REPORT

TO
MOORGATE FINANCE PTY LTD

ON
GEOTECHNICAL ASSESSMENT

FOR
PROPOSED RESIDENTIAL DEVELOPMENT

AT
7 LAWRENCE STREET AND
18 MARMORA STREET, FRESHWATER, NSW

21 March 2014
Ref: 27296ZHrpt

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1 INTRODUCTION

This report presents the results of a geotechnical assessment and limited scope geotechnical investigation for the proposed residential development at 18 Marmora Street, Freshwater, NSW. The assessment was commissioned by Mr David Thompson of Moorgate Finance Pty Ltd (MF) by signed 'Acceptance of Proposal' form, dated 4 March 2014. The commission was on the basis of our proposal, Ref: P38387ZN, dated 4 March 2014.

We were also commissioned by MF on 10 March 2014 to drill four boreholes and install two PVC standpipes for the purpose of groundwater monitoring. The commission was on the basis of our email sent to Mr Sam Petinsky of MF on 7 March 2014.

To assist with the preparation of this report, we have been supplied with, or have referred to, the following information:

1. A previous geotechnical investigation report prepared by GeoEnviro Consultancy Pty Ltd (GeoEnviro), Ref: JG07011A, dated July 2007. This previous report was completed for a different proposed development at the subject site;
2. A previous geotechnical investigation report completed by JK Geotechnics, Ref: 22337Srptrev4, dated 28 November 2013. This previous report was completed for a proposed mixed use development surrounding and including the subject site;
3. Preliminary architectural drawing extracts of the current proposed development prepared by Benson McCormack Architects [BMA] (Project Number: 1353A, Drawing Numbers: A-0100^{A}, A-0101^{A}, A-0102^{A}, ASK012 and A-0221^{A}, dated January or February 2014); and
4. A survey plan of the site prepared by Kiprovich & Associates Pty Ltd (Plan Number: 07_113A, dated 10 April 2007).

Based on the supplied architectural drawings, we understand that the proposed development will comprise demolition of the existing structures on site followed by construction of two, three storey buildings underlain by a common single level basement. The proposed basement level will be constructed at reduced levels (RL) between 12.10m and 13.0m. To achieve these levels, excavation to depths between about 2.5m (northern end) and 5.5m (southern end) of the proposed basement will be required. Three lifts are also proposed. We have assumed that the lift over-run pits will require a maximum excavation depth of about 1.5m below bulk excavation level. The southern end of the proposed building will extend beyond the proposed basement to the south. The outline of the proposed basement is shown on the attached Figure 1.

Structural loads typical of this type of development have been assumed.

In preparation of this current report, we have referred to seven of the closest previous boreholes drilled on, or immediately adjacent to, the subject site (ie. BH2, BH3 and BH4 from the previous 2007 GeoEnviro geotechnical investigation report and BH205, BH207, BH2078 and BH209 from our previous 2013 geotechnical investigation report). The approximate location of these previous boreholes have been plotted onto the attached Figure 1, which is based on the supplied survey plan. With the exception of BH3 and BH4, the boreholes included diamond coring of the sandstone bedrock. The boreholes were drilled to depths between 3.4m (BH3) and 9.33m (BH205) below existing grade.

The purpose of the assessment was to review the subsurface information indicated on seven previous borehole logs as presented in Appendix A and B of this report and to obtain additional geotechnical information on subsurface conditions at four borehole locations, as a basis for comments and recommendations on excavation conditions and support, groundwater, footings and the basement on-grade floor slab. We have also referred to in this report, the laboratory Point Load Strength Index Test results from our previous investigation which are presented in Appendix B.

2 INVESTIGATION PROCEDURE

This section refers to the current limited scope geotechnical investigation only.

The fieldwork was carried out on 14 March 2014 and comprised the drilling of four boreholes (BH301, BH301a, BH302 and BH302a), at the locations shown on the attached Figure 1, to depths between 2.92m (BH301a) and 4.5m (BH301 and BH302) below existing grade. The boreholes were auger drilled using our track mounted JK305 drill rig, which is equipped for site investigation purposes.

The borehole locations were set out by tape measurements from existing surface features and apparent site boundaries and were positioned as close as practical to the north-western corner of the site, which is where groundwater was encountered in some of the previous boreholes drilled, on and near to the site. The surface RL's indicated on the attached borehole logs were interpolated between spot level heights shown on the supplied survey plan prepared by Kiprovich & Associates Pty Ltd (Plan Number: 07_113A, dated 10 April 2007) and are therefore only approximate. The survey datum is the Australian Height Datum (AHD).

The nature and composition of the subsurface soil and rock horizons were assessed by logging the materials recovered during drilling. The relative compaction and strength of the subsoil profile were assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split-spoon sampler and by tactile examination. The strength of the underlying bedrock was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of recovered auger cuttings and correlations. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Groundwater observations were also made in the boreholes. A 50mm diameter Class 18 uPVC standpipe was installed into BH301a to 2.92m depth and into BH302a to 3.21m depth for groundwater level monitoring purposes. The standpipe installation details are shown on the respective borehole logs.

Our geotechnical engineer (Adrian Callus) was present full-time during the fieldwork to set out the borehole locations, nominate testing and sampling, prepare the attached borehole logs and to direct the standpipe installations. The Report Explanation Notes define the logging terms and symbols used.

Additional geotechnical laboratory testing and contamination testing of site soils and groundwater was outside the scope of this investigation.

On 17 March 2014 (ie. three days after the fieldwork) our geotechnical engineer returned to site to measure the groundwater level in the standpipes and to carry out a rising head infiltration test (also known as a pump-out test) in both standpipes. However, as both standpipes were 'dry', the rising head infiltration tests could not be completed.

3 SUBSURFACE CONDITIONS

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

Generally, the current and previous boreholes encountered topsoil and/or fill overlying natural sands and clays then weathered sandstone bedrock at relatively shallow and moderate depth. Reference should be made to the attached current and previous borehole logs for details at each specific location. A summary of the subsurface characteristics for the seven previous boreholes referred to and the current boreholes is provided below:

Topsoil and/or Fill

Topsoil and/or fill comprising predominantly silty sand was encountered at the surface of each borehole and extended down to depths between 0.1m (BH301, BH301a, BH302 and BH302a) and 1.2m (BH302 and BH302a).

Natural Soils

Natural soils comprising silty sand, clayey sand, sandy clay, silty clay and shaly clay (ie. clayey soil with weathered shale bands) were encountered below the topsoil and/or fill in each borehole. The sandy soils were very loose, loose and medium dense. The clayey soils were stiff, very stiff and hard. Both density and strength increased with depth.

As the SPT 'bounced' in BH2, it is possible that the clayey sand/sandy clay profile logged between 3.2m and 5.0m depth is actually weathered sandstone bedrock.

Weathered Sandstone Bedrock

Weathered sandstone bedrock was encountered in each borehole at depths between 2.9m (BH208) and 6.0m (BH205) below existing grade and extended down to the borehole termination depths.

The sandstone bedrock was mostly distinctly weathered and of low and medium strength. Occasionally the sandstone bedrock was extremely to distinctly weathered and of extremely low to very low strength. High strength sandstone bedrock was encountered in BH2. The strength of the sandstone bedrock was not logged on BH3 and BH4.

The diamond cored portions of BH2 and BH205, BH207, BH208 and BH209 contained defects including extremely weathered bands/seams, clay seams and occasionally inclined joints.

The 'core loss' zones encountered in BH207 and BH208 at depths of 5.0m and 4.87m, respectively, were 100mm (BH207) and 620mm (BH208) thick and are inferred to be extremely weathered bands or clay bands which have 'washed away' during the coring process.

