoid any dredging.

The marina will comprise a system of floating walkways, and associated berths. The floating walkways would be located between vertical piles driven into the lake bed. It is understood that the preferred pile type is tubular steel piles.

The marina will incorporate a breakwater around the southern and eastern boundaries. The proposed breakwater will consist of two rows of parallel tubular steel piles driven in to the lake bed, with timber slats supported on the outer side of each row of piles. The breakwater will also have a timber walkway, allowing access around the perimeter of the marina, and for access to the helipad.

The helipad will be an approximately 25 m by 25 m floating steel pontoon anchored to the lake bed, with an access gangway directly from the breakwater walkway. The current preference is that the anchors would be steel piles driven into the lake bed similar to piles for the breakwater and pontoons, however the piles would be cut off at the lake bed level.

In addition to the marina, there will be an associated on-shore village centre incorporating a café, restaurant, function centres, chandlery, general store and commercial offices.

2.3 Proposed Tourist/Accommodation Development

The southern portion of the site will incorporate apartment style accommodation (serviced tourist and permanent residential) with two to five storey buildings arranged in a series of three building clusters (Blocks E, F and G).

2.4 Pavements

Proposed pavement areas for the site include access roads and parking areas. It is understood that the majority of parking for Blocks A to D will be offered via under-croft parking beneath the proposed multi-storey buildings. It is understood that the under-croft parking in this area of the site will be at about RL 1.2 (AHD).

Blocks E to G will include basement car parking with preliminary basement floor levels ranging from 0.35 m to 4.85 m AHD.

2.5 Cut/Fill

Preliminary levels for under-croft car parking and basement car parking floor levels suggest approximate cut and fill depths could be in the order of the following:

Building Cluster	Approximate Ground Surface Level (AHD)	Preliminary Under-croft/Basement Floor Level (AHD)	Preliminary Approx Fill Depth (m)	Preliminary Approx Excavation Depth (m)
A	0.8	1.2	0.4	-
B	0.9	1.2	0.3	-
C	0.9	1.2	0.3	-
D	0.9 – 1.9	1.2	0.3	0.7
E	1.6 – 3.4	0.35	-	1.25 – 3.05
F	2.6 – 6.8	1.65 to 3.53	-	0.95 – 3.29
G	4.0 – 8.5	4.85	0.85	3.65

It is anticipated that excavations could also be required for installation of utilities, and also for swimming pool construction, although the final locations of these features are unknown at this time.

3. SITE DESCRIPTION AND REGIONAL GEOLOGY

The site is located to the north of, and on, Bluff Point on the Morisset Peninsula of the western shores of Lake Macquarie. The site is described as 49 Lakeview Road (Lot 31, Part Lot 32 and Part Lot 33, DP 1117408), Morisset Park. A plan showing the approximate location of the site is shown on Drawing 1, attached.

It is understood that the site used to contain several buildings, however these have been demolished. At the time of the investigation, the site was grassed with several stands of mature trees, particularly along the shoreline. Several stockpiles of building rubble and vegetation were located towards the southern part of the site.

Site elevations range from water level in the northern and eastern parts of the site up to about 8.5 m (AHD) at the southern end, which is known as Bluff Point. The site is relatively level in the northern part, where the marina is to be constructed, and slopes up to the high point at about 2º to 6º.

The following photographs show the general site area at the time of the investigation.

Photo 1 – set up on Bore 101, in the area of the proposed marina

Photo 2 – view of site from the Lake

Photo 3 – looking south towards the crest of Bluff Point, in the area of the proposed tourist village

Photo 4 – drill rig set up on modular barge, in proposed marina area

Reference to the 1:100,000 Newcastle Coalfield Geological series sheet indicates that the site is underlain the Narabeen Group of rocks. The Narabeen Group includes both the Terrigal Formation and the Clifton Subgroup. The Terrigal Formation typically includes sandstone and siltstone, while the Clifton Subgroup typically includes conglomerate, sandstone, siltstone and claystone.

4. FIELD WORK METHODS

The field work was undertaken in the period 25 September to 16 October 2007 and included the following:

- six cone penetration tests within the proposed marina area (CPT 1 to 6);
- four on-land bores within the proposed marina village area (Bores 101/A and 102/A);
- three on-land bores within the proposed tourist development (Bores 103 to 105);
- three over water bores within the proposed marina area (Bores 201 to 203); and
- ten test pits across the site (Pits 301 to 310).

The CPTs were taken to refusal, which ranged from 9.6 m depth (CPT 3) to 13.1 m depth (CPT 4). The tests comprised hydraulically pushing a 35 mm diameter instrumented cone and friction sleeve assembly into the ground from a ballasted truck.

Bores 101 to 105 were drilled using a truck mounted drilling rig, equipped for geotechnical sampling. In situ testing included standard penetration tests (SPTs) at regular depth intervals. A pocket penetrometer was also used to assess the strength of samples recovered from the SPTs. The target depth for Bores 101 and 102 was 6 m of rock core, while the target depth for Bores 103 to 105 was 5 m or refusal. Groundwater monitoring wells were installed in each of these bores on completion.

Bores 101A and 102A were drilled by a 4WD mounted drilling rig equipped with hollow flight augers for the purpose of installing a second, shallower groundwater monitoring well adjacent to each of Bores 101 and 102, respectively.

The over-water bores were also drilled using a truck mounted drilling rig, set up on a modular barge (refer Photo 4). The target depth for the over-water bores was 3 m into rock, however this was not able to be achieved at all locations. Bore 201 had to be abandoned early due to strong winds and unsafe working conditions.

The test pits were excavated using a backhoe to depths of between 2 m and 3 m.

The locations of the CPTs, Bores and Pits are indicated on attached Drawing 2.

The tests were set out by a geotechnical or geo-environmental engineer from DP who also logged the subsurface profile in each pit and bore and took regular samples for laboratory testing and identification purposes. Pocket penetrometer and dynamic cone penetrometer tests were performed at selected depths and locations.

All test locations were selected based on the proposed concept layout available at the time of the investigation. The locations were positioned approximately, with some measured from existing site features, and some positioned using a hand-held GPS unit. The on-land test locations were staked on completion and were subsequently surveyed for location and elevation by project surveyors, SurDevel Pty Ltd. The over-water bores were surveyed by the project surveyors while the rig was set up at the bore location.

5. FIELD WORK RESULTS

5.1 General

The subsurface conditions encountered are presented in detail in the attached CPT reports, borehole logs and test pit logs. These should be read in conjunction with the general notes preceding them, which explain the descriptive terms and classification methods used in the reports. The following is a summary of these subsurface conditions.

Marina Area (off-shore Portion)

In general, the lake bed sediments comprised a mixture of sand, silt and clay in varying proportions. The over-water bores (Bores 201 to 203) encountered soft lake sediment which ranged in thickness from about 1.7 m to 3.0 m. The underlying soils generally comprised clay, gravelly clay and clayey sand, which was in turn underlain by bedrock at depths which ranged from 5.8 m to 7.9 m below the lake bed.

Marina Village and Blocks A to D

Bores 101 and 102, and Pits 301 to 306 generally encountered sandy soils with variable proportions of clay, silt and gravel to depths of about 5 m. In the bores, the sandy soils were underlain by clay, sandy clay and gravelly clay. Rock was encountered in the bores at depths of 12.8 m and 11.4 m respectively.

The profile in CPT 1 indicates the presence of very soft to soft clay between about 1.8 m and 3.1 m depth, in the vicinity of the boat ramp and workshop.

Blocks E, F and G

Bores 103 to 105, and Pits 307 to 310 generally encountered filling (with the exception of Pit 309) to depths of up to 1.15 m over generally sandy and clayey soils. The clay in Pit 309 graded to clayey sand/extremely weathered sandstone below about 1.0 m, and backhoe refusal was encountered at 1.8 m depth. Rock was also encountered in Bores 104 and 105, with pebbly sandstone encountered below 4.2 m in Bore 104, and residual clay grading to an extremely low strength conglomerate below 4 m in Bore 105.

5.2 Bedrock

The following table summarises the depth to the top of bedrock and/or refusal in each of the tests.

Table 5.1 – Summary of Rock Depths

Project Component	Test	Approximate Surface RL (m)	Depth to Top of Rock (m)	Depth to Refusal (m)
Marina	201	-5.86	5.8	-
	202	-5.15	6.9	-
	203	-5.35	7.9	-
Marina Village	1	0.67	-	11.4
	2	0.81	-	12.6
	3	0.92	-	9.6
	4	0.99	-	13.1
	101	1.27	12.8	-
	102	0.89	11.4	-
Blocks A to D	5	0.78	-	10.6
	6	1.05	-	10.6
Blocks E to G	103	2.49	NE to 5.95	-
	104	3.82	4.2	-
	105	6.62	4.0*	-

Notes to Table 5.1:

NE – Not encountered
* Approximate depth at which soil started transitioning/grading to rock

5.3 Groundwater

Groundwater was observed in each of the remnant CPT holes. Groundwater monitoring wells were installed in each of the on-land bores (ie. Bores 101 to 105, 101A and 102A) to facilitate measurement of groundwater levels on different occasions and also sampling for groundwater chemistry analysis. Groundwater seepage was observed during excavation of the test pits, however the pits were only open for a relatively short period of time, and hence it is likely that these observations do not necessarily represent the static water level. The following tables summarise the groundwater observations made during field work, and also within the wells on the subsequent site visits.

Table 5.2 – Summary of Groundwater Observations in Remnant CPT Holes

Project Component	CPT	Approximate Surface Level (AHD)	Approximate Depth to Water in Remnant CPT Hole (m)	Approximate Groundwater Level in Remnant CPT Hole (AHD)
Marina Village	1	0.67	0.5	0.2
	2	0.81	0.5	0.3
	3	0.92	0.9	0.0
	4	0.99	0.8	0.2
Blocks A to D	5	0.78	0.4	0.4
	6	1.05	0.7	0.4

Table 5.3 – Summary of Groundwater Seepage Observations in the Test Pits

Project Component	Location	Approximate Surface Level (AHD)	Depth of Groundwater Seepage Observed During Field Work (m)
Marina Village	301	0.96	1.5
	302	0.97	1.3
	303	1.21	1.4
Blocks A to D	304	1.16	1.0
	305	1.15	1.0
	306	1.12	1.1
Blocks E to G	307	1.78	1.5
	308	2.6	Not encountered
	309	3.0	Not encountered
	310	4.4	Not encountered

Table 5.4 – Summary of Groundwater Measurements in Bores

Project Component	Bore	Approximate Surface Level (AHD)	Depth to Groundwater Below Ground Surface (m) and date				Range of Groundwater Levels Observed (AHD)
			5/10/07	9&10/10/07	16/10/07	24/10/07	
Marina Village	101	1.27	1.2	1.2	1.2	NM	0.1
	101A	1.27	NM	NM	1.15	1.22	0.0 to 0.1
	102	0.89	NM	0.61	0.88	NM	0.0 to 0.3
	102A	0.89	NM	NM	0.83	0.94	-0.1 to 0.1
Blocks E to G	103	2.47	1.51	1.57	1.63	NM	0.8 to 1.0
	104	3.82	2.83	2.85	2.93	NM	0.9 to 1.0
	105	6.62	Dry	Dry	Dry	Dry	-

It should be noted that groundwater levels are affected by factors such as climatic conditions and soil permeability and will therefore vary with time.

