REPORT

TO
MOUNT PRITCHARD & DISTRICT COMMUNITY CLUB

ON
GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATION

FOR
PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB

AT
80 EVANS STREET, FRESHWATER, NSW

17 June 2015
Ref: 25077ZH3rpt Rev2

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EXECUTIVE SUMMARY

A geotechnical investigation for the proposed Harbord Diggers Club Redevelopment at 80 Evans Street, Freshwater was carried out.

The proposed redevelopment envisages the demolition of the existing club and parking and the construction of several medium rise buildings over two common basement parking levels. The basements will extend to the site boundaries and will require maximum excavation depths of about 16m. The redevelopment will be staged to allow portions of the existing club to remain operational.

The investigation has revealed a generalised subsurface profile comprising shallow fill over residual soils then weathered shale and sandstone bedrock from relatively shallow depth. Localised and intermittent groundwater seepage was encountered.

Based on the investigation results, the following general comments and recommendations have been presented:

1. Issues with the stability of the site are not anticipated.
2. Significant volumes of excavation will be required with such excavations extending through the soil profile and well into the weathered bedrock of variable and often high strength.
3. The proposed excavation will need to be supported both during the construction period and over the long term. Various options of achieving such support have been provided, including full depth anchored soldier pile walls, and a combination of soldier piles and rock bolting. Parameters for the design of the retention systems have also been provided.
4. Options for temporary battering of the excavation between the stages of the redevelopment have been provided.
5. The proposed buildings should be supported using conventional pad or strip footings founded in Class III shale (or better) which is expected to be exposed over bulk excavation level. Allowable bearing pressures of 3,500kPa are recommended.
6. The proposed basement should be designed with behind wall and underfloor drainage. Pump-out facilities will be required to cater for a predicted inflow of about 5m/day or less than 2ML/year.
7. A detailed geotechnical associated inspection/monitoring has been presented, which addresses ground vibrations, shoring deflection, stability, footings and seepage volumes in particular.

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1 INTRODUCTION

This report presents the results of a geotechnical and hydrogeological investigation for the proposed redevelopment of Harbord Diggers Club ('the Club'), 80 Evans Street, Freshwater, NSW.

The investigation was carried out in two stages. The first stage was completed as per the *'Letter of Engagement'* with the Club, dated 3 August 2011. The initial Phase 2 scope of geotechnical consultancy services was outlined in our fee proposal, Ref: P34183WH, dated 8 July 2011, which forms part of the *'Letter of Engagement'*. The scope of work was subsequently revised, as outlined in our email (Ref 25077ZH2 email1) dated 6 September 2012.

The second stage was completed under a Minor Consultancy Agreement in accordance with our proposal (Ref P25077ZH Harbord Rev1) dated 3 February 2015.

Following the completion of the first stage of the investigation and preparation of our report (Ref 25077ZH3rpt) dated 30 July 2014, the proposed development was revised.

This current report uses the information from the first and second stages of the investigation and is a stand-alone report for the proposed club redevelopment as discussed below.

To assist with the geotechnical investigation, we have been supplied with the following information:

1. Survey plan of the site and its immediate surrounds, prepared by Lean and Hayward Pty Ltd (Drawing No 74722.07.D01, dated 26 August 2010).
2. Architectural drawings prepared by Architectus Sydney Pty Ltd and CHROFI (Drawing Nos A0200^{D}, A0500^{D}, A0501^{D}, A0502^{E}, A0600^{D} and A5074^{A}).
3. Structural drawings prepared by enstruct (Drawing Nos ST-0-001-01^{3}, -02^{3}, 002-50^{1}, -51^{2}, -52^{1}, 61^{1} to 66^{1}, -71^{1} and -72$^{1.}$
4. Civil drawings prepared by enstruct (Drawing Nos CIV-000^{2}, 300^{2}, 400^{2}, 401^{2}, 403^{2}, 0450^{2}, 0600^{1}, 064^{1}, 0900^{1} and 0901^{1}).

For the purpose of this report, we have taken Evans Street to bound the site to the south, with both 'Site North' and 'Survey North,' shown on Figure 1. Figure 1 is based on the supplied survey plan.

Based on the supplied information, we understand that the proposed redevelopment will include demolishing and adaptively re-using existing structures on site, followed by construction of a four storey building underlain by a two level basement. To achieve the lowest basement (Level 2) level at RL9.5m, excavation to depths between about 9m and 16m below existing grade will be required. The redevelopment will be undertaken in two phases. Portions of the existing club will remain operational during the first phase, which will include the construction of a new club. Once the new club is completed, Phase 2 will commence and will include demolition and bulk excavation of the existing club building. The existing electrical substation on the Evans Street frontage will be retained throughout.

We have not been provided with the structural loads for the proposed building, however, we expect the loads would be in the moderate to high range.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions at the site, and based on the results obtained, to present our comments and recommendations on site stability, excavation conditions and support, retaining walls, footings, soil aggression, basement floor slabs, hydrogeology and external pavements.

Environmental Investigation Services (EIS) [the environmental consulting division of the JK Group] have completed two Environmental Site Assessments at the site, and the results were presented in their reports (Ref E24001Krpt dated May 2010) and (Ref E24001K2rpt dated September 2012). EIS prepared a letter (Ref 24001Klet-draft dated 9 July 2014) which indicated that these two previous EIS reports were still valid. An additional Environmental Site Assessment report (Ref E24001KBrpt3) dated March 2015 was then completed. This geotechnical investigation report must be read in conjunction with the three previous EIS reports.

2 INVESTIGATION PROCEDURE

Prior to each stage of the fieldwork, a ‘Dial Before You Dig’ search was undertaken and the borehole locations were electromagnetically scanned by a specialist sub-contractor for buried services.