An indicative engineering classification of the sandstone bedrock (in accordance with Pells et al. 1998) has been carried out and is tabulated below. We note that RLs were not shown on the

GeoEnviro borehole logs and therefore the approximate RLs tabulated below for BH2, BH3 and BH4 have been interpolated from spot levels shown on the supplied survey plan (assuming previous and current surface levels were the same):

Borehole	Approx. Surface RL (m)	Indicative Engineering Classification of Sandstone Bedrock Depths (m)				
		Class V	Class IV	Class III	Class II	Class I
BH2	16.1	5.0 – 5.4*	-	-	5.4 – 7.88	-
BH3	16.7	3.0 – 3.4**	-	-	-	-
BH4	18.1	-	4.0 – 4.7*	-	-	-
BH205	18.7	5.8 – 7.3*	-	-	-	7.3 – 9.33
BH207	17.6	3.8 – 5.45*	5.45 – 7.08	-	-	-
BH208	14.5	2.9 – 5.49*	5.49 – 7.77	-	-	-
BH209	17.9	4.6 – 5.4	3.5 – 4.6*	5.4 – 7.12	-	-
BH301	18.7	-	3.2 – 4.5*	-	-	-
BH302	15.5	-	3.7 – 4.5*			

* based (wholly or in part) on the augered portion of the borehole
** Weathering and strength not indicated on augered borehole log – Class V assumed.

We have also shown on the cored borehole logs the various rock Classes.

Groundwater

The following table summarises the groundwater measurements made within the boreholes during the previous and current geotechnical investigations.

Borehole	Groundwater Depths and Reduced Levels (m)		
	During Drilling	On Completion of Auger Drilling	After some time from completion of drilling
BH2	Not recorded	2.8 (RL13.3)	-
BH3	2.8 (RL13.9)	Not recorded	-
BH4	'Dry'	'Dry'	-
BH205	No groundwater observations made during, or on completion of, auger drilling		
BH207	1.8 (RL15.8)	0.7 (RL 16.9) [on completion of coring and after 2 hours]	-
BH208	2.8 (RL11.7)	Not recorded	-
BH209	'Dry'	'Dry'	1.6 (RL16.3) After 3 weeks from completion of drilling 2.85 (RL15.05) on 17 March 2014
BH301	'Dry'	'Dry'	3.45 (RL15.25) after 1.5 hours from completion of drilling
BH301a	'Dry'	'Dry'	2.9 (RL15.8) after three days from the completion of drilling
BH302	'Dry'	'Dry'	-
BH302a	'Dry'	'Dry'	'Dry' after three days from the completion of drilling

We note that water is introduced into the borehole during the coring process and therefore the measured groundwater level on completion of coring in BH207 is almost certainly influenced by the drill flush water. With exception of BH2 where there was only a 50% return of the drill flush water during coring, the remaining cored boreholes showed a near full return, which indicates a relatively impermeable rock mass.

We note that groundwater levels at the time of the previous investigations were at, or just above, the soil/rock interface in BH2, BH3 and BH208.

The current boreholes were predominantly 'dry' during drilling, on completion of drilling and after a short time from completion of drilling, with the exception of BH301 where groundwater was measured just below the soil/rock interface about 1.5 hours from completion of drilling. Groundwater was measured in the base of the standpipe at BH301a at 2.9m depth after three days from the completion of drilling.

The measured groundwater level in BH209 on 17 March 2014 was just above the soil/rock interface and at about the same RL as the groundwater level measured in BH301 shortly after the completion of drilling.

To supplement the previous groundwater level monitoring, a data logger was installed to record groundwater levels in BH209 between March 2010 and October 2012. The groundwater level recordings for BH209 are presented in the attached Appendix B and show recorded groundwater levels fluctuate at that location between about RL13.4m and RL16.2m. We note that there were comments made in the previous report which suggested that there was a 'leak' into the standpipe at that location and most likely explains the rapid groundwater level response to rainfall during the monitoring period, which is unusual in a sand and clay subsurface profile.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

Based on our review of the seven previous boreholes drilled on, or close to, the subject site and with reference to the current subsurface information, we consider the following to be the primary geotechnical issues for the proposed development:

- The excavation cuts will extend through the soil and weathered bedrock profiles and will require support by shoring walls. The shoring walls must be installed prior to the commencement of excavation;
- Excavation for the proposed basement will need to be carried out carefully due to the presence of buried services which pass through, or very close to, the site, as well as the presence of neighbouring structures on or close to the site boundaries. Care must be taken during excavation so as to not damage, undermine or remove lateral support from the property boundaries and neighbouring structures;
- Vibrations will need to be controlled whenever hydraulic impact rock hammers are used during excavation;
- Groundwater seepage is expected at, or just above, the soil/bedrock interface, especially after rainfall, and will therefore need to be controlled. We do not expect the seepage inflows to be continuous, since rainfall does not occur all the time. Pumping of groundwater seepage collected within the basement sumps is only expected to occur on a periodic basis;
- The presence of high strength sandstone bedrock, which will present 'hard' rock excavation and piling conditions; and
- The footing system for the ground floor level which extends beyond the southern side of the basement will require careful consideration.

The above geotechnical issues are addressed in detail in the following sections of this report.

4.2 Dilapidation Surveys

We recommend that dilapidation surveys be completed on all structures within 30m of the proposed development footprint. We also recommend that the condition of the sewer main which is located on the southern side of the basement be assessed by CCTV survey. The dilapidation and CCTV surveys must be carried out prior to the commencement of demolition, excavation and dewatering.

The dilapidation surveys must include detailed internal and external inspections where all defects are rigorously described including defect type, length and width. Colour photographs of the defects must also be provided.

The respective owners should be asked to confirm that the dilapidation reports present a fair record of existing conditions. The dilapidation reports and CCTV survey may then be used as a benchmark against which to assess possible future claims for damage arising from the works. We could prepare a proposal to carry out the dilapidation surveys, if requested.

4.3 Excavation Conditions

Reference should be made to the Code of Practice *'Excavation Work'*, dated July 2012 prepared by Safe Work Australia for guidance on demolition and excavation.

The proposed development will initially require demolition of existing structures within the footprint of the proposed development. Following demolition, all grass, topsoil and any deleterious or contaminated fill within the development footprint should be stripped and disposed appropriately off site. Reference should be made to Section 5 below for guidance on the off-site disposal of soil.

Care must be taken during demolition, subsequent stripping works and excavation, to not remove support from the site boundaries and damage any buried services which pass below, and immediately adjacent to, the subject site. Existing services may require diversion prior to the commencement of excavation or otherwise be temporarily supported during excavation.

During demolition, but prior to the commencement of excavation, we recommend that details be obtained (such as by excavation of test pits and/or review of as-built structural drawings) for any adjoining buildings which are be located within 2H of the bulk excavation, where H is the depth of the excavation. This will enable appropriate consideration to be made during the shoring design phase.

Based on the investigation results, the excavations will encounter the soil and Class V sandstone bedrock profiles. Excavation of the soils may be readily completed using buckets fitted to hydraulic excavators. It will be possible to excavate the Class V bedrock using a 'digging' bucket fitted to a large excavator (at least 30 tonnes). However, ripping tyne and/or rock hammer assistance will be required to excavate low and medium strength bands within the Class V bedrock.

Rock excavations using hydraulic rock hammers, if used, will need to be controlled as there may be direct transmission of ground vibrations to adjoining structures and nearby buried services. We recommend that quantitative vibration monitoring be carried out whenever hydraulic rock hammers are used on this site, as a safeguard against possible vibration induced damage. By referencing the German Standard DIN 4150-3:1999-02, the vibrations along the site boundaries should be limited to a peak particle velocity of 5mm/s, subject to review of the dilapidation survey reports. If it is found that transmitted vibrations are excessive, then it would be necessary to use a smaller rock hammer or further geotechnical advice sought. The following procedures are recommended to reduce vibrations, if rock hammers are used:

- Maintain the rock hammer orientation towards the face and enlarge the excavation by breaking small wedges off the face.
- Operate the rock hammer in short bursts only to reduce the amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should have all appropriate statutory and public liability insurances and should be provided with a full copy of this report.

4.4 <u>Excavation Retention</u>

4.4.1 Support Systems

As the proposed basement will extend to, or close to, the site boundaries, temporary batter slopes through the soil and weathered bedrock profiles will not be possible and therefore the proposed vertical cuts will need to be supported by an engineered shoring system. The shoring system will need to be designed and installed so that adverse impacts on adjacent structures from shoring wall deflections are reduced.

Based on the expected variable subsurface profile comprising sands and clays, we consider the most suitable retention system for this site would be construction of a contiguous pile retaining wall, using grout injected auger piles, otherwise known as continuous flight auger (CFA) piles.