Groundwater pH, Electrical Conductivity, Dissolved Oxygen and Turbidity were also measured in the wells following installation, with the results summarised in Table 5.5, below:

Table 5.5 – Summary of Groundwater Properties in Bores

Bore No	Range of pH values	Range of EC values	DO(%)	Turbidity (NTU)
101	7.1 to 7.3	1.7 to 3.8	31 to 49	1450 to 2618
101A	7.2 to 7.7	0.6 to 0.8	47	2541
102	6.8 to 7.3	8.7 to 2.1	42 to 95	1277 to 1324
102A	7.4 to 7.7	1.2 to 2.1	77	2452
103	5.0	0.6	43	2262
104	4.1 to 4.2	5.6 to 6.8	51	2619
105	dry	dry	Dry	Dry

Notes to Table 5.5:

EC – Electrical Conductivity
DO – Dissolved Oxygen

6. LABORATORY TESTING

Geotechnical laboratory testing comprised the following:

- ten particle size distribution tests;
- nine plasticity index tests;
- two linear shrinkage tests;
- two soil aggressivity tests (pH, chlorides and sulphates);
- 32 point load index tests on recovered rock core to assess rock strength.

In addition, groundwater samples were collected from each of the wells to obtain background water quality data, and also groundwater aggressivity data. The well in Bore 105 was dry, and hence no sample was collected. One sample (D1) was submitted for QA/QC purposes. Groundwater was tested for the following:

- Metals: Arsenic (As); Antimony (Sb); Barium (Ba); Beryllium (Be); Boron (B); Cadmium (Cd); Chromium (Cr); Copper (Cu); Cobalt (Co); Lead (Pb); Manganese (Mn); Molybdenum (Mo); Nickel (Ni); Selenium (Se); Zinc (Zn); and Mercury (Hg);
- Nitrite, Nitrate, Chloride, Sulphate;
- Total Phosphorous; Total Nitrogen;
- Total Iron.

Limited soil geochemical and acid sulphate soil testing was undertaken concurrent with the geotechnical investigation. The results are reported separately and have not been included in this report (Refs 1 to 3).

The results of the point load index testing are shown on the attached borehole logs. The detailed results of other laboratory testing are presented in the attached laboratory report sheets, and are summarised in the following tables.

Table 6.1 – Summary of Geotechnical Laboratory Testing

Project Component	Bore	Depth (m)	Description	% Sand and Gravel	% Fines (Passing 75 micron sieve)	W_L	W_P	PI	LS (%)
Marina Village	101	1.0 – 1.45	Gravelly sand	89	11	-	-	-	-
	102	1.0 – 1.45	Sand	88	12	-	-	N/P	-
		1.0 – 4.45	Silty sand	75	25	-	-	N/P	-
Marina	201	0.0 – 0.45	Silty sand/sandy silt	55	45	-	-	-	-
		2.4 – 2.75	Silty clay	2	98	41	15	26	-
	202	0.0 – 0.45	Sandy silty clay	69	31	-	-	-	-
		4.0 – 4.45	Clayey sand	64	36	34	18	16	-
	203	2.5 – 2.95	Sandy silty clay	42	58	34	15	19	-
		5.0 – 5.45	Clay	85	15	58	15	43	-
Blocks E to G	103	1.0 – 1.45	Silty gravelly sand	63	37	17	15	2	-
	104	2.5 – 2.95	Silty clay	-	-	46	25	21	11.0
	105	1.0 – 1.45	Silty sandy clay	-	-	35	18	17	10.5

Notes to Table 6.1:

W_P – Plastic Limit
W_L – Liquid Limit
PI - Plasticity Index
LS – Linear Shrinkage
N/P – Non-plastic

Table 6.2 – Summary of Soil Aggressivity Results

Project Component	Bore	Depth (m)	Description	pH	Chloride, Cl (mg/kg)	Sulphate, SO_4 (mg/kg)
Marina village	101	2.5 – 2.95	Gravelly clayey sand	8.0	14	26
	102	5.5 – 5.95	Silty clay	7.5	820	170

Table 6.3 – Summary of Laboratory Results for Groundwater Chemistry - Metals

Project Component	Location	Analyte (µg/L)																Analyte (mg/L)						
		Antimony	Arsenic	Barium	Beryllium	Boron	Cadmium	Chromium	Copper	Cobalt	Lead	Manganese	Molybdenum	Nickel	Selenium	Zinc	Tin	Total Iron	Nitrate as N	Chloride, Cl	Sulphate, SO4	Total Phosphorus as	Total Nitrogen	Mercury
Marina village	101	<PQL	<PQL	33	<PQL	470	<PQL	1.2	<PQL	<PQL	<PQL	260	2.5	<PQL	<PQL	12	<PQL	2.4	<PQL	850	110	0.40	4.6	<PQL
	102	<PQL	6.4	190	<PQL	1500	<PQL	6.3	1.3	22	<PQL	130(	2.6	11	23	12(	0.03	250	<1	340C	1300	<0.5	3.3	<PQL
locks E t G	103	<PQL	<PQL	40	<PQL	53	<PQL	<PQL	1.1	2.1	5.4	77	<PQL	3.4	<PQL	33	<PQL	).25	<PQL	190	44	0.13	<PQL	<PQL
	104	<PQL	<PQL	140	3.6	120	0.64	15	3.9	16	40	300	<PQL	13	<PQL	11(	<PQL	15	<0.1	260C	180	<PQL	1.0	<PQL
A Sampl	D1	<PQL	<PQL	34	<PQL	480	<PQL	<PQL	<PQL	<PQL	<PQL	250	2.5	<PQL	<PQL	14	<PQL	NT	NT	NT	NT	NT	NT	<PQL
Laboratory PQL		1	1	1	1	1	0.1	1	1	1	1	1	1	1	2	1	0.03	0.01	0.05	1	1	0.1	1	5E-04

Notes to Table 6.3:

Sample D1 is a duplicate of Sample 101
PQL – Practical quantification limit
NT – Not tested

7. COMMENTS

7.1 General

The comments presented herein primarily relate to the portion of the site which includes the Marina, Marina Village and Blocks A to D. Comments related to other areas of the site (ie Blocks E to G) are preliminary in nature.

All of the comments assume that detailed, targeted investigation will be undertaken during the detailed design stage of the project, once the building layout and proposed earthworks details are confirmed.

In general, the lower lying portions of the site are underlain by weak alluvial soils, with groundwater present at depths of about 0.5 m to 1.0 m. Zones of very loose sandy soils, and very soft to soft clayey and silty soils were encountered to depths of up to about 5.5 m, with conditions below this depth improving, but still including zones of loose sandy soils and/or firm clays to depths of generally about 6 m to 8 m, but up to about 11.5 m (Bore 101 and CPT 2).

These soils present limitations for the support of the proposed structures (low-rise, high-rise and pavements) because they will settle under loads from buildings, filling and their own self weight. These soils may also be at risk of liquefaction if subjected to a seismic event, however additional analysis would be required to assess this further.

The geotechnical conditions will likely result in the need to consider deep foundations (piles) for the majority of the structures proposed within the Marina area, Marina Village and Blocks A to D.

Conditions improve gradually as site elevations rise to the south, however, it is expected that most multi-storey buildings constructed on the site will likely require the use of footings founded in bedrock due to relatively high structural loads. It may be possible to found some of the smaller structures located in the southern portion of the site on shallow foundations, however this will require specific targeted investigation during the design stage of the project, once structural loads are known.

The presence of shallow groundwater combined with the poor ground conditions in the lower lying areas of the site also present potential access issues on the site, and hence it is likely that bridging layers will be required to form working platforms on which construction equipment can operate, and to support at-grade features such as pavements, slabs etc.

Comments regarding these and other geotechnical aspects of the proposed development are presented in the following sections of this report.

7.2 Groundwater

7.2.1 General

Groundwater chemistry data is presented in Section 6 of this report. This data has not been compared to any guidelines at this point in time, and was collected to provide background water quality data for future reference.

Groundwater was encountered at depths as shallow as 0.4 m below ground surface during the investigation. It is possible that there may be some tidal influence in the groundwater levels in the low-lying area of the site, however this has not been assessed in detail. Groundwater levels may therefore fluctuate depending on the water level in Lake Macquarie, as well as prevailing weather conditions.

Anecdotal evidence indicates that the water level in Lake Macquarie rose by about 1 m above average levels during the recent June long weekend storms, with much of the Lake's low-lying foreshore areas inundated. It is not known whether the Trinity Point project area was inundated or not during this time. However this recent weather event illustrates the potential for low lying areas to become inundated, and hence groundwater levels to potentially rise to the ground surface during extreme weather events.

The relatively shallow groundwater, combined with potential fluctuations, means that a number of structural elements, such as slabs, shallow footings etc, may need to be designed to accommodate potential buoyancy or uplift forces, depending on site grades.

7.2.2 Dewatering

Excavations within the lower-lying Marina, Marina Village and Blocks A to D components are likely to encounter groundwater, and may require dewatering. At the time of the investigation, groundwater was encountered at depths as shallow as 0.4 m, however the groundwater response to rainfall events and/or tidal fluctuation has not been assessed at this time.

It is considered that if dewatering is required within the lower lying areas of the site, then additional testing and analysis may be required once excavation levels are confirmed to assess soil permeability and appropriate dewatering methods.

Within the southern part of the site (Blocks E to G) it is possible that sump and pump arrangements may become suitable, as the soils increase in clay content and the depth to groundwater increases.

If excavations requiring dewatering are likely, it is recommended that additional investigation include in situ testing to assess soil permeability, and also monitoring of groundwater level response to weather events and tidal fluctuations.

Dewatering at the site will need to consider acid sulphate soils (Ref 1).