The fieldwork for the first stage of the investigation was carried out on 11, 12 and 14 September 2012 and comprised the drilling of five boreholes (BH101 to BH105). The fieldwork for the second stage of the investigation was carried out on 19, 20, 23, 24 and 25 February and 10 and 11 March 2015 and comprised the drilling of six boreholes (BH201 to BH206) and the completion of eight Dynamic Cone Penetration (DCP) tests (DCP207 to DCP214). The borehole and DCP test locations, as indicated on attached Figure 1, were set out by taped measurements from existing surface features and apparent site boundaries. Figure 1 is based on the supplied survey plan. The surface RLs indicated on the attached borehole logs were interpolated between spot level heights shown on the supplied survey plan and are therefore only approximate. The survey datum is the Australian Height Datum (AHD).

The boreholes were auger drilled to depths between 0.22m (BH205) and 5.59m (BH104) using our track and truck mounted JK250, JK300 and JK500 rigs and our portable Melvelle drill rig. Each borehole was extended into the underlying bedrock using rotary diamond coring techniques with an NMLC triple tube core barrel and water flush to final depths between 9.79m (BH104) and 19.31m (BH202). We note that BH203 was abandoned at 7.39m depth due to problems with drilling techniques and BH203A was subsequently drilled to its full depth of 14.98m.

The DCP tests were extended to refusal depths between 0.22m and 1.75m.

The nature and composition of the subsurface soil and rock horizons were assessed by logging the materials recovered during drilling. The relative compaction, strength and density of the subsoil profile were assessed from the DCP tests and the Standard Penetration Test (SPT) ‘N’ values, augmented by hand penetrometer readings on clayey samples recovered in the SPT split spoon sampler. The DCP refusal depths can also provide an indicative depth to bedrock, although refusal can also occur on buried obstructions, other hard layers, etc and not necessarily on bedrock. The strength of the upper weathered bedrock profile was assessed by observation of auger penetration resistance when using a tungsten carbide (TC) bit, together with examination of recovered rock cuttings and correlation with subsequent moisture content tests. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory Point Load Strength Index ($I_{S(50)}$) tests.

Groundwater observations were made in each borehole during the fieldwork. Slotted PVC standpipes were installed in BH201 and BH204 for longer term groundwater monitoring (refer to the relevant borehole logs for the standpipe construction). On 10 March 2015, the standpipes were purged and a data logger installed to record the rate of recovery of the groundwater level (pump-out tests). Longer term groundwater monitoring was not carried out.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineers were present on a full-time basis during both stages of the fieldwork to set out the borehole locations, direct the electromagnetic scanning, nominate the testing and sampling, prepare the attached borehole logs and carry out the pump-out tests. The Report Explanation Notes define the logging terms and symbols used.

Selected soil and rock chip samples were returned to NATA registered laboratories (Soil Test Services Pty Ltd [STS] and Envirolab Services Pty Ltd) for moisture content, soil pH, chloride and sulphate, Standard compaction and four day soaked CBR testing. The test results are summarised in Table A, B and D. The Envirolab Services Pty Ltd *'Certificate of Analysis'* is attached to this report.

The recovered rock cores were photographed and returned to STS for Point Load Strength Index testing. The photographs are enclosed facing the relevant cored borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and are also summarised in the attached Table C. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table C.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The following site description should be read in conjunction with the attached Figure 2 and complimented with a site visit, if the reader is unfamiliar with the site.

The site is located at the crest of a large east-west oriented headland that is characterised by sub-vertical sandstone cliff lines around the perimeter. The cliff faces were controlled by orthogonal sub-vertical joint planes within the sandstone bedrock that were typically orientated (bearing) approximately north-south (bearings ranging between about 350˚ and 015˚) and east-west (bearings ranging between about 95˚ and 120˚). The headland is surrounded to the north and east by the Tasman Sea and to the south by Queenscliff Bay.

The site is bound by Carrington Parade to the west, Lumsdaine Drive to the north and Evans Street to the south. These surrounding streets are relatively flat to gently sloping, with typical grades less than or equal to about 8°. To the east of the site is McKillop Park, which is a mostly vacant area covered by dense bushland. However, there were several monuments/memorial structures located along the western side of the park, adjacent to an on-grade asphaltic concrete (AC) surfaced car park, as shown on Figure 2. The AC surfacing was in fair condition and contained some longitudinal and transverse cracks, as well as some potholes. The northern side of the car park had been filled to an estimated maximum height of about 3m, to create a more level ground surface.

The Club building occupied the central portion of the site, as shown on Figure 2. The eastern side of the Club comprised a two to three storey on-grade brick and cement rendered brick building. The western side of the Club comprised a two level basement car park, with several bowling greens on its roof. The lowest level of the basement car park had been cut into the hillside to maximum depths of about 3m and 1.5m along its northern and southern sides, respectively. The basement walls were concrete block or masonry. The south-western corner of the basement comprised a suspended concrete slab that was about least 1m higher than surrounding ground surface levels. The ground surface within the basement was mostly surfaced with AC. The Club building and AC surfacing within the basement appeared to be in generally good condition, based on a cursory inspection from within the site.

The areas surrounding the Club building, apart from the on-grade AC car park located on the eastern side of the Club building, were generally covered with grass and/or concrete pathways.

At the north-western corner of the site, there was an existing two storey brick residence (No. 4A Lumsdaine Drive) that appeared to be in good external condition, based on a cursory inspection from within the site.

To the south and west of the site across Carrington Parade and Evans Street, there were several neighbouring residential buildings, ranging in size from single storey cottages to multi-storey apartment buildings, that were all set back at least 20m from the subject site.

We note from obtained 'Dial Before You Dig' drawings of the site and immediate surroundings, there are buried services passing below the surrounding streets and footpath reserves, including water and sewerage pipes.