The shoring piles must be founded with sufficient embedment to satisfy stability and founding considerations. Additional lateral restraint in the form of anchors or internal props to reduce deflections may be required, as discussed further below. We recommend that the shoring piles terminate at a depth of not less than 0.5m below bulk excavation level and into sandstone bedrock (including an allowance for footings, services and other localised excavations below bulk excavation level, such as for the proposed lift pits). A greater depth of embedment will probably be required for stability of the shoring wall. The piles can also be used as load bearing piles for the proposed new building, if founded in the appropriate Class of bedrock.

As the shoring piles are to be socketted into the underlying sandstone bedrock, the formation of the rock sockets can result in 'ground loss' due to continued auger rotation during drilling, which may result in withdrawal of sand quantities in excess of the pile volume. Such 'ground loss' can result in surface settlements in the vicinity of the piles.

Construction of the contiguous pile retaining walls should be of high quality, taking care to prevent soil loss through gaps that will most likely occur between the piles, as this would add to the possibility of settlement occurring outside the excavation. Such gaps must be rectified progressively during excavation, such as by mass concrete infill or shotcrete.

Due to the loose nature of the near surface soils in some areas, the drilling of the contiguous piles may cause ground surface movements due to vibrations associated with pile drilling and possible collapse or draw-down of soils into the pile drill holes, as noted above. Care will be required by the piling contractor and builder. Continual monitoring of the ground surface between the contiguous piles and adjoining surface levels should be undertaken by the site foreman. If there are any signs of ground surface movement, particularly when adjacent to neighbouring structures, then the piling operations should be immediately halted and further geotechnical advice sought.

4.4.2 Seepage

Groundwater inflows into the excavation are expected as local seepage flows within the fill, at the fill/natural soil interface, at the soil/rock interface, and through joints and bedding partings within the bedrock profile, particularly after heavy rain. The results of the previous investigation indicate that the area may be susceptible to short term storm surcharge.

Seepage volumes into the excavation are expected to be controllable by conventional sump and pump methods. Notwithstanding, groundwater seepage monitoring should be carried out during excavation so that any unexpected conditions can be timeously addressed.

4.4.3 Retention Design Parameters

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the static design of the retaining walls:

- For allowable bearing pressure recommendations, reference should be made to Section 4.5 below.
- For progressively propped or anchored shoring systems, where minor wall movements can be tolerated (for example, walls which support grass or landscaped areas), a uniform rectangular earth pressure distribution of 6H (kPa) should be adopted for the soil and Class V bedrock profiles, where H is the retained height in metres.
- For progressively propped or anchored shoring systems located in areas that are sensitive to lateral movement (for example, walls which are adjacent to a movement sensitive buried service or adjoining building), a uniform rectangular earth pressure distribution of 8H (kPa) should be adopted for the soil and Class V sandstone bedrock profiles, where H is the retained height in metres.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile above the groundwater table.
- All surcharge loads affecting the walls (eg. adjacent footings, construction loads, live loads etc) should be taken into account in the design using an 'at rest' earth pressure coefficient (K_0) of 0.55 for propped or anchored shoring systems. If inclined retained surfaces are proposed, then they should be treated as a surcharge.
- The contiguous pile retaining walls should be designed as drained and provision made for complete and permanent drainage of the ground behind the walls. The drainage should comprise weepholes made up of, say, 50mm PVC pipes which are grouted into gaps or holes between adjacent piles at say, 1.2m horizontal spacing and located about 0.3m above the proposed basement floor slab level. The embedded end of such weepholes must be covered by a non-woven geotextile filter fabric, such as Bidim A34 or similar, to act as a filter against subsoil erosion.
- Shoring piles embedded into the underlying weathered sandstone bedrock below excavation level may be designed for a maximum allowable lateral toe resistance of 200kPa. The upper 0.5m depth of the socket should not be taken into account to allow for tolerance and disturbance effects during excavation. The passive restraint from the overlying soil profile must be ignored due to strain incompatibility.

4.4.4 Anchors

If soil or rock anchors are to extend outside the site boundaries, then permission must be sought from the respective neighbouring property owner, prior to installation. Our experience has shown that this process can take time and therefore should be completed early on in the construction process.

Soil anchors bonded into loose or denser sands or stiff or harder clays may be designed for an effective angle of internal friction, ϕ', of 30° or undrained shear strength of 50kPa, respectively, with the bond length being fully beyond a line drawn up at 45° from bulk excavation level.

Temporary rock anchors should have a free length of not less than 4m and should be bonded at least 3m into sandstone bedrock, with the bond length being fully beyond a line drawn up at 45° from bulk excavation level. Temporary rock anchors may be designed on the basis of a maximum allowable bond stress of 200kPa in the weathered sandstone bedrock.

All anchors must be proof-loaded to at least 1.3 times the design working load before being locked off at 85% of the working load, all under the direction of an engineer independent of the anchoring contractor. We recommend that only experienced contractors be considered for the anchor installations.

We have assumed that permanent lateral support of the shoring system will be provided by the proposed building. If not, then the anchors will need to be designed for corrosion resistance and for long-term durability.

4.5 Footings

On completion of bulk excavation, sandstone bedrock will be exposed at, or be present a short distance below, bulk excavation level. For uniformity of support we recommend that the proposed building be uniformly founded within the underlying sandstone bedrock.

So as to reduce differential settlements, the portion of building which extends beyond the southern side of the basement should also be supported using CFA piles founded in the underlying sandstone bedrock. The CFA piles must be founded below a 45° line inclined up from bulk excavation level.

Where bedrock is exposed at, or just below bulk excavation level, pad and/or strip footings founded in Class V or better quality sandstone bedrock may be designed for a maximum allowable end bearing pressure of 1,000kPa, subject to inspection of the initial stages of footing excavation by a geotechnical engineer.

If the footings are deepened and are founded in Class IV or better quality sandstone bedrock these may be designed on the basis of a maximum allowable end bearing pressure of 2,000kPa, subject to a geotechnical engineer inspecting each footing excavation.

Shoring and internal CFA piles embedded into Class V or Class IV (or better quality bedrock) may be designed for maximum allowable end bearing pressures of 1,000kPa and 2,000kPa, respectively. Pile embedment deeper than 0.5m into bedrock must only proceed after the piling contractor carries out trails to optimise his drilling techniques over the centre of the site, that the required depth can be achieved without experiencing the ground loss problems described in Section 4.4.1 above.

Pad and/or strip footings surrounding the proposed lift pits must be founded below a 45° line inclined up from the lift pit base.

The provided bearing pressures are based upon serviceability criteria of deflections at the footing base of less than 1% of the minimum footing dimension/pile diameter. The prospective piling contractors should be provided with a full copy of this report so that appropriate drilling rigs and equipment are brought to site.

All footing excavations should be cleaned out and inspected by a geotechnical engineer, immediately prior to pouring. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering.

The initial stages of CFA piling should be compared to the borehole information by a geotechnical engineer to confirm that the founding depths are consistent with the borehole data.

4.6 On-Grade Basement Floor Slab

Slab-on-ground construction is feasible and assuming a 'drained' basement is adopted, we expect a combination of bedrock and natural (mostly clayey) soils to be exposed at bulk excavation level.

Where soil is exposed at bulk excavation level, we recommend that the subgrade be proof rolled with six passes of a small static roller (at least two tonnes deadweight). The final pass should be completed in the presence of a geotechnical engineer. If the subgrade level needs to be 'topped up' following compaction or if there are any 'soft' or heaving areas detected, then we recommend that ground levels be raised using engineered fill, as outlined below.

The excavated natural soils above the groundwater table are suitable for reuse as engineered fill provided they are free of organic matter and do not contain any particle sizes greater than 50mm. The engineered fill must be compacted in maximum 150mm thick loose layers to a density ratio between 98% and 102% of Standard Maximum Dry Density (SMDD) and at a moisture content within 2% of the Standard Optimum Moisture Content (SOMC).

Density tests should be carried out to confirm the above specification has been achieved. The frequency of density testing should be at least one test per layer per 1,000m^2, or three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798 ("Guidelines on Earthworks for Commercial and Residential Developments"). The geotechnical testing authority (GTA) should be directly engaged by the client and not by the earthworks contractor.

We recommend that underfloor drainage be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system. Discharge into the stormwater system may require Council approval.

Joints in the concrete basement level on-grade floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.7 Hydrogeological Issues

Based on the investigation results, we expect that intermittent groundwater seepage following periods of rainfall will flow over the bedrock surface and through joints and bedding planes within the bedrock.