7.3 Site Classification

7.3.1 General

Site classification to AS 2870 (Ref 4) is not strictly applicable to this site due to it being a commercial and high-rise development rather than a traditional low-rise residential development. However, the principles of footing design and site maintenance presented therein should be taken into account for the buildings proposed for the site.

Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with seasonal variation in moisture and is based on procedures presented in AS 2870-1996 (Ref 4), the typical soil profiles revealed in the tests, and the results of laboratory testing. The process of cutting and filling will affect the site classification, and hence the classifications should be revised once details of site cutting and/or filling are known, as required by AS 2870-1996 (Ref 4).

The site classifications for the Marina Village and Blocks A to D are presented in the following sections, and are based on the information obtained from test pits and bores and on the results of laboratory testing. The classifications have involved some interpolation between data points, and in the event that the conditions encountered during construction are different to those presented in this report, it is recommended that advice be sought from this office.

Articulation joints should be provided within masonry walls in accordance with TN61 (Ref 5) in order to reduce the effects of differential movement.

It should be noted that the classifications are dependent on proper site maintenance.

7.3.2 Marina Village

The marina village is designated Class P due to the poor ground conditions. Footings should be designed therefore in accordance with engineering principles as required by AS 2870-1996 (Ref 1). Site maintenance should be carried out in accordance with the attached CSIRO BTF 18 and Appendix B of AS 2870-1996 for a Class S site.

7.3.3 Blocks A to D

Blocks A, B and C are each greater than three storeys in height, and hence will require design by engineering principles. Site classification to AS 2870-1996 will not apply.

Block D contains buildings between two and five storeys in height. This area of the site is also designated Class P due to the poor ground conditions, and will therefore require design by engineering principles.

7.4 Shallow Footings

The loose sandy soils encountered within the upper profile of the CPTs, bores and pits in the low lying area of the site (ie Marina Village and Blocks A to D) are not suitable to support shallow footings. It is considered likely that deep footings (piles) will be required to support most structural loads within this area of the site.

Shallow footings may become an option for lightly loaded structures as development progresses uphill to the south, as ground conditions improve, or in areas where more than 0.5 m of engineered filling is present below footings. This will need to be delineated and further analysed during future geotechnical investigation for the southern portion of the site.

Raft slabs constructed on a layer of engineered filling may be suitable to spread loads and avoid the use of piles in some areas. Slabs should be configured to transfer a maximum pressure of 10 kPa to the underlying soils.

A minimum of 0.5 m of engineered filling should be present beneath the slab to allow bridging over the underlying weak soils. Recommendations for the preparation of the bridging layer are presented in Section 7.9 of this report. Addition of 0.5 m of filling will result in settlements which are estimated to be in the order of about 25 mm. Due to the generally sandy nature of the soils, the majority of settlement is estimated to occur during construction. Very soft to soft clay was encountered in CPT 1. Consolidation of soft clay will not occur as quickly as settlement of sandy soils. Therefore, additional testing and analysis may be required in the area of the proposed boat ramp and workshop to assess the rate and magnitude of settlement in this part of the site.

For raft slabs proportioned for the maximum allowable bearing pressure of 10 kPa, settlement, additional to that caused by the filling, is estimated to be in the order of about 25 mm (ie. total of about 50 mm). If slabs are proportioned for an allowable bearing pressure of 5 kPa, then the additional settlement is estimated to be in the order of 15 mm (ie. total of about 40 mm). The majority of settlement attributable to the structural loads is similarly estimated to occur during construction for the sandy profiles. Consolidation of soft clay, such as that found in CPT 1 is expected to occur over a longer period of time. Differential settlements between similarly sized and loaded footings are expected to be approximately one-half to two-thirds of the total settlement.

Opportunities to reduce post-construction settlement include:

- undertake settlement monitoring of the engineered filling and commence construction once settlement has slowed to an acceptable rate;
- surcharge the area by placing a pre-determined additional depth of granular filling (also called a pre-load), to accelerate settlement, then remove the surcharge after an appropriate proportion of the settlement has occurred. A bridging layer will still likely need to remain in place;
- construct a piled raft.

The above options will require additional assessment if they are to be considered further.

Excavations for footings will need to consider the presence of acid sulphate soils (Ref 1).

If the settlements cannot be tolerated, or if the site cannot accommodate the inclusion of a bridging layer, then slabs will need to be fully suspended and supported on piles.

7.5 Deep Footings

7.5.1 General

Deep footings (piles) will be required to support the proposed marina walkways and breakwater.

Most structural loads within the Marina Village, and Blocks A to D will also need to be carried on piles. Most piles will need to be supported on, or in, the underlying bedrock, which was encountered at depths ranging from about 9.6 m to 13.1 m in this part of the site.

It is understood that driven tubular steel piles are the preferred pile type for the marina structures, and bored concrete piles are the preferred pile type for the on-land buildings in the Marina Village and Blocks A to D.

Due to the presence of saturated sand within the on-shore profiles, unsupported bored pile holes will likely collapse and hence are not considered suitable. Suitable pile types, along with their potential benefits and limitations are follows:

- ***Bored Concrete Piles***: installation of bored piles will require the use of temporary or permanent liners to support the water charged sandy soils. Alternatively the piles could be formed under bentonite, with the concrete placed by tremie method, provided the design pressures are reduced by 20% to allow for reduction in shaft adhesion and the absence of inspection/checking of the pile base. It is likely that casing will need to be driven ahead of the pile boring, particularly in the upper 5 m.

- ***Concrete Screw-cast Piles***: a concrete screw-cast pile is screwed into the ground its natural pitch so that the soil is displaced rather than removed. After reaching its intended depth, the reinforcement cage is placed down the centre stem of the auger and the mandrel is filled with concrete as the auger is backed out, again at natural pitch. Piling contractors provide concrete screw-cast piles as proprietary products, eg. Frankipile's Atlas piles;

- ***Driven Piles***: Select driven pile types would generally drive with relative ease through the soils, although some of the gravelly bands may prove problematic in some areas (eg. Bore 102). The geotechnical capacity of piles driven to refusal on rock approaches the structural capacity of the pile, which is dependent on the pile type and the area of the section used.

7.5.2 Marina Village and Blocks A to D

Estimates of geotechnical pile capacity have been made using the CPT results for a range of diameters for bored concrete piles and concrete screw-cast piles. Once structural loads are known, other pile types and/or diameters can be analysed for suitability. The estimated capacities for various single piles are shown on the attached pile capacity charts. The charts do not include the 20% reduction, as discussed above, and this will need to be taken into account by the designer.

R_{ug} is the ultimate geotechnical strength, which was calculated using static theory, and therefore represents an estimate only. The geotechnical strength reduction factor, ϕ_g, depends on a number of factors including the extent of investigation, type of analysis and testing regime during construction. For the estimates presented above, a ϕ_g = 0.55 was adopted. Higher values of ϕ_g may be justifiable if sufficient load testing is conducted as per AS2159-1995 (Ref 6). The traditional 'allowable' capacity is related to 'working' load and is generally lower than R^*_g, depending on the structural factors applied to determine S*. Allowable (working) capacities may be estimated as approximately 75% of R^*_g.

If the structural loads require socketing into bedrock, then the following parameters maybe used:

Table 7.2 – Indicative Rock Strength Parameters for Pile Design

Project Component	Rock Strength	Approximate Range of Depths to Top of Rock Layer as Encountered in Bores (m)	Allowable Shaft Adhesion (kPa)	Allowable End Bearing Pressure (kPa)
Marina Village	Extremely low strength	11.4 – 12.8	40	550
	Low strength	13.6 – 15.2	120	1200
	Medium strength	16.0 – 19.0	280	2800

The above rock strength parameters include a 20% reduction of typical values, based on the assumption that inspection/checking of the pile base will be difficult.

At the time of the field investigation, the location of on-land structures had not been confirmed, and as such the cored bores (Bores 101 and 102) no longer fall within the footprint of the tallest buildings. It is recommended that the depth to, and presence of, the above listed rock strength layers are confirmed by targeted geotechnical investigation during the detailed design stage of the project.

For some of the smaller buildings, and depending on the structural loads, timber piles driven to refusal on the underlying bedrock could be used to support the proposed loads. It should be noted however, that splicing of the piles may be required if they are not available in lengths which would allow a single pile to be driven to the expected rock depths.

Piles driven to refusal on rock will approach the structural capacity of the pile. The following table shows an extract from a Koppers handbook regarding the structural capacity for softwood and hardwood timber piles of various diameters.

Table 7.3 – Maximum Safe Loads for Treated Softwood Piles (kN)

Pile Type		Pile Toe Diameter (mm)			
		125	**150**	**175**	**200**
De-barked Piles	**F11**	82	126	182	250
	F14	100	153	220	304
Peeled Piles	**F11**	74	113	163	225
	F14	90	138	198	274

Table 7.4 – Maximum Safe Loads for Treated Hardwood Piles (kN)

Pile Type	Pile Toe Diameter (mm)			
	150	**210**	**250**	**300**
F27 Stress Grade	362	710	1007	1450
F17 Stress Grade	230	450	638	919

It should be noted that vibrations associated with pile driving can lead to settlement of soil profiles, especially in very loose and/or saturated soils. Accordingly there is a risk of damage to adjacent structures during pile driving, depending on the construction sequence.

The capacity of driven piles should be proven by the installation method and the opportunity to apply dynamic testing, such as wave equation analysis.

7.5.3 Marina

The estimated loads for the marina and boardwalk structures were not known at this time.

The proposed driven tubular piles are expected to be appropriate for the proposed marina and boardwalk structures, provided that an appropriately sized section can be selected for the structural loads. It is not known at this stage whether penetration into rock will be required to carry the structural loads.

Piles driven to virtual refusal will approach the structural capacity of the piles. Prospective piling contractors should confirm the expected rock penetration and pile capacities achievable with their equipment. The actual load carrying capacity of driven piles should be checked from the results of pile driving sets during construction based on a suitable dynamic method.

Bedrock was encountered at depths of between 5.8 m and 7.9 m below the lake bed in Bores 201 to 203, and refusal was encountered at depths of between 9.6 m and 12.6 m in each of CPTs 1 to 3, which were undertaken near the lake edge.

The following indicative parameters may be used for marina pile design if socketing is required.

Table 7.5 – Indicative Rock Strength Parameters for Pile Design

Project Component	Rock Strength	Approximate Range of Depths to Top of Rock Layer as Encountered in Bores (m)	Allowable Shaft Adhesion (kPa)	Allowable End Bearing Pressure (kPa)
Marina	Extremely low strength	5.8 – 7.9	40	550
	Very low strength	6.4 – 11.0	120	1200

7.5.4 Settlement

Pile settlement will depend on the applied working load, but is expected to be less than about 1% to 2% of the pile diameter for the loads in the above tables.