A summary of the primary geotechnical features identified at the site is presented below. The numbered items below correspond to the circled numbers in 'blue' shown on Figure 2. Photographs of the features were taken during our initial walkover inspection of the site and immediate surrounds in 2011 and are still considered valid.

1. **Sandstone Outcrops:** At the south-eastern corner of the on-grade AC car park, outcrops of distinctly weathered sandstone bedrock of low and medium strength were visible. The sandstone outcrops contained inclusions of fine to coarse grained, sub-angular and sub-rounded quartz gravel. The surface of the sandstone was 'dry'. Refer to Plates 1a and 1b below.

Plate 1(a): Sandstone outcrops at south-eastern corner of AC car park

Plate 1(b): Close up view showing quartz gravel inclusions within the sandstone outcrops

2. **Fill Batter Slope:** Along the northern side of the on-grade open air AC car park, there was a fill batter slope that was up to about 3m high. The fill batter slope graded between about 24˚ and 28˚ and was mostly vegetated. Where the vegetation cover was sparse, the fill batter slope surface exposed silty sand, with inclusions of sandstone gravel, cobbles and boulders. No evidence of deep seated slope instability (eg. landslides, bulging along the toe, tension cracks, etc.) were observed. Refer to Plate 2 below.

Plate 2: Fill batter slope along northern side of AC car park

3. **Shallow Cutting:** Along the northern side of the existing Club building, there was an existing sub-vertical cut, which was less than about 1m deep. The cut exposed a shallow soil cover (generally less than about 0.5m thickness) overlying sub-horizontally bedded, distinctly weathered sandstone bedrock. The soil comprised mostly silty sand fill overlying a thin layer of residual clayey sand. The bedrock was assessed to be initially extremely weathered sandstone of extremely low strength grading into distinctly weathered sandstone and of very low, low and medium strength. There was no evidence of any groundwater seepage through the cut slope or of cut slope instability. Refer to Plate 3 below.

Plate 3: Shallow cutting along northern side of existing Club building.

4. **'Boggy' Ground Surface:** During our walkover inspection in 2011, the ground surface at the toe of the grass covered slope at the north-western corner of the site was 'boggy' under foot. It is likely the soils had become saturated from wet weather at that time, as a result of seepage flows at the fill/natural soil interface. Refer to Plate 4 below.

   We note that during the current fieldwork, however, the ground surface had dried out in that area and trafficability was good for the drill rig.

Plate 4: Area of 'boggy' ground (approximately indicated in 'red) at north-western corner of site (2011)

5. **Sandstone Outcrops:** On the eastern side of No 4A Lumsdaine Drive, an outcrop of distinctly weathered sandstone bedrock of medium and high strength, was visible. The sandstone contained inclusions of fine to coarse grained, sub-angular and sub-rounded quartz gravel. Refer to Plate 5 below.

Plate 5: Sandstone outcrops on the eastern side of No.4A Lumsdaine Drive

6. **Mortared Sandstone Block Retaining Walls:** At the south-western corner of the site, adjacent to the intersection of Carrington Parade and Evans Street, there were several mortared sandstone block retaining walls, which supported the subject site to a maximum height of about 1.1m. During our 2011 walkover inspection, groundwater seepage emanated from the base of the retaining wall, which ran along the Evans Street footpath. The retaining walls spanned a shallow gully feature. The retaining walls appeared to be in good condition. Distinctly weathered sandstone bedrock of at least medium strength outcropped along the sides of the gully feature. Refer to Plate 6 below.

Plate 6: View looking across Evans Street towards the south-western corner of the site, showing low height mortared sandstone block retaining walls and a shallow gully feature.

3.2 Subsurface Conditions

With reference to the 1:100,000 geological map of Sydney, the site is underlain by Hawkesbury Sandstone. Underlying the Hawkesbury Sandstone are rocks from the Newport Formation of the Narrabeen Group which comprise *'Interbedded laminate, shale and quartz, to lithic-quartz sandstone: minor red claystone north'*. The Newport Formation is a deeply weathered rock formation with the rock quality being extremely variable.

There is no indication on the geological map that dykes (a dyke is a sub-vertical igneous intrusion) pass through the site or the immediate surrounding area.

In summary, the boreholes have disclosed a subsurface profile comprising fill and/or residual soils overlying weathered sandstone and shale bedrock at shallow and moderate depth. Reference should be made to the attached borehole logs for specific details at each location.

Graphical borehole summaries are presented as Figures 4 to 9, with the proposed bulk excavation level indicated on each figure.

A summary of the subsurface characteristics is presented below:

Pavements

A 25mm thick AC wearing surface was encountered at the top of BH103. A reinforced concrete slab 220mm, 240mm, 120mm and 200mm thick was encountered at the surface of BH203, BH203A, BH205 and BH206, respectively.

The concrete slab at BH206 was surfaced with tiles.

Fill

Fill was encountered below the pavements in BH102, BH203, BH203A, BH205 and BH206 and from the ground surface in the remaining boreholes and extended down to depths between 0.2m (BH201) and 2.8m (BH102) below existing grade. Inclusions of ironstone and sandstone gravel, root fibres and concrete, ceramic and glass fragments were present in the fill.

Where tested, the fill was assessed to be either poorly compacted (BH105) or well compacted (BH102).

The deepest fill in the boreholes was encountered in BH102 which was located within the gully feature (ie. at the south-west corner of the site).

Residual Soils

Residual soils comprising either sand, clayey sand or silty clay were encountered below the fill in BH101, BH102, BH104, BH201 and BH204. The silty clay was of medium or high plasticity and stiff or hard strength. The clayey sand was loose and medium dense.