The proposed excavation will intersect the groundwater seepage paths, though provision for drained retraining walls will permit groundwater through-flow and will reduce the possibility of groundwater levels building up behind the basement retaining walls.

In view of the above, the proposed development should not adversely affect the existing transient groundwater flows to the extent that there will be any significant impact on surrounding buildings and structures, provided the recommendations presented in this report are adopted.

Furthermore, dewatering during construction or tanking measures over the long term are considered unwarranted.

4.8 Additional Geotechnical Input

We summarise below the previously recommended additional work that needs to be carried out:

1. Dilapidation surveys.
2. Vibration monitoring.
3. Witness the proof loading of temporary anchors, if installed.
4. Proof rolling inspections, if appropriate.
5. Density testing of engineered fill, if appropriate.
6. Witnessing of CFA pile installations.
7. Footing inspections.

5 GENERAL COMMENTS

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
301
1/1

Client: MOORGATE FINANCE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 7 LAWRENCE STREET & 18 MARMORA STREET, FRESHWATER, NSW

Job No. 27296ZH
Date: 14-3-14
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 18.7m
Datum: AHD
Logged/Checked by: A.P.C./A.J.H

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0		TOP SOIL: Silty sand, fine to medium grained, dark brown, with roots and root fibres.	M			GRASS COVER
				CL	SANDY CLAY: medium plasticity, yellow brown, fine to medium grained sand, with root fibres, trace of fine to medium grained sub rounded ironstone gravel and silt fines.	MC>PL	VSt		
	DB	N = 12 5,6,6						350 300 320	
			1		as above, but yellow brown and red brown, with fine to medium grained sub rounded ironstone gravel.				
	DB	N = 13 5,6,7							
			2	CL-CH	SANDY CLAY: medium to high plasticity, light grey mottled red brown, fine to medium grained sand, trace of root fibres.		VSt-H		
	DB	N > 6 10,6/50mm REFUSAL	3					450 350 380	
AFTER 1.5 HRS				-	SANDSTONE: fine to medium grained, yellow and orange brown.	DW	L		MODERATE RESISTANCE
	DB		4		SANDSTONE: fine to medium grained, light grey and orange brown.		L-M		MODERATE TO HIGH RESISTANCE
					END OF BOREHOLE AT 4.5m				
			5						
			6						
			7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
302
1/1

Client: MOORGATE FINANCE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 7 LAWRENCE STREET & 18 MARMORA STREET, FRESHWATER, NSW

Job No. 27296ZH
Date: 14-3-14
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 15.5m
Datum: AHD
Logged/Checked by: A.P.C./A.J.H

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION						0			FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	M			GRASS COVER
					N = 2 2,2,0	1			FILL: Silty sand, fine to medium grained, dark brown and yellow brown, with roots and root fibres.				APPEARS POORLY COMPACTED
					N = 5 1,2,3	2		CL	SANDY CLAY: low to medium plasticity, light grey mottled orange brown, fine to medium grained sand, with fine to medium grained sub rounded to rounded ironstone gravel, trace of root fibres.	MC>PL	St	120 110 130	
					N = 27 9,12,15	3			SANDY CLAY: medium plasticity, orange brown mottled red brown.		VSt	350 400 380	
						4		-	SANDSTONE: fine to coarse grained, orange and yellow brown.	DW	L		LOW TO MODERATE RESISTANCE
											L-M		MODERATE RESISTANCE
						5 6 7			END OF BOREHOLE AT 4.5m				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
301A
1/1

Client: MOORGATE FINANCE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 7 LAWRENCE STREET & 18 MARMORA STREET, FRESHWATER, NSW

Job No. 27296ZH
Date: 14-3-14
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 18.7m
Datum: AHD
Logged/Checked by: A.P.C./A.J.H

Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			TOP SOIL: Silty sand, fine to medium grained, dark brown, with roots and root fibres.	M			GRASS COVER
					CL	SANDY CLAY: medium plasticity, yellow brown, fine to medium grained sand, with root fibres, trace of fine to medium grained sub rounded ironstone gravel and silt fines.	MC>PL	(VSt)		
			1			as above, but yellow brown and red brown, with fine to medium grained sub rounded ironstone gravel.				
			2		CL-CH	SANDY CLAY: medium to high plasticity, light grey mottled red brown, fine to medium grained sand, trace of root fibres.		(VSt - H)		
			3			END OF BOREHOLE AT 2.92m				MONITORING WELL INSTALLED TO 2.92m DEPTH, MACHINE SLOTTED PVC FROM 0.92m TO 2.92m, CASING FROM 0.92m TO SURFACE, 2mm SAND FILTER PACK FROM 0.65m TO 2.92m, BENTONITE SEAL FROM 0.1m TO 0.65m, COMPLETED WITH A GATIC COVER AND LOCKABLE CAP
			4							
			5							
			6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

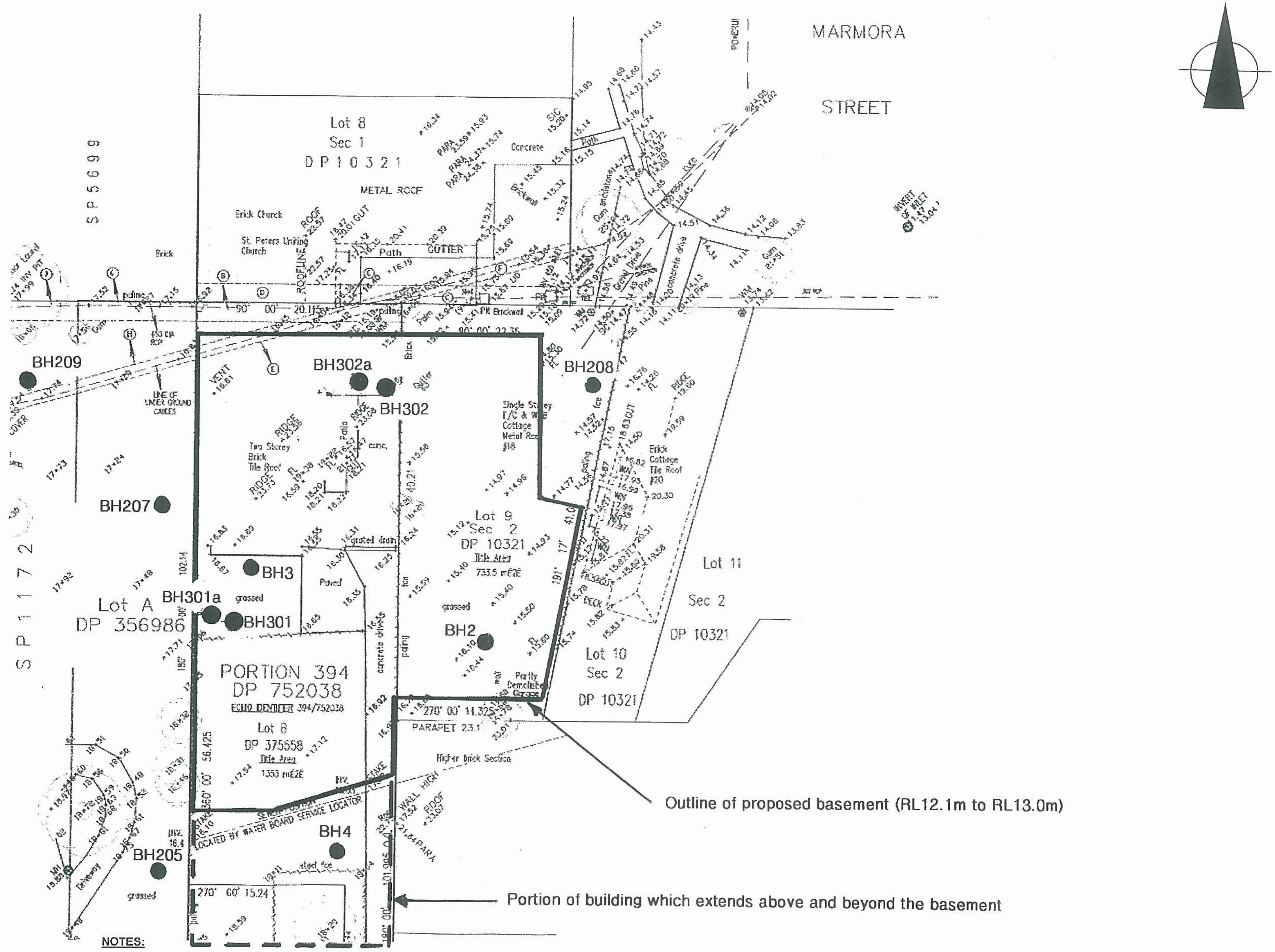
BOREHOLE LOG

Borehole No.
302A
1/1

Client: MOORGATE FINANCE PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 7 LAWRENCE STREET & 18 MARMORA STREET, FRESHWATER, NSW