7.6 Soil Aggressivity

With reference to Tables 6.1 and 6.3 in AS 2159 (Ref 6), piles in water would be classified as follows:

Table 7.6 – Exposure Classification Piles in Seawater

Pile Type	Exposure Condition	Exposure Classification
Steel Piles	Seawater – submerged	Severe
	Seawater – tidal/splash zone	Very severe
Concrete Piles	Seawater – submerged	Moderate
	Seawater – tidal/splash zone	Severe

For piles in soil, the results of laboratory testing suggest the following exposure classifications:

- steel piles in soil – non-aggressive to mild;
- concrete piles in soil – mild to moderate.

It is noted however that the groundwater within the low-lying areas may be impacted by the adjacent tidal marine water, and hence buried concrete and steel structures should be protected accordingly.

Corrosion protection of the structural elements should be designed by an appropriately qualified engineer.

7.7 Mine Subsidence

The site lies within the West Lake Mine Subsidence District, and as such, the proposed development will require the approval of the Mine Subsidence Board. The Mine Subsidence Board (MSB) has indicated that although the current proposal exceeds surface development guidelines, they would consider development of structures up to seven storeys in height (including basement) (refer attached correspondence from MSB).

Discussions between the client and the Mine Subsidence Board indicates that there are no previous workings located beneath the site. Approval would, however, be subject to the structural design accommodating the following parameters, to minimise potential damage if mining were to extend below the site in the future:

(a)	maximum vertical subsidence	150 mm;
(b)	maximum ground strains	±2 mm/m;
(c)	maximum tilt	2 mm/m.

The MSB will require submission of final structural design drawings prior to construction, and also a structural engineer's work-as-executed certification on completion.

Additional details are contained within correspondence from the MSB, copy attached.

7.8 Excavations

It is understood that bulk excavations area are not proposed within the low-lying Marina Village and Blocks A to D. Excavations are possible for installation of buried services, construction of footings and swimming pools, at locations yet to be finalised.

Excavations within this area of the site will likely encountered wet or saturated soils and groundwater, and will need to consider the presence of acid sulphate soils (Ref 1).

Excavations of up to about 3 m are shown for Blocks E to G. It is possible that as the project progresses uphill, rock will become more shallow, and has the potential to be encountered during excavation.

Excavations will need to be supported, and may encounter groundwater. Methods of support for excavations should be further assessed during the design stage of the project.

7.9 Site Preparation

Due to the poor ground conditions, it is anticipated that initial site preparation in the low-lying Marina, Marina Village and Blocks A to D could prove problematic, depending on the size of equipment used, and the prevailing weather conditions at the time of construction.

The field work for this investigation followed a period of relatively fine weather, however the CPT rig, drill rig and a crane used to lift the barge into the water, each came very close to becoming bogged in the low lying areas of the site, and each left ruts in the ground after being positioned in one location for a period of time.

It is considered that the upper topsoil forms a partial crust over the underlying loose and wet sandy soils, and hence should not be completely removed.

Therefore, care will be required when stripping topsoil prior to construction, to avoid over-stripping of the surface crust. It may also be prudent to consider smaller earthworks equipment for these initial stages of construction.

In any event, given the likely need for larger construction equipment to traverse the site during construction, it is recommended that bridging layers be constructed to create a working platform for construction equipment.

Excavation and replacement of the poor soils is not recommended due to the presence of shallow groundwater, and likelihood that conditions will not improve significantly in the upper 1 m or so of the ground surface. In addition, bulk excavation and replacement will need to consider the presence of acid sulphate soils (Ref 1).

Therefore, it is recommended that a granular bridging layer be placed by carefully stripping the existing vegetation and then placing, spreading and compacting an appropriate granular material, such as recycled crushed concrete or similar, to form the bridging layer. It is possible that the bridging layer may need to be about 0.5 m thick. The incorporation of a geogrid may assist in minimising the thickness of the bridging layer. Construction of a trial pad may assist in determining an appropriate bridging layer thickness for the development of the low-lying areas of the site.

The bridging layer should comprise a granular material with a nominal diameter of less than 150 mm. The selected maximum particle size should consider the need for future excavation through the material for features such as buried services. The bridging layer material should be placed with sufficient fines to avoid the occurrence of voids, and should have a California bearing ratio (CBR) of 15% or greater.

The bridging layer should be compacted to achieve at least 100% dry density ratio (Standard) for the upper 0.3 m. It should be placed under geotechnical inspection and tested in accordance with AS 3798 (Ref 7).

7.10 Engineered Filling

Where raising of site levels is required, filling should be placed as engineered filling if it is to support structural elements, such as footings, slabs, pavements, etc.

The following procedure is recommended for placement of engineered filling:

- remove any topsoil, uncontrolled filling or deleterious materials;
- prepare the site surface as outlined in Section 7.9 above;
- suitable filling should be placed in horizontal layers not exceeding 300 mm loose thickness and compacted to a dry density ratio of at least 100% Standard for clayey soils and 80% density index for sandy soils. Moisture content should be in the range -3% OMC (dry) to OMC, where OMC is the optimum moisture content at standard compaction.

Geotechnical inspections and testing should be performed during construction.

7.11 Pavements

7.11.1 Preliminary Pavement Thickness Design

The following preliminary pavement thickness design is in accordance with Austroads (Ref 8) and AP-T36/06 (Ref 9).

The field testing indicates that natural subgrade soils are likely to comprise sandy soils. Based on the poor ground conditions, it has been recommended that at a bridging layer of at least

0.5 m thickness be placed over the natural site soils to improve accessibility. Therefore, the bridging layer will act as a 'select subgrade' layer in proposed pavement areas.

For the purpose of the preliminary pavement thickness design, a subgrade CBR of 5% has been adopted for the natural sandy soils, based on previous experience. A CBR of 15% has been adopted for the select subgrade, based on the recommendations presented in Section 7.9, above. This will result in an effective subgrade CBR of about 8%, which will be used for the preliminary pavement thickness design.

Indicative traffic loadings have been adopted from AP-T36/06 (Ref 9) based on the following:

Street Type (as defined in Ref 9)	Possible Application	Indicative Design Traffic (ESA)
"Minor with two lane traffic"	Carpark and driveway areas subject only to light vehicle traffic (ie. cars up to 3 tonnes)	8×10^3
"Local access in industrial area"	Driveways which include delivery vehicles	3×10^5

ESA – equivalent standard axles

It is important that the pavement areas are carefully considered and separated into those areas likely to see truck traffic and those that are unlikely to see truck traffic. If trucks are allowed to traffic pavement areas which have been designated for car traffic, there is a risk of reduced design life and pavement damage. The above loadings are not applicable for traffic such as forklifts, loaders, etc. Heavy duty pavement areas will require specific pavement design once vehicle types and loads are known.

The above traffic loadings should be reviewed as more detailed information on traffic loading becomes available. In particular, the likely number and types of trucks should be confirmed to assess the suitability of the suggested pavement thickness.

The recommended pavement thickness design is as presented in Table 7.7, below.

Table 7.7 – Preliminary Indicative Pavement Thickness

Pavement Layer	Indicative Thickness (mm)	
	Effective Subgrade (CBR ≥8%)	
	Main Driveways (3×10^5 ESA)	Carpark (8×10^3 ESA)
Wearing course	40[1]	30[2]
Basecourse	115	100
Subbase	100	100
Total	**225**	**200**

Notes to Table 7.7:

* Where asphalt is to be used as a wearing course, a 7 mm prime seal should first be laid
1 – AC 14 or equivalent
2 – AC 10 or equivalent

The pavement thicknesses presented above are dependant on the provision and maintenance of adequate surface and subsurface drainage. Depending on finished levels, subsoil drainage may be required beneath pavement areas.

7.11.2 Material Quality and Compaction Requirements

Recommended pavement material quality and compaction requirements are presented in Table 7.8, below.

Table 7.8 – Material Quality and Compaction Requirements

Pavement Layer	Material Quality	Compaction
Basecourse	CBR > 80%, PI ≤ 6%, Grading in accordance with RTA Form 3051 or Ref 9	Compact to at least 98% dry density ratio Modified (AS 1289.5.2.1)
Subbase	CBR > 30%, PI ≤ 12%, Grading in accordance with RTA Form 3051 or Ref 10	Compact to at least 95% dry density ratio Modified (AS 1289.5.2.1)
Select subgrade (bridging layer)	CBR ≥15%	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1)
Natural sandy subgrade	CBR ≥5%	Compact to at least 80% density index (AS 1289.6.2.1)

7.11.3 Subgrade Preparation

The subgrade should be prepared in accordance with the site preparation measures presented in Section 7.9 above, so that a minimum of 0.5 m of select subgrade is present beneath the top of subgrade level.

Geotechnical inspections and testing should be performed during construction, in accordance with AS 3798 (Ref 7).

8. LIMITATIONS

Conditions on site different to those identified during this assessment may exist. Therefore Douglas Partners Pty Ltd (DP) cannot provide unqualified warranties nor does DP assume any liability for site conditions not recorded in the data available for this assessment.

This report and associated documentation and the information herein have been prepared solely for the use of Johnson Property Group Pty Ltd. Any reliance on this report assumed by other parties shall be at such party's own risk. Any ensuing liability resulting from use of the report by other parties cannot be transferred to DP.

DOUGLAS PARTNERS PTY LTD

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REFERENCES

1. Douglas Partners Pty Ltd Report "Acid Sulphate Soil Assessment, Proposed Trinity Point Marina and Tourist Development, 49 Lakeview Road, Morisset Park", Project 39823A, December 2007.

2. Douglas Partners Pty Ltd Report "Geochemical Assessment, Proposed Trinity Point Marina, Morisset Park, Lake Macquarie", Project 39823B, December 2007.

3. Douglas Partners Pty Ltd Draft Report "Waste Classification, Proposed Trinity Point Marina and Marina Village, 49 Lakeview Road, Morisset Park", Project 39823C, November 2007.

4. Australian Standard AS 2870-1996 "Residential Slabs and Footings – Construction", Standard Association of Australia.

5. Cement & Concrete Association of Australia, TN61 "Articulated Walling".

6. Australian Standard AS 2159-1995 "Piling – Design and Installation", Standards Australia.

7. Australian Standard AS 3798 2007: Guidelines on Earthworks for Commercial & Residential Development.

8. "Pavement Design: A Guide to the Structural Design of Road Pavements", AUSTROADS 2004.