Weathered Sandstone and Shale Bedrock

Weathered sandstone and shale bedrock was encountered below the fill and residual soils in each borehole at depths between 0.22m (BH205) and 3.8m (BH102) and extended down to the borehole termination depths.

Based on a visual assessment of the rock cores, the upper weathered rock profile comprised Hawkesbury Sandstone which was assessed to extend to depths between about 4.3m (BH203) and 7.8m (BH202) below existing grade. Below the Hawkesbury Sandstone, the rock profile comprised weathered shale and mostly fine grained sandstone bedrock from the Newport Formation. The weathered rocks assessed to be from the Newport Formation were extremely variable in both quality and degree of weathering.

In each borehole, with the exception of BH101, the weathered shale and sandstone bedrock was interbedded between depths of about 3.5m and 18.0m, below existing grade.

The weathered shale and sandstone bedrock profile was extremely weathered and of extremely low strength to slightly weathered and fresh of medium and high strength. In BH101 at 2.2m depth, there was a 450mm thick band of very high strength sandstone. Quite often there were thick extremely weathered bands within zones of more competent bedrock.

The cored portions of the bedrock contained sub-horizontal defects including extremely weathered seams/bands, clay seams and bedding partings. Inclined joints were also encountered in each borehole. With the exception of BH104, BH202, BH204 and BH206, core loss zones were encountered in each borehole at depths between 1.08m and 10.65m and ranged between 100mm and 750mm thick. The core loss zones are inferred to be extremely weathered bands or clay bands which have “washed away” during the coring process.

An indicative engineering classification of the bedrock (in accordance with Pells *et al* 1998) has been carried out and is tabulated below:

Borehole	Approx. Surface RL (m) AHD	Indicative Engineering Classification of Bedrock Depths (m)				
		Class V	**Class IV**	**Class III**	**Class II**	**Class I**
BH101	22.0	0.6 – 3.25 (Sandstone) 7.9 – 10.4 (Sandstone)	–	3.25 – 5.8 (Sandstone) 10.4 – 12.93 (Shale)	–	5.8 – 7.9 (Sandstone)
BH102	19.5	3.8 – 7.4 (Sandstone/Shale), 10.1 – 11.2 (Sandstone/Shale)	–	–	7.4 – 10.1 (Shale)	–
BH103	24.1	0.4 – 8.8 (Sandstone)	–	8.8 – 11.1 (Shale) 11.1 – 15.13 (Sandstone)	–	–
BH104	18.0	3.5 – 5.1 (Shale)	–	5.1 – 7.0 (Sandstone)	7.0 – 9.79 (Sandstone/Shale)	–
BH105	23.5	1.65 – 10.4 (Sandstone/Shale)	10.4 – 12.3 (Sandstone/Shale)	–	12.3 – 15.55 (Sandstone)	–
BH201	20.7	2.8 – 4.4 (Sandstone) 5.5 – 8.2 (Sandstone)	0.5 – 1.1 (Sandstone) 4.4 – 5.5 (Sandstone) 13.3 – 15.2 (Sandstone)	1.1 – 2.8 (Sandstone)	8.2 – 13.3 (Sandstone)	–
BH202	23.5	7.8 – 9.2 (Shale)	0.7 – 1.7 (Sandstone) 5.0 – 7.8 (Sandstone) 17.1 – 19.3 (Sandstone)	–	1.7 – 5.0 (Sandstone)	9.2 – 17.1 (Shale)
BH203	20.2	2.0 – 4.0 (Sandstone) 4.0 – 5.2 (Shale)	0.4 – 2.0 (Sandstone)	–	5.2 – 7.4 (Shale)	–
BH203A		0.5 – 4.3 (Sandstone) 4.3 – 5.5 (Shale) 11.7 – 13.6 (Shale)		5.5 – 7.3 (Shale) 13.6 – 15.0 (Sandstone)	7.3 – 11.7 (Shale)	
BH204	17.8	1.5 – 3.8 (Shale) 10.8 – 12.6 (Sandstone)	3.8 – 4.5 (Sandstone)	4.5 – 6.2 (Sandstone)	6.2 – 9.2 (Sandstone) 9.9 – 10.8 (Shale)	9.2 – 9.9 (Sandstone) 12.6 – 13.4 (Sandstone)
BH205	22.7	0.2 – 6.3 (Sandstone) 6.3 – 7.2 (Shale)	–	–	–	7.2 – 10.0 (Shale)
BH206	25.1	11.7 – 13.2 (Sandstone)	2.1 – 2.9 (Sandstone)	2.9 – 4.9 (Sandstone) 7.1 – 11.7 (Sandstone)	13.2 – 17.9 (Shale) 17.9 – 19.5 (Sandstone)	4.9 – 7.1 (Sandstone)

Groundwater

All boreholes, with the exception of BH102, were ‘dry’ during auger drilling and on completion of auger drilling. In BH102, groundwater seepage was encountered at 4m depth (ie. just below the soil/rock interface), with groundwater measured at the same depth on completion of auger drilling.

On completion of coring, groundwater was measured at depths between 2.0m and 3.6m. As water is used in the coring process, these groundwater levels have most likely been affected by the introduced drill flush water. There was generally a full return of the drill flush water, which indicates a relatively impermeable rock mass. However, we note that a 50% return was estimated over the basal portion of BH201 and no return of the drill flush water was achieved in BH205. The latter indicate open defects in the rock mass.

The following longer term groundwater levels were measured:

Borehole	Date Drilled	Groundwater Depth (Level)		
		22.02.15	28.02.15	09.03.15
BH201	19.02.15	8.05m (12.65m AHD)		8.32m (12.38m AHD)
BH204	25.02.15		2.95m (14.85m AHD)	3.03m (14.77m AHD)

3.3 Laboratory Test Results

The results of the moisture content and Point Load Strength Index tests carried out on recovered rock chip samples and recovered rock cores generally correlated well with our field assessment of bedrock strength. The estimated UCSs ranged between less than 1MPa and 68MPa. However, UCSs of 110MPa, 96MPa and 108MPa were estimated in BH101 (2.61m depth), BH201 (8.71m depth) and BH202 (10.8m depth), respectively.