Job No. 27296ZH
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 15.5m
Date: 14-3-14
Datum: AHD
Logged/Checked by: A.P.C./A.J.H

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Silty sand, fine to medium grained, dark brown, with root fibres.	M			GRASS COVER
									FILL: Silty sand, fine to medium grained, dark brown and yellow brown, with roots and root fibres.				APPEARS POORLY COMPACTED
						1		CL	SANDY CLAY: low to medium plasticity, light grey mottled orange brown, fine to medium grained sand, with fine to medium grained sub rounded to rounded ironstone gravel, trace of root fibres.	MC>PL	(St)		
						2							
						3			SANDY CLAY: medium plasticity, orange brown mottled red brown.		(VSt)		
								-	SANDSTONE: fine to coarse grained, orange brown.	DW	L		LOW 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 3.21m				MONITORING WELL INSTALLED TO 3.21m DEPTH, MACHINE SLOTTED PVC FROM 1.2m TO 3.21m, CASING FROM 1.2m TO SURFACE, 2mm SAND FILTER PACK FROM 0.75m TO 3.21m, BENTONITE SEAL FROM 0.15m TO 0.75m, COMPLETED WITH A GATIC COVER AND LOCKABLE CAP
						4							
						5							
						6							
						7							



NOTES:

1. TO BE READ IN CONJUCTION WITH THE TEXT OF THE REPORT.
2. BH2, BH3 AND BH4 ARE FROM A PREVIOUS GEOTECHNICAL INVESTIGATION COMPLETED BY GEOENVIRO CONSULTANCY PTY LTD IN 2007.
3. BH205, BH207, BH208 AND BH209 ARE FROM A PREVIOUS GEOTECHNICAL INVESTIGATION COMPLETED BY JEFFERY AND KATAUSKAS PTY LTD IN 2010.
4. BH301, BH301a, BH302 AND BH302a ARE FROM THE CURRENT INVESTIGATION.
5. A 50mm DIAM' 'R STANDPIPE WAS INSTALLED INTO BH301a AND BH302a

BOREHOLE LOCATION PLAN

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No 27296ZH Figure No. 1

APPENDIX A

GeoEnviro Consultancy Pty Ltd

Borehole Report

Borehole no: 2

Client:	Blackmore Design Group	Job no:	JG07011A
Project:	Proposed Mixed-Use Building	Date:	27/7/07
Location:	18 Mamora Street and Nos 5-7 Lawrence Street, Harbord	Logged by:	JC
		Checked By:	SL

Drill Model and Mounting: P160 — Slope: 90 degrees — R.L. Surface:

Hole Diameter: 100 mm — Bearing: - — Datum: AHD

Method	Support	Water	Notes: Samples, Tests, etc	Depth(m)	Classification Symbol	Unified Soil Classification	Material Description Soil Type, Plasticity or Particle Characteristic, colour, secondary and minor component	Moisture Content	Consistency/Density Index	Dynamic Cone Penetrometer	Structure and Additional Observations
V - B I T	N I L			0.0			Topsoil: Silty Sand, fine to medium grained dark grey				
						SM	Silty Sand: fine to medium grained, grey yellow brown				
			N=5 1,2,3	1.0		SC/CI	Sandy Clay/Clayey Sand: fine to medium grained, brown with some gravel	MC =PL	St-Vst	180 200	
				2.0			As above but with some ironstone gravel/ cobbles	M-W			
		▼						W			Groundwater at 2.8m
				3.0							Proposed Basement FFL RL13.0m
						SC	Clayey Sand/Sandy Clay: fine to medium grained, yellow brown with some ironstone gravel	MC >PL	MD		
			N>13 13/60mm Bounce	4.0							
				5.0			Sandstone: yellow brown				
							Refer to Cored Borehole at below 5.2m				
				6.0							
				7.0							
				8.0							

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Form no. R007/Ver02/06/99

GeoEnviro Consultancy Pty Ltd

Cored Borehole Report

Borehole no: 2

Client: Blackmored Desin Group	Job no: JG07011A
Project: Proposed Mixed Use Building	Date: 27/7/07
Location: 18 Marmora Street and Nos 5-7 Lawrence Street, Harbord	Logged by: JC
	Checked By: SL

Drill Model and Mounting: P160	Slope: -	R.L Surface: -
Hole Diameter: 100 mm	Bearing: -	Datum: -

Method	Support	Barrel Lift	Water Loss/ Level	Depth(m)	Graphic Log	Core Description Rock type, grain characteristics, colour, structure, minor components.	Weathering	Strength	Point Load Index Strength Is(50) VL M VH EL L H EH	Defect Spacing (mm) 300 50 10 500 100 30	Defect Details: Description type, inclination, thickness, planarity, roughness, coating
				4–5		Start Coring at 5.2m					
NMLC	NIL	HALF RETURN		5.2–7.88		Sandstone: fine to medium grained grey brown	EW SW	VL H	x x x x x		EW Seam: 2mm thick 2 EW Seam: 1mm thick EW Seam: 2mm thick
				8–12		End of Cored Borehole at 7.88m					

GeoEnviro Consultancy Pty Ltd

Borehole Report

Borehole no: 3

Client:	Blackmore Design Group	Job no:	JG07011A
Project:	Proposed Mixed-Use Building	Date:	27/7/07
Location:	18 Mamora Street and Nos 5-7 Lawrence Street, Harbord	Logged by:	JC
		Checked By:	SL

Drill Model and Mounting: P160	Slope: 90 degrees	R.L. Surface:
Hole Diameter: 100 mm	Bearing: -	Datum: AHD

Method	Support	Water	Notes: Samples, Tests, etc	Depth(m)	Classification Symbol	Unified Soil Classification	Material Description Soil Type, Plasticity or Particle Characteristic, colour, secondary and minor component	Moisture Content	Consistency/Density Index	Dynamic Cone Penetrometer	Structure and Additional Observations
V - BIT	NIL			0.0			Topsoil: Silty Sand, dark grey				
						SC	Clayey Sand: fine to medium grained, yellow brown	M			
				1.0		CI	Sandy Clay: medium plasticity, brown with some ironstone gravel	MC =PL	(St)		
				2.0							
		▶	N>8 4,8/50mm	3.0		CI	Sandy Silty Clay: medium plasticity, brown a trace of ironstone gravel	MC >= PL	Vst		Groundwater Seepage
TC							Sandstone: fine to medium grained, yellow brown				
							End of Borehole at 3.4m				Proposed Basement FFL RL 13.0m
				4.0							
				5.0							
				6.0							
				7.0							
				8.0							

GeoEnviro Consultancy Pty Ltd

Borehole Report

Borehole no: 4

Client:	Blackmore Design Group	Job no:	JG07011A
Project:	Proposed Mixed-Use Building	Date:	27/7/07
Location:	18 Mamora Street and Nos 5-7 Lawrence Street, Harbord	Logged by:	JC
		Checked By:	SL

Drill Model and Mounting: P160	Slope: 90 degrees	R.L. Surface:
Hole Diameter: 100 mm	Bearing: -	Datum: AHD

Method	Support	Water	Notes: Samples, Tests, etc	Depth (m)	Classification Symbol	Unified Soil Classification	Material Description Soil Type, Plasticity or Particle Characteristic, colour, secondary and minor component	Moisture Content	Consistency/Density Index	Dynamic Cone Penetrometer	Structure and Additional Observations
TC - BIT	NIL	DRY		0.0			Topsoil: Silty Sand, fine to medium grained dark grey	M			
						CL-CI	Clayey Sand: fine to medium grained, grey brown	M			
				1.0		CI	Sandy Clay: low to medium plasticity, yellow brown As above but with some ironstone gravel	MC =PL	(St)		
				2.0							
				3.0		CI	Shaley Clay: medium plasticity, grey with some ironstone gravel	MC <= PL	Vst -H		
				4.0			Sandstone: fine to medium grained, low strength, yellow brown				
				5.0			End of Borehole at 4.7m				Proposed Basement FFL RL 13.0m
				6.0							
				7.0							
				8.0							