9. AP/T-36 "Pavement Design for Light Traffic: A supplement to AUSTROADS Pavement Design Guide" AUSTROADS 2006.

10. ARRB Special Report 41 "A Structural Design Guide for Flexible Residential Street Pavements", Australian Road Research Board, April 1989.

Foundation Maintenance and Footing Performance: A Homeowner's Guide

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction
There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion
All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation
This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil
All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure
This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

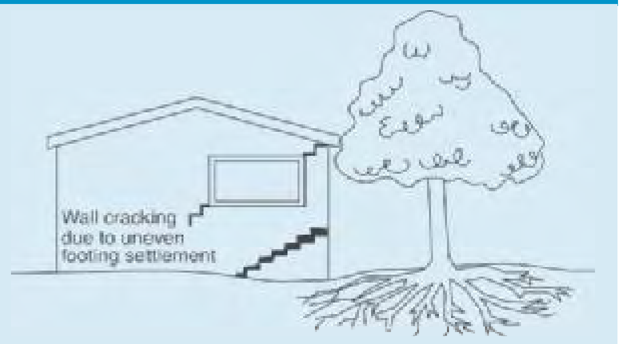
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

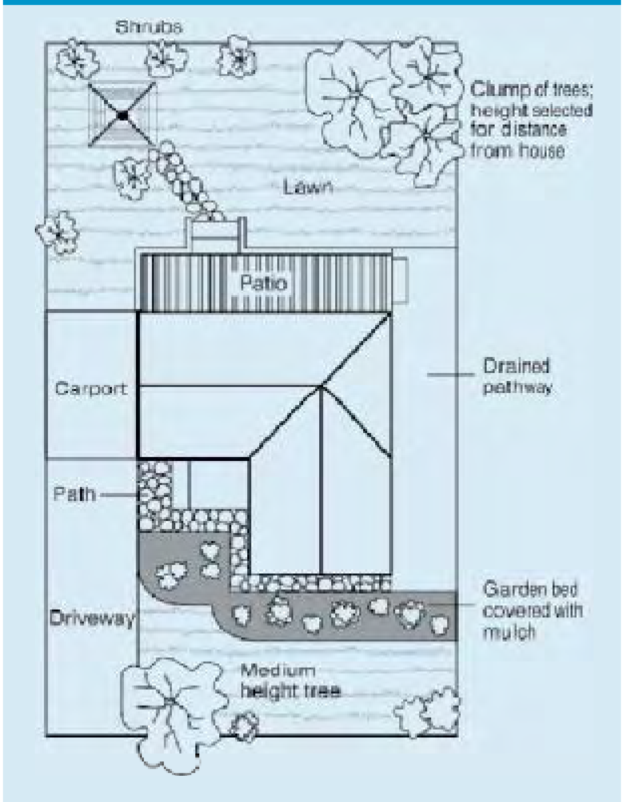
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation
In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden
The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees
Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs
State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation
Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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NOTES RELATING TO THIS REPORT

Introduction

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

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT “N” Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil 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 are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are 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 ground water.

Non-core Rotary Drilling — the hole is 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.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. 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.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining 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 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

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

  as 15, 30/40 mm.

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

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end 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 friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical 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 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

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

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

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

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is 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.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, 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. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

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 condition, 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 depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

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 than 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.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

AN ENGINEERING CLASSIFICATION OF SEDIMENTARY ROCKS IN THE SYDNEY AREA

This classification system provides a standardized terminology for the engineering description of the sandstone and shales in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Under this system rocks are classified by Rock Type, Degree of Weathering, Strength, Stratification Spacing, and Degree of Fracturing. These terms do not cover the full range of engineering properties. Descriptions of rock may also need to refer to other properties (e.g. durability, abrasiveness, etc.) where these are relevant.

ROCK TYPE DEFINITIONS

Rock Type	Definition
Conglomerate:	More than 50% of the rock consists of gravel sized (greater than 2mm) fragments
Sandstone:	More than 50% of the rock consists of sand sized (.06 to 2mm) fragments
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06mm) granular particles and the rock is not laminated
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated
Shale:	More than 50% of the rock consists of silt or clay sized particles and the rock is laminated

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, e.g. clayey sandstone, sandy shale.

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh	Fs	Rock substance unaffected by weathering, limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

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 Society of Rock Mechanics (Reference).

Strength Term	Is(50) MPa	Field Guide	Approx. qu MPa*
Extremely Low:		Easily remoulded by hand to a material with soil properties	
	0.03		0.7
Very Low:		May be crumbled in the hand. Sandstone is "sugary" and friable.	
	0.1		2.4
Low:		A piece of core 150 mm long x 50 mm 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		7
Medium:		A piece of core 150 mm long x 50 mm dia. can be broken by hand with considerable difficulty. Readily scored with knife.	
	1		24
High:		A piece of core 150 mm long x 50 mm dia. cannot be broken by unaided hands, can be slightly scratched or scored with knife.	
	3		70
Very High:		A piece of core 150 mm long x 50 mm dia. may be broken readily with hand held hammer. Cannot be scratched with pen knife.	
	10		240
Extremely High:		A piece of core 150 mm long x 50 mm dia. is difficult to break with hand held hammer. Rings when struck with a hammer.	

* The approximate unconfined compressive strength (qu) shownin the table is based on an assumed ratio to the point load index of 24:1. This ratio may vary widely.

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks

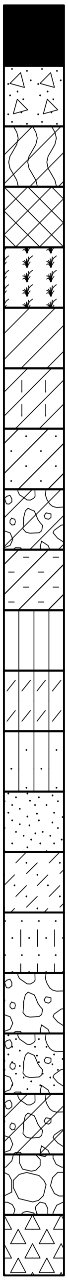
Term	Description
Fragmented:	The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than the core diameter.
Highly Fractured:	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured:	Core lengths are mainly 30 mm - 100 mm with occasional shorter and longer sections.
Slightly Fractured:	Core lengths are generally 300 mm - 1000 mm with occasional longer sections and occasional sections of 100 mm - 300 mm.
Unbroken:	The core does not contain any fracture.

REFERENCE

International Society of Rock Mechanics, Commission on Standardisation of Laboratory and Field Tests, Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index, Committee on Laboratory Tests Document No. 1 Final Draft October 1972

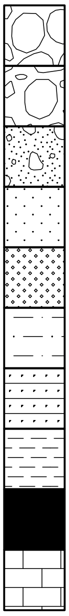
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



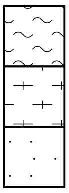
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
SILT
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
CLAYEY GRAVEL
COBBLES/BOULDERS
TALUS

SEDIMENTARY ROCK



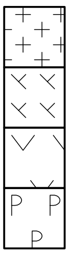
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK



SLATE, PHYLITTE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

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ABBREVIATIONS USED IN DISCONTINUITIES COLUMN OF TEST BORE LOGS

Abbreviation	Meaning
DB	Drill Break
P	Parting
J	Joint
Fr	Fracture
F	Fault
un	Undulating
ro	Rough
H	Healed
pl	Planar
fg	Fragmented
cs lam	Carbonaceous lamination
sm	Smooth
ti	Tight
di	Probably drilling induced
st	Stepped
sl	Slickensided
Fe	Ironstained
hor	Horizontal
V	Vertical
sh	Subhorizontal
sv	Subvertical
cy	clay
ca	calcite

Examples:

1. At 62.04 m, P, 30°, un, st, ro, cs lam
   At 62.04 m Parting, 30°, undulating, stepped, rough, on carbonaceous siltstone lamination

2. At 65.08 m, Fr, 70°, pl, ro, st, fr
   At 65.08 m, fracture, planar, rough, stepped, fragmented.

CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

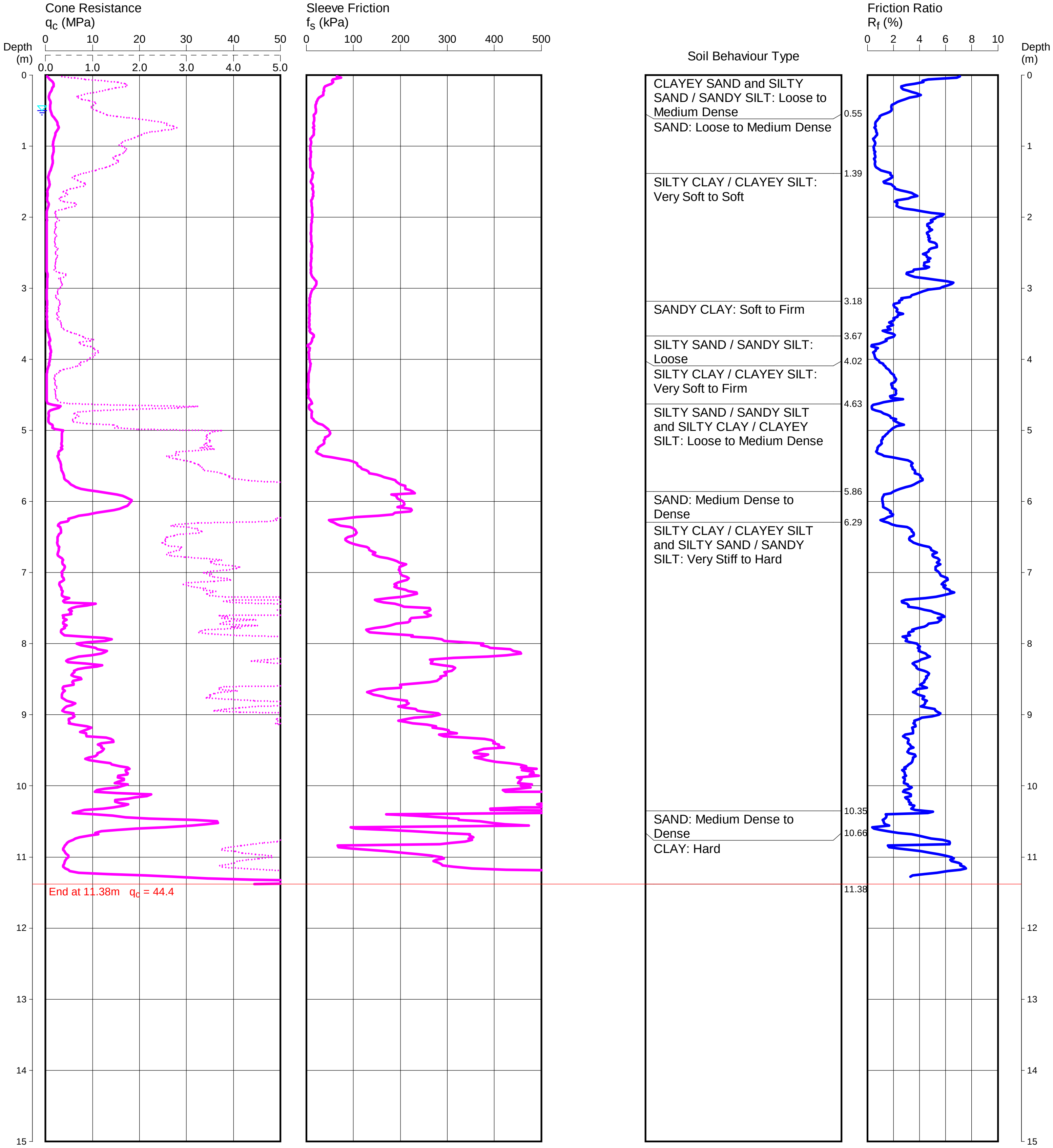
PROJECT No: 39823

CPT 1

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 0.665



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.5 m
MGA Coordinates: E363772.903, N 6334208.428