The soil pH tests results were between values of 5.0 and 8.8, which show the samples tested to be acidic to slightly alkaline. The soil sulphate and chloride test results were less than or equal to 240mg/kg, which indicate low sulphate and chloride contents.

The four day soaked CBR test carried out on a residual silty clay sample from BH104 resulted in a value of 1% when compacted to 98% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The sample was compacted prior to CBR testing at close to its Standard Optimum Moisture Content (SOMC). The insitu moisture content of the sample tested was 2.2% ‘wet’ of the SOMC.

3.4 Pump-Out Test Results

The groundwater levels in the standpipes which were installed in BH201 and BH204 were measured on 9 March 2015 (approximately 18 days and 12 days after drilling, respectively), and the standpipes flushed. A second flush was carried out and the rate of recovery of the groundwater levels was measured using electronic data loggers. Using established seepage formulae, a mass permeability for the rock mass of about 10^{-7}m/sec is indicated.

4 STABILITY ASSESSMENT

Based on the results of the geotechnical mapping carried out, we did not observe any obvious signs of deep seated instability at, or in the immediate vicinity of, the subject site. Furthermore, where the sandstone bedrock does not outcrop, the site is underlain by sandstone bedrock at relatively shallow depth and is situated on the crest of a headland and therefore deep seated instability is not expected at this site.

With reference to Part E10 (Landslip Risk) of Warringah Council's Development Control Plan (DCP) and the Council's Landslip Risk Map, the site is located in Area B (Flanking slopes from 5 to 25 degrees).

With reference Section 6.4 Part 1(a) and 3(a) of Council's Local Environmental Plan (LEP) 2011, we consider the likelihood of a deep seated failure ie. a landslide, through the subsurface profile where bedrock is relatively shallow to be *'Barely Credible'*, based on the guidelines given in the Australian Geomechanics Society (AGS) (2007c) *'Practice Note Guidelines for Landslide Risk Management'*. The attached Appendix B defines the terminology, together with a flow chart illustrating the Risk Management Process. We note that if a deep seated landslide did occur on the site, a *'Major'* consequence to property would result, with reference to the attached AGS extract. The corresponding risk level to property is 'Very Low', which is considered 'Acceptable'.

We have also assessed the risk to life for a potential deep seated landslide at the site, as well as a localised failure of an excavation cut face, both during the construction period and on completion of construction, in accordance with the AGS guidelines referenced above. A summary of the risk to life for these potential landslide hazards are tabulated below.

Potential Landslide Hazard – Deep Seated Failure ie. Landslide

Assessed Likelihood	Barely Credible (10^{-6})
Person at Risk	Person inside excavation (During Construction) Occupant inside proposed Club Building (After Construction)
Duration of Use of Area Affected (Temporal Probability)	Say 8 hours per day for 6 days per week during construction (assuming one year). Assume failure could occur over a 10m length of an approximate 50m long cut face. Therefore, 8/24 x 6/7 x 10/50 = 0.06 Say 16 hours per day for 7 days per week after construction. Assume failure could occur over a 10m length of an approximate 50m long cut face. Therefore, 16/24 x 7/7 x 10/50 = 0.13
Probability of Not Evacuating Area Affected	0.9 (during construction) – failure could be rapid. 0.1 (after construction) – structure to be engineer designed, some warning signs of movement likely, ie. cracking, bulging retaining wall etc.
Vulnerability to Life if Failure Occurs Whilst Person Present	1.0 (both during and after construction)
Total Risk for Person Most at Risk	$5.4x10^{-8}$ (during construction) $1.3x10^{-8}$ (after construction)

The resulting Total Risk for the Person Most at Risk is about 5×10^{-8} (during construction) and 1×10^{-8} (after construction), which would both be considered 'Acceptable', in relation to the AGS criteria.

Potential Landslide Hazard – Failure of Excavation Cut Face (During Construction)

Assessed Likelihood	Rare (10^{-5}) – Assumes cut faces are inspected by a geotechnical engineer as per the recommendations in this report
Person at Risk	Person inside excavation
Duration of Use of Area Affected (Temporal Probability)	Say 8 hours per day for 6 days per week during construction (assuming one year). Assume failure could occur over a 3m length of an approximate 50m long cut face. Therefore, 8/24 x 6/7 x 3/50 = 0.02
Probability of Not Evacuating Area Affected	0.9 – failure could be rapid.
Vulnerability to Life if Failure Occurs Whilst Person Present	1.0
Total Risk for Person Most at Risk	$1.8x10^{-7}$

The resulting Total Risk for the Person Most at Risk is about 2×10^{-7}, which would be considered 'Acceptable', in relation to the AGS criteria.

In relation to Part 6.4 1(b) of the Council LEP noted above, we assume that all stormwater runoff from the site would be collected and subsequently discharged in a controlled manner to the stormwater system. Assessing the disposal of stormwater is not a geotechnical issue and would be the responsibility of either the civil or hydraulic engineer. Provided stormwater runoff is disposed of in a controlled manner, no adverse impact on stability of the subject site and surrounding land is expected as a result of stormwater disposal.

5 COMMENTS AND RECOMMENDATIONS

5.1 Suitability of the Site for Redevelopment

Based on the investigation results and our stability assessment, it is our opinion that the proposed redevelopment is feasible from a geotechnical perspective, provided the comments and recommendations below are adopted in their entirety.