APPENDIX B

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. **205**
1/2

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY
Date: 9-2-10
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 18.7m
Datum: AHD
Logged/Checked by: M.T./WT

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Gravel, medium to coarse grained, grey.	M			
	N = 4 1,2,2			FILL: Silty sand, fine to medium grained, brown, with a trace of root fibres.	M			
ON COMPLETION OF CORING		1	SM	SILTY SAND: fine to medium grained, light brown, with a trace of clay fines.		VL		
	N = 11 3,4,7	2	SC	CLAYEY SAND: fine to medium grained, orange red brown, with a trace of fine to medium grained ironstone gravel.		MD		RESIDUAL
	N = 30 9,13, 17/50mm	3	CL	SILTY CLAY: medium plasticity, grey and red brown, with fine to medium grained ironstone gravel.	MC<PL	H	>600 >600 >600	
	N = 37 4,18,19	4, 5		as above, but with fine grained sand.			>600 >600 >600	Proposed Basement FFL RL 13.0m
		6	-	SANDSTONE: fine to medium grained, light brown.	DW	L		LOW 'TC' BIT RESISTANCE
		7		REFER TO CORED BOREHOLE LOG				

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

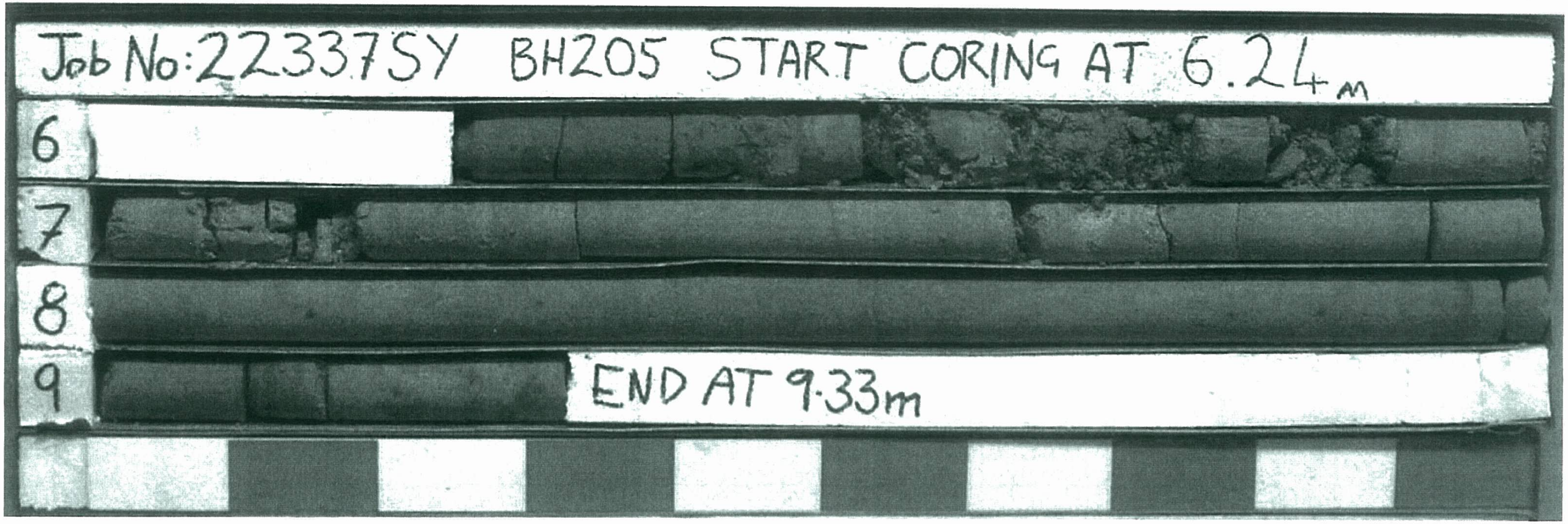
Borehole No.
205
2/2

Client:	FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY	**Core Size:** NMLC	**R.L. Surface:** ≈ 18.7m
Date: 9-2-10	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK350	**Bearing:** -	**Logged/Checked by:** M.T./[illegible]

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		5								
		6		START CORING AT 6.24m						
90% RETURN				SANDSTONE: fine to medium grained, orange brown.	DW	M	×		XWS, 0°, 350mm.t	Class V
				as above, but grey.	XW	EL			XWS, 0°, 90mm.t	
		7			DW	L-M	×		CS, 0°, 20mm.t XWS, 0°, 100mm.t	
					FR	H	×		XWS, 5°, 30mm.t	class I
		8					×			
				as above, but with brown staining.	SW		×			
		9					×			
							×			
				END OF BOREHOLE AT 9.33m						
		10								
		11								

JEFFERY & KATAUSKAS PTY LTD



Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **207**

1/2

BOREHOLE LOG

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD

Project: PROPOSED MIXED USE DEVELOPMENT

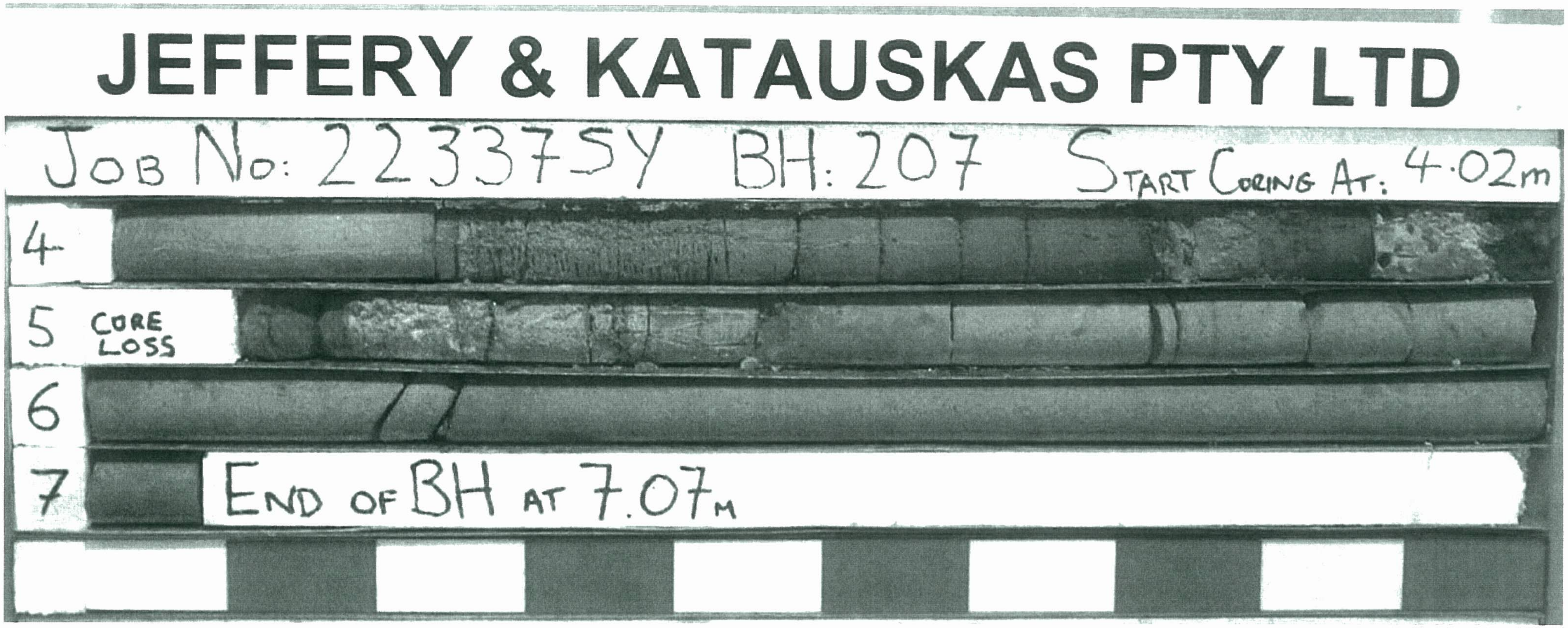
Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY | **Method:** SPIRAL AUGER JK350 | **R.L. Surface:** ≈ 17.6m