Date	
Plotted	
Checked	

File: P:\39823\Field\39823-01.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

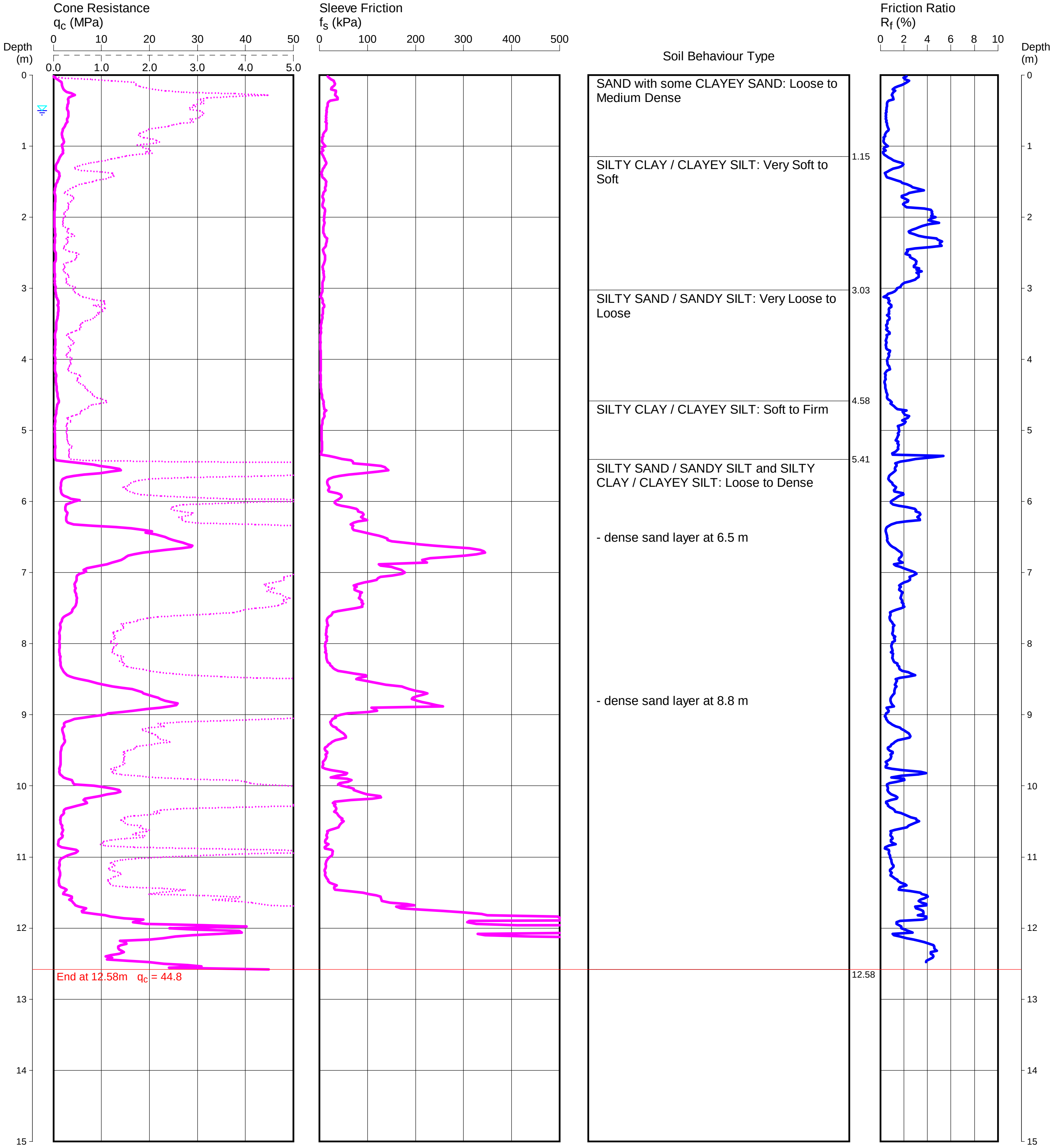
PROJECT No: 39823

CPT 2

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 0.81



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.5m
MGA Coordinates: E363824.4, N6334193.0

Date	
Plotted	
Checked	

File: P:\39823\Field\39823-02.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

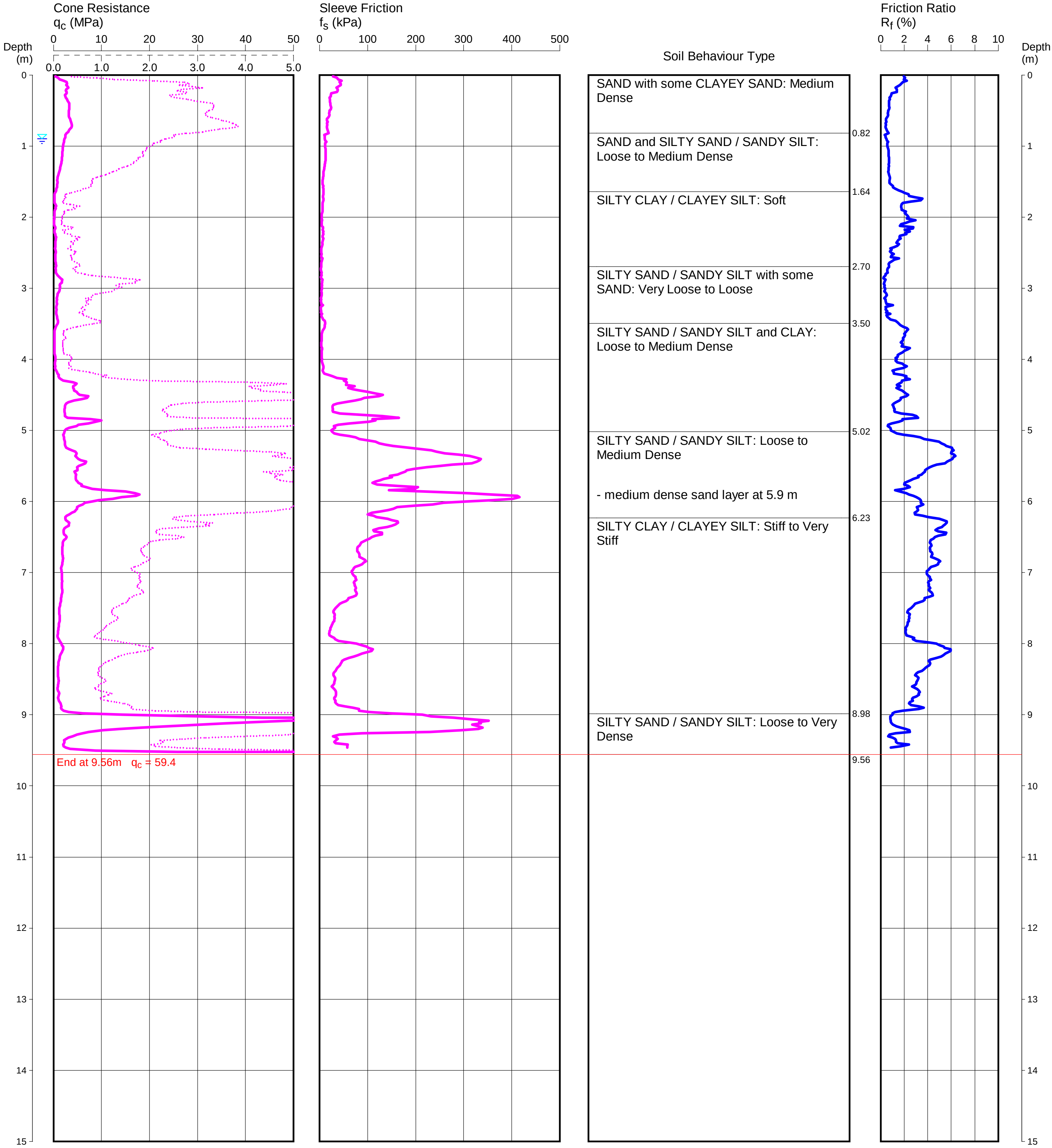
PROJECT No: 39823

CPT 3

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 0.92



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.9 m
MGA Coordinates: E363867.4, 6334172.0

Date	
Plotted	
Checked	

File: P:\39823\Field\39823-03.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