The proposed redevelopment will incorporate common construction techniques and methodologies carried out on many sites throughout Sydney and within the local area.

5.2 Geotechnical Issues

The primary geotechnical issues associated with the proposed redevelopment will be to maintain stability to the adjoining footpath reserves, roads and the on-grade AC car park to the east, other nearby structures and buried services, both during excavation and in the long term. Furthermore, there will the need to reduce the risk of vibration induced damage to nearby buildings and structures, during demolition and subsequent excavation.

We strongly recommend that prior to the commencement of demolition and excavation, a pre-construction meeting be held with representatives from the Club, the architect, the builder, the excavation contractor, the structural engineer and the geotechnical engineer, so that the geotechnical issues and constraints can be discussed, understood and accepted.

The geotechnical investigation has provided a basis for the comments and recommendations which follow. However, it will be essential during excavation and construction work that frequent geotechnical inspections are carried out to assess exposed subsurface conditions, so as to provide appropriate geotechnical advice.

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

5.3 Excavation

The excavation recommendations provided below should be complemented by reference to the Safe Work Australia 'Code of Practice – Excavation Work'.

5.3.1 Dilapidation Surveys

Prior to the commencement of demolition and excavation, we recommend that detailed dilapidation reports be compiled on the monuments/memorial structures located within McKillop Park to the east, the on-grade AC car park to the east, the adjoining road surfaces and the neighbouring residential properties located on the western and southern sides of Carrington Parade and Evans Street.

Dilapidation surveys should include detailed inspections, where all defects are vigorously described (including defect type, length and width) and photographed.

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

5.3.2 Site Preparation

The site preparation works will comprise demolition of the existing structures on site and the house located on No. 4A Lumsdaine Drive, as well as removal of plants and trees, including their root balls. All grass, topsoil, root affected soils and any deleterious or contaminated existing fill should also be stripped. Reference should be made to the EIS reports for guidance on the offsite disposal of soil.

5.3.3 Excavation Conditions

Excavation of the soil and extremely weathered bedrock profiles can be completed using large hydraulic excavators. It may be possible to remove the upper very low to low strength sandstone bedrock encountered in BH105 using a ‘digging’ bucket fitted to a very large excavator, however, ripping tyne and/or rock hammer assistance may also be required.

During bulk excavations, we expect that the excavation of the low and higher strength bedrock will present ‘hard rock’ excavation conditions. However, the presence of weaker and more weathered bands within the rock mass will assist with excavation. Ripping to Class III sandstone bedrock will be possible with a Caterpillar D9 dozer or equivalent. However, for Class II or I bedrock and also to improve excavation production rates, a very generous allowance should be made for rock hammer assistance to the ripping. Excavation production rates are likely to be very low and shoe wear rates high, particularly in the more competent bedrock. Further, higher wear and tear rates of the excavation equipment should be expected due to the presence of quartz gravel inclusions within the rock mass. Grid sawing the sandstone bedrock in conjunction with ripping and/or hammering would also help to facilitate excavation.

For detailed excavations below bulk level, eg. for footings, trenches, lift pits etc., we suggest that the perimeter of the proposed excavation be saw cut and hydraulic hammers or ripping tynes be used for the intermediate rock.

Dust suppression by spraying with water should be carried out whenever rock saws are being used.

Rock excavations using hydraulic rock hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to nearby structures and buried services. We recommend that quantitative vibration monitoring be carried out whenever hydraulic rock hammers are used during rock excavation on this site, as a safeguard against possible vibration induced damage. By referencing the relevant German Standard DIN4150-3:1999-02 and British Standard BS7385-2:1993, the vibrations on the closest nearby houses should be limited to a peak particle velocity of 5mm/s (at 10Hz), subject to review of the dilapidation survey reports. It should be noted when vibration limits are exceeded, they should be assessed against the attached Vibration Emission Design Goals sheet, as higher vibrations may be acceptable depending on the vibration frequency. If it is found during monitoring that transmitted vibrations are excessive, then it would be necessary to change to a smaller rock hammer. Otherwise, geotechnical advice could be sought with respect to alternative excavation options.

The monitoring must include the installation of vibration monitors (equipped with data loggers which provide graphical presentation of vibration velocity versus vibration frequency) which measure transverse, vertical and longitudinal ground vibrations and their vector sum. The monitors must be installed on the structure at No 22 Carrington Parade, No 67 Evans Street and No 73 Evans Street (subject to owner's approval).

The following procedures are recommended to reduce vibrations if rock hammers are used:

- Rock saw the perimeter faces. This will increase the path and effectively reduce ground borne vibrations provided the base of the rock saw slot is maintained at a lower level than the adjacent excavation level at all times. Rock sawing would also improve the aesthetics of the finished rock faces.
- Maintain rock hammer oriented towards the face and enlarge excavation by breaking small wedges off face.
- Operate the hammer in short bursts only, to reduce 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.

We recommend that a copy of this report be provided to the prospective excavation contractors so that they can make their own assessment of excavation conditions.

5.3.4 Groundwater Seepage

Groundwater inflows into the excavation are expected as local seepage flows within the fill, at the fill/residual soil interface, through gravel bands or relic joints/fissures within the residual silty clay and through joints and bedding partings within the bedrock profile, particularly after heavy rain.

Using the mass permeability of the rock formations underlying the site (refer Section 3.4 above), we have predicted an infiltration rate into the bulk excavation of about 5m^3/day or less than 2ML/year.

However, given the location of the site on a headland with an elevation significantly higher than the adjacent Freshwater valley to the south, an upslope catchment of limited extent and the lack of significant visible groundwater seepage from the clifflines below the site to the north, south and east, the above predicted inflow rate of groundwater seepage into the bulk excavation is likely to be an upper limit.