Date: 9-2-10 | **Datum:** AHD

Logged/Checked by: M.T./WT

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES		0			TOPSOIL/FILL: Clayey silty sand, fine to medium grained, brown, with a trace of root fibres.	M			GRASS COVER
ON COMPLETION & AFTER 2 HRS	DS	N = 4 1,2,2			SM	SILTY SAND: fine to medium grained, light brown.	M	L	-	RESIDUAL
DURING AUGERING	DS	N = 9 3,5,4	1		SC	CLAYEY SAND: fine to medium grained, orange and red brown.	W			
	DS	N = 22 6,9,13	2–3		CL	SILTY CLAY: medium plasticity, light grey, with a trace of fine to medium grained sand.	MC>PL	H	540 480 510	
			4		-	SANDSTONE: fine to medium grained, orange brown.	DW	L-M	-	LOW TO MODERATE 'TC' BIT RESISTANCE
						REFER TO CORED BOREHOLE LOG				
			5							
			6							
			7							


JEFFERY & KATAUSKAS PTY LTD
JOB No: 22337SY BH: 207 START CORING AT: 4.02m
4
5 CORE LOSS
6
7 END OF BH AT 7.07m

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **207** 2/2

Client:	FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY	**Core Size:** NMLC	**R.L. Surface:** ≈ 17.6m
Date: 9-2-10	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK350	**Bearing:** -	**Logged/Checked by:** M.T./ [illegible]

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		3								
				START CORING AT 4.02m						
		4		SANDSTONE: fine to medium grained, orange brown.	DW	L			- XWS, 0°, 10mm.t	Class V
						VL			Proposed Basement FFL RL 13.0m	
						L-M				
						VL			- XWS, 0°, 10mm.t - XWS, 0°, 5mm.t - CS, 0°, 30mm.t - XWS, 0°, 70mm.t	
		5		CORE LOSS 0.10m						
90% RET-URN				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-3°.	XW	EL				
					DW	VL			- CS, 0°, 15mm.t	
					SW	L-M			- CS, 5°, 10mm.t	
		6								Class IV
		7								
				END OF BOREHOLE AT 7.08m						
		8								
		9								

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No.
208
1/2

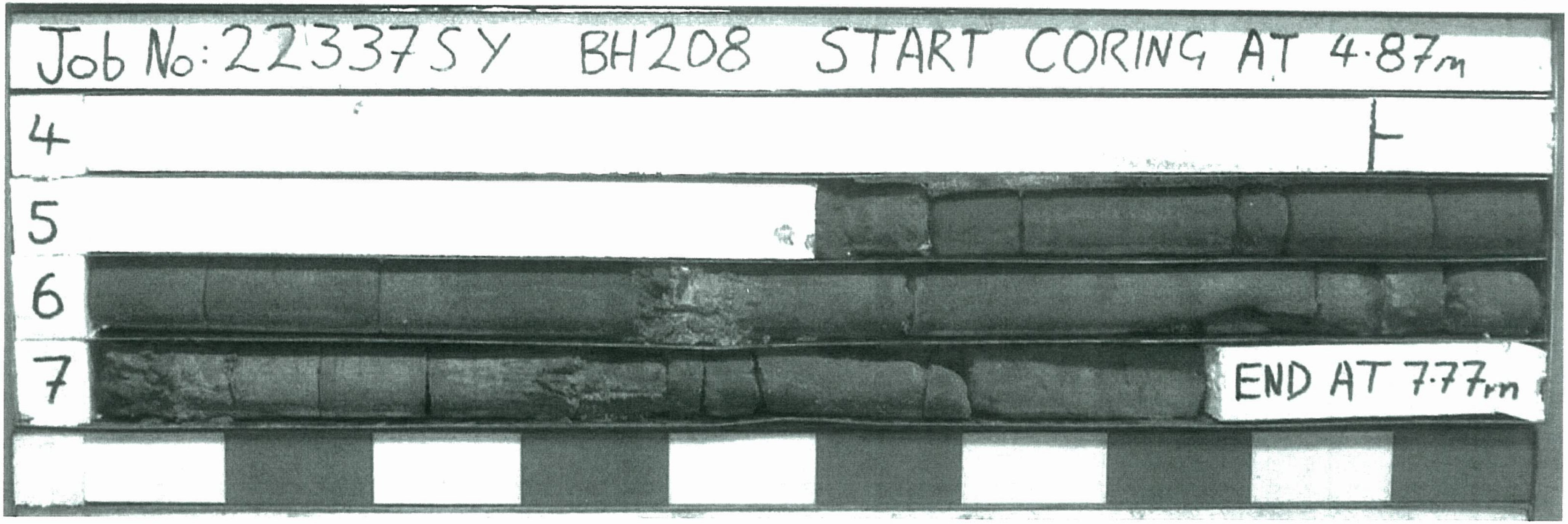
BOREHOLE LOG

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY **Method:** SPIRAL AUGER JK350 **R.L. Surface:** ≈ 14.5m
Date: 10-2-10 **Datum:** AHD
Logged/Checked by: M.P./ [illegible]

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty sand, fine to medium grained, dark brown and brown, with a trace of root fibres.	M			
		N = 2 1,1,1	1		SM	SILTY SAND: fine to medium grained, light brown and orange brown, with clay fines.	M	VL		RESIDUAL
		N = 10 4,4,6	2			as above, but orange brown and light grey.		MD		Proposed Basement FFL RL12.1m
DURING AUGER-ING							W			
			3 4		-	SANDSTONE: fine to medium grained, red brown, with VL strength bands.	XW-DW	EL-VL	-	EXTREMELY LOW TO VERY LOW 'TC' BIT RESISTANCE
			5 6 7			END OF BOREHOLE AT 4.87m				

JEFFERY & KATAUSKAS PTY LTD



Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No. **208** 2/2

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY | **Core Size:** NMLC | **R.L. Surface:** ≈ 14.5m
Date: 10-2-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK350 | **Bearing:** - | **Logged/Checked by:** M.P./WT

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		4		START CORING AT 4.87m						
		5		CORE LOSS 0.62m						Class V
FULL RET-URN		6		SANDSTONE: fine to medium grained, light brown and red brown.	DW	L	× ×		XWS, 0°, 25mm.t	Class IV
				as above, but light grey and red brown.	SW	L-M	×		XWS, 0°, 20mm.t CS, 0°, 10mm.t XWS, 0°, 3mm.t 6.71-6.87m J, 70-90°, Un, S	
		7		as above, but medium to coarse grained.			×		CS, 0°, 50mm.t	
				SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-5°.	DW SW	VL-L L-M	× ×		XWS, 0°, 20mm.t J, 70-80°, Un, R	
		8		END OF BOREHOLE AT 7.77m						
		9								
		10								

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
209
1/2

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD

Project: PROPOSED MIXED USE DEVELOPMENT

Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY **Method:** SPIRAL AUGER JK350 **R.L. Surface:** ≈ 17.9m

Date: 9-2-10 **Datum:** AHD

Logged/Checked by: M.T./WT

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING		0		FILL: Clayey silty sand, fine to medium grained, dark brown, with a trace of root fibres.				GRASS COVER
	N = 4 1,2,2		SM	SILTY SAND: fine to medium grained, light brown.	M	L	-	
AFTER 3 WEEKS ON 2-3-10	N = 11 2,5,6	1 – 2	SC	CLAYEY SAND: fine to medium grained, light grey mottled orange and red brown.	M	MD	-	RESIDUAL
	N = 22 4,8,14	3	CL	SILTY CLAY: medium plasticity, light grey, with bands of ironstone gravel and sand.	MC≈PL	VSt-H	350 320 510	
			-	SANDSTONE: fine to medium grained, orange and red brown.	DW	L		LOW 'TC' BIT RESISTANCE
		4 – 7		REFER TO CORED BOREHOLE LOG				50mm DIAMETER CLASS 18 PVC STANDPIPE INSTALLED TO 5.5m DEPTH. MACHINE SLOTTED FROM 2.5m TO 5.5m, BACKFILLED WITH 2mm SAND AND BENTONITE COLLAR. FINISHED WITH GATIC COVER