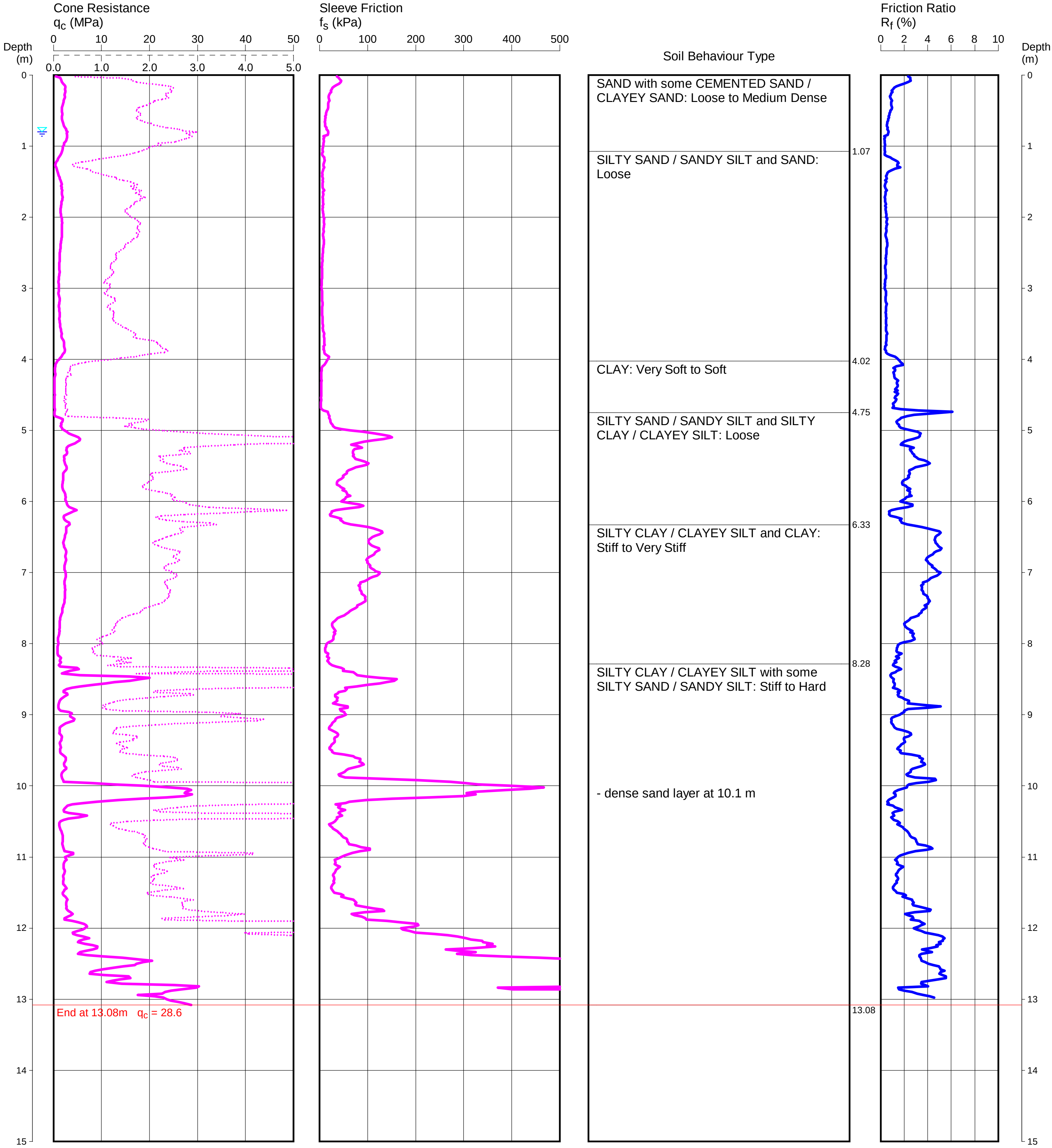
PROJECT No: 39823

CPT 4

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 0.99



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.8 m
MGA Coordinates: E363828.683, N6334161.2

Date	
Plotted	
Checked	

File: P:\39823\Field\39823-04.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

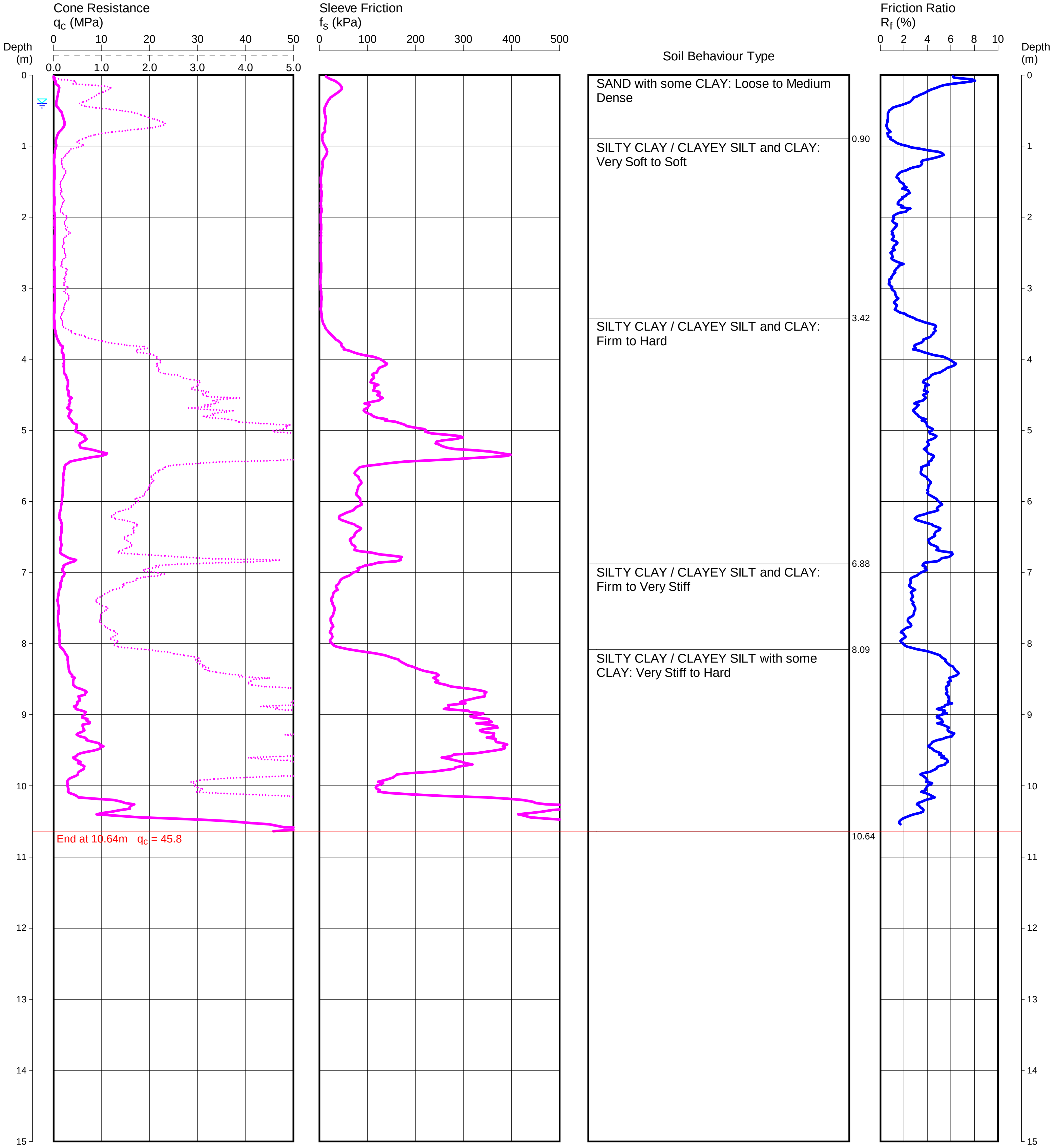
PROJECT No: 39823

CPT 5

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 0.78



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.4 m
MGA Coordinates: E363845.3, N6334130.1

Date	
Plotted	
Checked	

File: P:\39823\Field\39823-05.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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CONE PENETRATION TEST

CLIENT: JOHNSON PROPERTY GROUP

PROJECT: TRINITY POINT MARINA & MIXED USE RESORT

LOCATION: OFF HENRY ROAD, MORISSET PARK

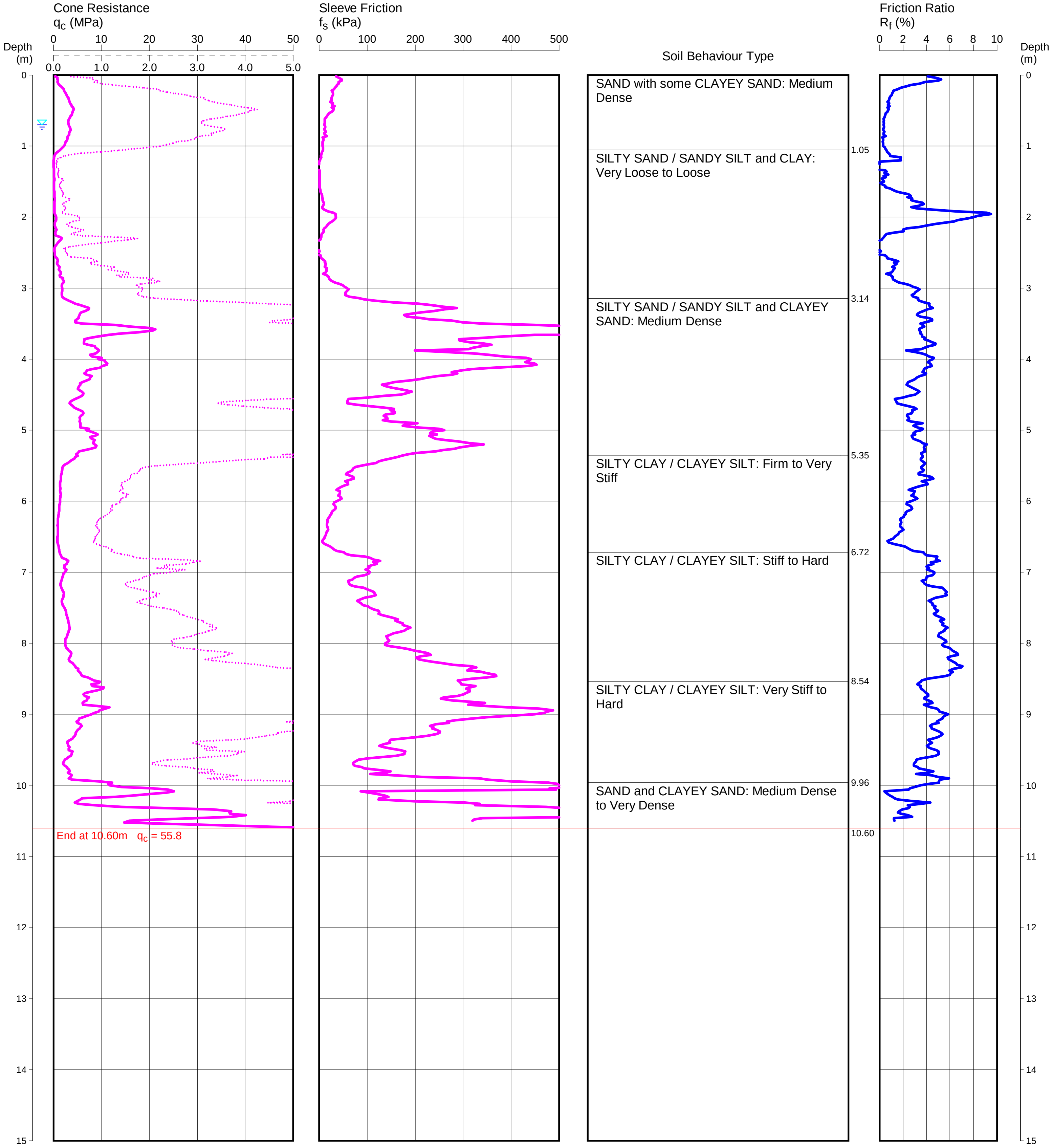
PROJECT No: 39823

CPT 6

Page 1 of 1

DATE 25/09/2007

SURFACE RL: 1.05



REMARKS: DEPTH TO WATER AT COMPLETION OF TEST : 0.7 m
MGA Coordinates: E363877.37, N6334115.6