The above estimated 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.

5.3.5 Stress Relief

In Sydney, there is a relatively high in-situ horizontal stress field. When excavations extend down into the sandstone bedrock, the horizontal stresses are relieved, resulting in movement of the excavated faces into the excavation. These movements occur along sub-vertical bedding partings and are generally in the order of about 0.5mm to 1mm for each metre depth of excavation into the sandstone bedrock. Therefore, a predicted lateral movement between about 10mm and 20mm may occur in the vicinity of the deepest portion of the excavation where the sandstone outcrops, with movements expected to reduce with distance away from the cut face. However, as the site is located on a headland, we consider that most of the stress relief has already occurred, and therefore lateral movements due to stress relief are to be expected to be at the lower end of this range, but probably even less.

Due to the high magnitude of the insitu stresses, it is not feasible to restrain the excavated faces from these movements. In our opinion, and based on the above, we do not consider these stresses will adversely impact surrounding buildings.

This will significantly reduce the extent of temporary stabilisation works which will subsequently be removed to allow the second phase of the works to commence.

To further assess the magnitude of the lateral movements due to horizontal stress relief, further detailed investigation comprising insitu rock stress testing followed by finite element modelling could be undertaken. We note, however, that this level of detailed investigation and subsequent analysis is rarely undertaken in Sydney, with exception of where rail infrastructure immediately adjoins the development site.

5.4 Excavation Support

5.4.1 Batter Slopes

The following temporary and permanent batter slopes apply for the proposed bulk excavation, provided surcharge loads are kept well away from the batter slope crests:

Material Type	Temporary batter Slope	Permanent Batter Slope (provided the batter is protected from erosion)
Sandy soils	1 Vertical (V) in 1.5 Horizontal (H)	1V in 2H
Clayey soils	1V in 1H	1V in 2H
Class V Sandstone and Class V/IV Sandstone Bedrock	1V in 1H	1V in 1.5H
Class IV Sandstone Bedrock	1V in 0.5H	1V in 1H
Class III or better Sandstone and Shale Bedrock	Can be cut vertically, subject to geotechnical inspections every 1.5m depth of excavation to check for adverse defects, weathered seams, etc that require stabilisation.	

Based on the borehole logs, architectural drawings and survey plan, the above batters cannot be accommodated within the site geometry. However, the temporary excavated face which will be required adjacent to the club building for the initial phase of the works can be cut with the soil batter as above and set back at least 0.5m from the building. A horizontal bench 0.5m wide should then be provided at the toe of the soil batter and the underlying rock excavated at a batter of 1V in 1H. Although areas of steeper batters in the better quality rock as above are feasible, a uniform batter slope has been recommended for practical considerations. Erosion protection of the soil batter will be required, and can comprise shotcrete, stone pitching, etc.

5.4.2 Support Systems

We recommend that the proposed vertical cuts in the soil and weathered bedrock profiles be supported by a soldier pile wall with reinforced shotcrete infill panels. However, in those areas where sandy fill deeper than about 1m is present (such as in the vicinity of BH102 and BH105) the proposed vertical cuts should be supported by a contiguous pile wall. Alternatively, timber lagging can be installed progressively behind the soldier piles as excavation proceeds. The lateral extent of the contiguous piled walls or timber lagging should be assessed at the commencement of piling by excavation of a few test pits in the vicinity of BH102 and BH105 in the presence of a geotechnical engineer. However, based on the DCP test results, the lateral extent should be tentatively assumed to extend between DCP207/DCP208 and DCP210 in the vicinity of BH102 and northwards to DCP211 in the vicinity of BH105.

The piles must be installed prior to the commencement of excavation and must be progressively shotcreted/timber lagged and anchored, or internally propped, as excavation proceeds (ie. once the restraining point has been uncovered). Careful control of the construction sequence will be required to reduce potential movements.

We strongly recommend that a full copy of this report be provided to the prospective piling contractors so that appropriate drilling rigs and equipment are brought to site.

Construction of the pile walls must be of high quality. The shotcrete/timber infill panels must be completed without delay to (1) reduce the shrinkage of clay soils immediately outside the excavation and (2) limit potential rock wedge failures between soldier piles, if appropriate. Construction of the contiguous pile walls and solider pile walls with shotcrete/timber lagging must also be constructed with care so as to prevent soil loss through gaps that will most likely occur between piles/timbers, 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. Consideration would also have to be given to final treatment of exposed pile faces depending on aesthetic/architectural requirements.

Where sandy soils are present, the drilling of the piles may cause ground surface movements due to vibrations associated with pile drilling and possible collapse or ‘drawdown’ of soils into the pile drill holes and therefore care will be required by the piling contractor. Continual monitoring of the ground surface between the contiguous pile wall and adjoining surface levels should also be undertaken by the builder. If there are any signs of ground surface movement, then the piling operations should be immediately halted and further geotechnical advice sought.

The following options can be considered in terms of the extent of the above soldier pile wall retention system, viz full depth installation, terminate the piles above bulk excavation level such that the soil and upper Class V rock is retained, or terminate the piles above bulk excavation level such that only the soil profile is retained.

Full Depth Retention System

The proposed piles must be founded with sufficient embedment to satisfy stability and founding considerations for all stages of excavation and anchoring. We recommend that the shoring piles terminate at a depth not less than 0.5m below bulk excavation level (including nearby footings, service trenches and lift pits). The piles can be used as load bearing piles for the proposed new building if taken down to the appropriate founding depths.

The rock face between soldier piles must be progressively inspected by an experienced geotechnical engineer or engineering geologist at no more than 1.5m depth increments to assess the need for temporary support (eg. rock bolts, dowels, etc) of potentially unstable rock wedges. It may be possible in some areas for the rock faces to be left exposed (ie. the reinforced shotcrete could be omitted) in the long term, subject to geotechnical inspection.