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CORED BOREHOLE LOG

Borehole No.
209

Client: FRESHWATER VILLAGE DEVELOPMENTS PTY LTD
Project: PROPOSED MIXED USE DEVELOPMENT
Location: CNR. LAWRENCE STREET AND ALBERT STREET, FRESHWATER, NSW

Job No. 22337SY | **Core Size:** NMLC | **R.L. Surface:** ≈ 17.9m
Date: 10-2-10 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK350 | **Bearing:** - | **Logged/Checked by:** M.T./ wT

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		3								
		4		START CORING AT 4.02m						
FULL RET-URN				SANDSTONE: fine to medium grained, light grey mottled light brown, with dark grey laminae at 0-5°.	DW	L				Class IV
		5							- XWS, 0°, 10mm.t - CS, 0°, 15mm.t - XWS, 0°, 5mm.t - XWS, 0°, 10mm.t - XWS, 0°, 10mm.t - XWS, 0°, 130mm.t - XWS, 0°, 10mm.t - XWS, 0°, 5mm.t	Class V
					SW	M			Proposed Basement FFL RL 12.1m	
		6		SANDSTONE: fine to medium grained, light grey.	FR				- CS, 5°, 5mm.t	Class III
		7		END OF BOREHOLE AT 7.12m						
		8								
		9								

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
207	4.02-4.04	0.2	4
	4.44-4.48	0.1	2
	4.59-4.63	0.3	6
	5.22-5.26	0.04	<1
	5.42-5.46	0.04	<1
	5.90-5.93	0.2	4
	6.37-6.40	0.6	12
	7.04-7.07	0.4	8
208	5.60-5.63	0.1	2
	6.17-6.20	0.1	2
	6.55-6.59	0.2	4
	7.16-7.19	0.3	6
	7.35-7.38	0.1	2
	7.46-7.50	0.3	6
209	4.17-4.21	0.3	6
	4.78-4.81	0.3	6
	5.19-5.21	0.2	4
	5.86-5.89	0.8	16
	6.30-6.33	0.4	8
	6.95-6.99	0.9	18

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\,(50)}$

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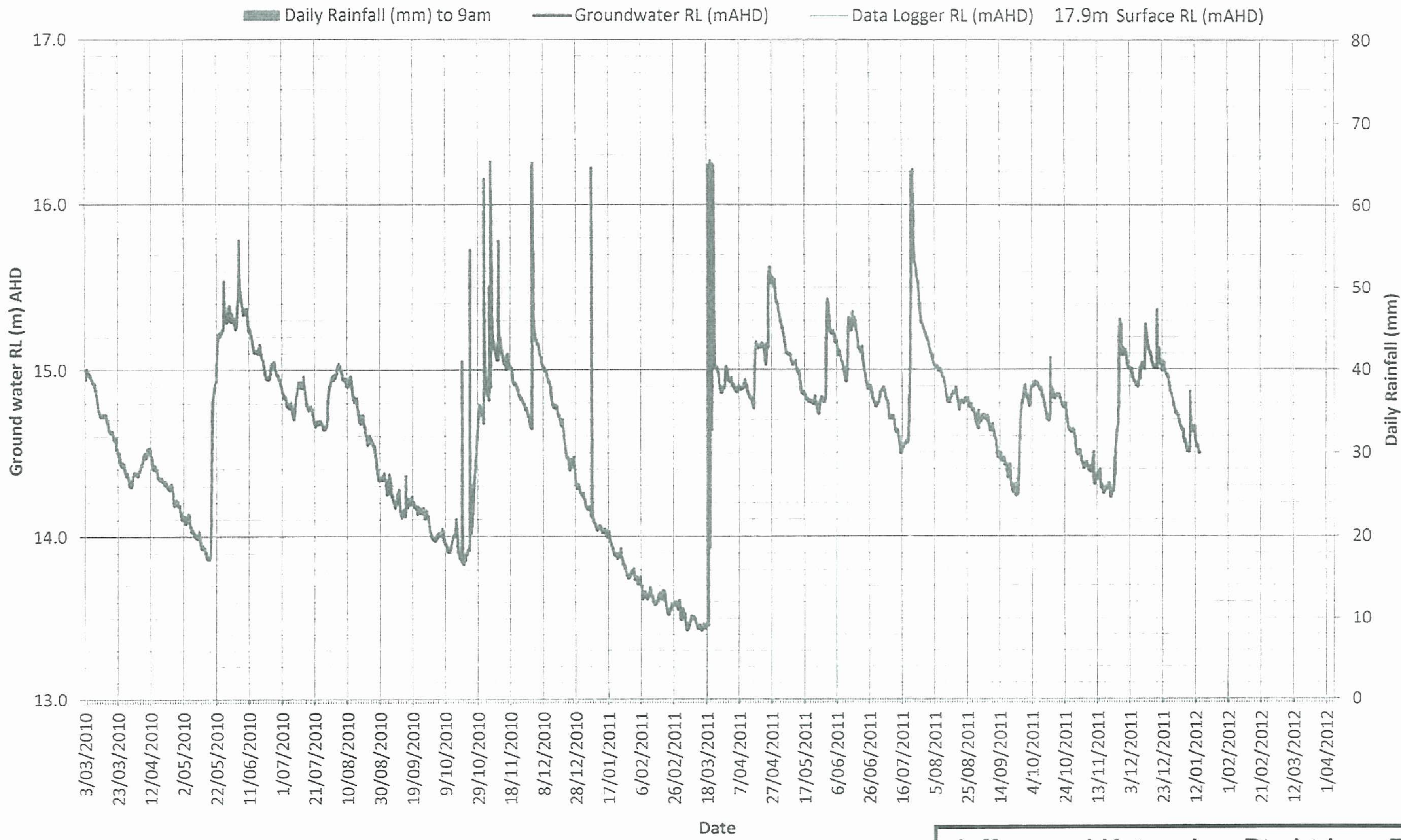
TABLE A
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
203	12.75-12.78	1.9	38
	13.16-13.21	1.7	34
	13.70-13.73	1.9	38
	14.19-14.23	1.4	28
204	6.35-6.37	0.2	4
	6.78-6.80	0.6	12
	7.29-7.32	0.3	6
	7.77-7.79	0.2	4
	8.29-8.32	0.3	6
	8.62-8.65	0.9	18
	9.30-9.34	1.1	22
	9.84-9.88	2.5	50
	10.32-10.35	1.9	38
205	6.32-6.34	0.8	16
	6.93-6.96	0.7	14
	7.38-7.42	1.7	34
	7.84-7.87	0.8	16
	8.28-8.31	1.1	22
	8.74-8.79	1.4	28
	9.29-9.33	1.4	28
206	6.63-6.66	1.0	20
	7.14-7.17	1.1	22
	7.79-7.83	1.0	20
	8.21-8.25	1.0	20
	8.74-8.77	0.8	16
	9.32-9.35	0.9	18

NOTES:SEE PAGE 3 OF 3

File Name: Data Logger Chart BH209
Date Printed: 8/10/2012

Ground Water Level and Daily Rainfall -v- Time Plot
209



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Report No. 22337S

Figure No 4

Rainfall data from Cromer Golf Club , Station No. 066044

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REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

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

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

The information provided on the charts comprise:

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

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if water observations are to be made.

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

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soil for Engineering Purposes'*. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

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

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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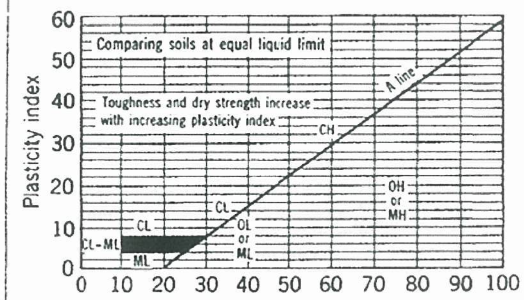
Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)				Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils: More than half of material is *larger* than 75 µm sieve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

		Identification Procedures on Fraction Smaller than 380 µm Sieve Size			Group Symbols	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)				
Fine-grained soils: More than half of material is *smaller* than 75 µm sieve size (The 75 µm sieve size is about the smallest particle visible to naked eye)	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses	Use grain size curve in identifying the fractions as given under field identification
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity		
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils		



Plasticity chart
for laboratory classification of fine grained soils

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

LOG SYMBOLS

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

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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