Date
Plotted
Checked

File: P:\39823\Field\39823-06.CP5
Cone ID: 413 **Type:** 2 Standard

ConePlot Version 5.8.1

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BOREHOLE LOG

CLIENT: Johnson Property Group
PROJECT: Trinity Point Marina & Tourist Development
LOCATION: 49 Lakeview Road, Morisset Park

SURFACE LEVEL: 1.27 AHD
EASTING: 363834
NORTHING: 6334174
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 39823
DATE: 26/9/07
SHEET 1 OF 2

Depth (m)	Description of Strata	Water	Sampling Type	Test Results & Comments
0.0–0.35	FILLING: Generally comprising brown fine to coarse grained gravelly silty sand, humid		A	
0.35–1.7	GRAVELLY SAND: Very loose to loose grey-brown fine to coarse grained gravelly sand, with trace silt and clay, damp. From 0.6m, moist to wet. From 1.0m, saturated	▼ (1.0m)	A S	1,0,1 N = 1
1.7–3.0	GRAVELLY CLAYEY SAND: Very loose to loose grey-brown fine to coarse grained gravelly sand, with some silt, shell fragments, saturated		S	1,0,0 N = 0
3.0–4.05	GRAVEL: Loose grey and brown fine to medium sized gravel, with some sand and shells and trace silt, saturated			
4.05–5.5	GRAVELLY SAND: Loose grey fine to medium grained silty gravelly sand, with some shells, saturated		S	5,2,2 N = 4
5.5–6.3	GRAVELLY CLAY: Very stiff to hard grey-brown and brown gravelly clay, with some sand, M~Wp		S	5,14,16 N = 30
6.3–7.0	GRAVELLY SANDY CLAY: Very stiff light grey-brown gravelly sandy clay, M~Wp			
7.0–7.8	SILTY CLAY: Very stiff grey-brown and red-brown silty clay, M~Wp		S	3,7,12 N = 19
7.8–	SANDY SILTY CLAY: Firm to stiff grey-brown sandy silty clay, with some gravel, M~Wp. From 8.55m to 8.8m, soft to firm		pp pp S pp	30-50 kPa 30-50 kPa 1,0,4 N = 4 80-100 kPa

RIG: Scout 2 **DRILLER:** Ground Test (Driver) **LOGGED:** Reid **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (tc-bit) to 2.5m, then wash boring to 5.5m; then rotary with mud to 13.25m; then NMLC coring to 19.9m

WATER OBSERVATIONS: Free groundwater observed at 1.0m during drilling

REMARKS: Coordinates are MGA. 50mm diameter Class 18 PVC piezometer installed to 4m; screened from 1.0m to 4.0m; 5mm gravel filter from 0.4m to 4.0m; bentonite plug from surface to 0.4m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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BOREHOLE LOG

CLIENT: Johnson Property Group
PROJECT: Trinity Point Marina & Tourist Development
LOCATION: 49 Lakeview Road, Morisset Park

SURFACE LEVEL: 1.27 AHD
EASTING: 363834
NORTHING: 6334174
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 39823
DATE: 26/9/07
SHEET 2 OF 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
10.0	GRAVELLY SANDY CLAY: Stiff grey-brown gravelly sandy clay, M~Wp		S pp			120-140 kPa 5,4,6 N = 10
			S			7,4,11 N = 15
	From 11.9m, stiff to very stiff					
12.8	CONGLOMERATE: Extremely low strength, extremely weathered orange-brown and light grey conglomerate		S			23,25/80mm
	From 13.25m, extremely low to very low strength, extremely to highly weathered	13.32m: P, 5º, ro, un 13.41m: P, 5º, ro, un 13.64m: J, 10º, ro, un	C	100	73	PL(A) = 0.67MPa PL(D) = 0.26MPa
	From 13.56m to 13.59, low strength					
	From 13.7m, low to medium strength, highly to moderately weathered	14.85m: J, 45º, ro, un	C	100	94	
15.25	CORE LOSS:	15.25m: CORE LOSS: 50mm				
15.3	CONGLOMERATE: Medium strength, moderately weathered brown conglomerate	15.77m: J, 10º, sm, pl 16.7m: P, 5º, sm, pl, Fe	C	98	98	PL(A) = 0.7MPa PL(D) = 0.35MPa
17.15	CLAYSTONE: Very low strength, moderately weathered brown conglomerate	17.15m: P, 10º, ro, pl, Fe				PL(A) = 0.07MPa PL(D) = 0.05MPa
17.9	PEBBLY SANDSTONE: Low strength, moderately weathered light grey fine to coarse grained pebbly sandstone	17.9m: P, 5º, ro, pl	C	97	90	
18.0	CORE LOSS:	18m: CORE LOSS: 50mm				
18.05	PEBBLY SANDSTONE: Extremely low strength, moderately weathered light grey fine to coarse grained pebbly sandstone	From 18.05m to18.15m, highly Fg (1mm to 10mm)				
	From 18.45m, medium to high strength		C	100	100	PL(A) = 1.57MPa PL(D) = 1.06MPa
19.9	Bore discontinued at 19.9m, limit of investigation					

RIG: Scout 2 **DRILLER:** Ground Test (Driver) **LOGGED:** Reid **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (tc-bit) to 2.5m, then wash boring to 5.5m; then rotary with mud to 13.25m; then NMLC coring to 19.9m

WATER OBSERVATIONS: Free groundwater observed at 1.0m during drilling

REMARKS: Coordinates are MGA. 50mm diameter Class 18 PVC piezometer installed to 4m; screened from 1.0m to 4.0m; 5mm gravel filter from 0.4m to 4.0m; bentonite plug from surface to 0.4m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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Bore 101 – 13.25 m to 19.9 m

BOREHOLE LOG

CLIENT: Johnson Property Group
PROJECT: Trinity Point Marina & Tourist Development
LOCATION: Off Henry Street, Trinity Point

SURFACE LEVEL: 1.27
EASTING: 363834
NORTHING: 6334174
DIP/AZIMUTH: 90°/--

BORE No: 101A
PROJECT No: 39823
DATE: 16 Oct 07
SHEET 1 OF 1

Depth (m)	Description of Strata	Water	Well Construction Details
0.3	FILLING: Generally comprising brown fine to medium grained gravelly silty sand, humid		From 0.05m to 0.4m, bentonite
1.7	GRAVELLY SAND: Very loose to loose grey-brown fine to coarse grained sand, some silt and clay, damp From 0.65m, wet to saturated	▼ (1.15m)	
3.0	GRAVELLY CLAYEY SAND: Very loose to loose grey-brown fine to coarse grained gravelly sand, with some silt, saturated		From 0.4m to 3.5m, 5mm gravel filter From 0.6m to 3.5m, screen
3.5	GRAVEL: Loose grey-brown fine to coarse gravel, with some sand and trace silt, saturated		
	Bore discontinued at 3.5m, limit of investigation		

Sampling & In Situ Testing: Type | Depth | Sample | Results & Comments

RIG: Truck mounted rig **DRILLER:** Atkins **LOGGED:** Karpiel **CASING:** -
TYPE OF BORING: 150mm hollow flight auger
WATER OBSERVATIONS: Free groundwater observed at 1.15m below ground level during drilling
REMARKS: Endcap dislodged during removal of casing, screen backfilled inside well to 1.84m below ground level

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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BOREHOLE LOG

CLIENT: Johnson Property Group
PROJECT: Trinity Point Marina & Tourist Development
LOCATION: 49 Lakeview Road, Morisset Park

SURFACE LEVEL: 0.89 AHD
EASTING: 363828.6
NORTHING: 6334140.7
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 39823
DATE: 08 Oct 07
SHEET 1 OF 2

Degree of Weathering: EW, HW, MW, SW, FS, FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
Fracture Spacing (m): 0.01, 0.05, 0.10, 0.50, 1.00
Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

Depth (m)	Description of Strata	Water	Sampling Type	Test Results & Comments
0.0	TOPSOIL: Generally comprising dark brown-black clayey sandy silt, with trace rootlets to 0.2m, damp			
0.4	SILTY SAND: Dark brown fine to medium grained silty sand, damp		A	
0.7	SAND: Very loose brown fine to medium grained sand, with trace silt, clay and shell fragments, moist		A	
			S	1,0,0 N = 0
1.3	SILTY SAND: Very loose grey fine to medium grained silty sand, with trace clay and shell fragments, saturated	▼ (1.3m)		
			S	0,0,0 N = 0 (weight of hammer)
			S	3,1,2 N = 3
4.2	CLAYEY SAND/SANDY CLAY: Very soft grey-brown medium grained clayey sand/sandy clay, saturated			
4.7	SILTY SANDY CLAY: Firm to stiff light brown sandy clay, with fine grained gravel, M>Wp			
5.3	SILTY CLAY: Firm to very stiff light brown silty clay, with some sand, M<Wp		pp pp S pp	80 kPa 150 kPa 3,3,6 N = 9 320 kPa
			S	3,8,25/130mm
7.15	GRAVELLY SANDY CLAY: Very stiff to hard light brown gravelly sandy clay, M~Wp			
8.0	SILTY CLAY: Stiff light brown silty clay, with some fine grained sand, M>Wp		S pp	3,4,5 N = 9 170 kPa
	From 9.5m, very stiff to hard, slightly sandy			

RIG: Scout 2 **DRILLER:** Ground Test (Driver) **LOGGED:** Reid **CASING:** HW to 7.2m, HQ to 11.65m
TYPE OF BORING: 100mm diameter solid flight auger (tc-bit to 4.5m), then rotary wash boring to 11.65m, then NMLC coring to 17.75m
WATER OBSERVATIONS: Free groundwater observed at 1.3m during drilling
REMARKS: Coordinates are MGA. 50mm diameter Class 18 PVC piezometer installed to 4.0m depth on completion

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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