Due to the presence of medium, high and very strength bedrock, only high torque drilling rigs equipped with rock augers and/or coring buckets, should be brought to site. If the sandy fill encountered is found to collapse into open pile holes, then the fill will need to be supported using temporary or sacrificial liners, or alternatively grout inject auger (otherwise known as continuous flight auger) piles may be used.

This is the least risky option and has the advantage that the geotechnical inspections and any localised stabilisation measures that may be required will be greatly reduced when compared to the other options.

The soldier pile wall should be designed based on the recommendations presented in Section 5.4.3 below. The construction sequence must be fully specified and carefully controlled to reduce potential movements. The sequencing and control must include close liaison with the geotechnical engineer.

Piles Terminated above Bulk Excavation to Support Soil and Class V Rock

The proposed soldier piles can be terminated above bulk excavation level to a depth such that the soil profile and upper Class V rock are retained. The soldier pile wall should be designed using the appropriate recommendations presented in Section 5.4.3 below (ie. i, ii, v and vii), but with the retention depth ‘H’ taken to the base of the upper Class V rock. A second lower row of anchors will be required to support the pile toes with excavation in front of the toe delayed until the anchors have been installed. For the long term, the piles will need to be supported by the floor slabs of the proposed buildings and this must be reflected in the pile depth and design.

The rock face below the soldier piles must be progressively inspected by an experienced geotechnical engineer as excavation proceeds to assess the need for temporary support, particularly of the Class IV and any lower Class V bedrock. We recommend that the above inspections be carried out at 1.5m depth intervals. It is likely that pattern bolting and shotcrete will be required for Class V and Class IV rock. The Class III or better rock may require localised stabilisation of adverse joints, bed parting or weathered seams.

The construction sequence must be fully specified and carefully controlled to reduce potential movements. The sequencing and control must include very close liaison with the geotechnical engineer.

__Piles Terminated above Bulk Excavation Level to Support the Soil Profile__

The proposed soldier piles can be terminated above bulk excavation level to a depth such that the soil profile is retained. The soldier pile wall should be designed using the appropriate recommendations presented in Section 5.4.3 below (ie. i, ii, v and vii), but with the retention depth ‘H’ taken to the base of the Class V rock. A second lower row of anchors will be required to support the pile toes with excavation in front of the toe delayed until the anchors have been installed. For the long term, the piles will need to be supported by the floor slab of the proposed buildings and this must be reflected in the pile depth and design.

The rock face below the soldier piles must be progressively inspected by an experienced geotechnical engineer as excavation proceeds to assess the need for temporary support, particularly of the Class IV and Class V bedrock. We recommend that the above inspections be carried out at 1.5m depth intervals. It is likely that pattern bolting and shotcrete will be required for Class IV rock. The Class III or better rock may require localised stabilisation of adverse joints, bed parting or weathered seams.

The Class V rock will need to be positively retained using a soil nail/rock bolt system designed on the basis of the pressures recommended in (i) and (ii) of Section 5.4.3 below, with ‘H’ taken to the base of the upper Class V rock.

We note that this option is associated with relatively high risks as the Class V rock will require immediate stabilisation on exposure and the final depth to the base of the Class V rock may not be evident. In order to control the risks associated with stability of the cut face, we consider that an experienced geotechnical engineer would need to be present on a full time basis during bulk excavation with the stabilisation subcontractor (for anchors and shotcrete) also in full time attendance or on stand-by. The sequencing of the bulk excavation will need to be carefully considered with provision for flexibility to adapt to the different conditions being exposed. The construction sequence must also be carefully controlled to reduce potential movements. The sequencing and control must be directed by the geotechnical engineer.

As a starting point, we would recommend that as the bulk excavation approaches the final cut face, it be stopped, say 3m short, and the face be inspected by the geotechnical engineer. The final approach should then be completed in ‘hit and miss’ sections 5m wide with each section inspected and appropriately stabilised prior to the adjacent section being excavated.

There would also be a risk associated with costs of the excavation and stabilisation measures as these cannot be quantified to any degree of accuracy, pre-commencement.

Rock Bolt Details for Estimation Purposes

The retention options of the excavation provided above will need to be designed using the parameters presented in Section 5.4.3 below.

However for planning and estimation purposes the following rock bolt stabilization measures are provided:

- Class V shale and sandstone – rock bolts at 2m centres vertical and horizontal with drained shotcrete. Half the rock bolts should b 6m long and half 4m long.
- Class IV shale and sandstone – rock bolts + shotcrete as above but all 4m long.
- Class III or better – individual rock bolts; assume 1x4m long and 1x6m long for each 10 linear metres of the excavation perimeter.

We reiterate that the above is for estimation purposes and not to be used without detailed engineering design of the excavation support.

5.4.3 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 a static design of the retention system.

(i) For progressively anchored or propped walls, where some minor movements can be tolerated (ie. assuming that there are no movement sensitive buried services within the zone of influence of the piled walls), we recommend the use of a trapezoidal earth pressure distribution and a lateral earth pressure of 6H (kPa) for the soil profile and upper weathered bedrock profile (Class V & IV), where H is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system. For the shotcrete infill panel design, a trapezoidal earth pressure distribution and a lateral earth pressure of 4H (kPa) can be adopted for the soil and weathered bedrock profiles.

(ii) For progressively anchored or propped walls located in areas which are highly sensitive to lateral movement (ie. where there are movement sensitive buried services located within the zone of influence of the piled walls), we recommend the use of a trapezoidal earth pressure distribution and a lateral earth pressure of 8H (kPa) for the soil profile and upper weathered bedrock profile (Class V & IV), where H is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system. For the shotcrete infill panel design, a trapezoidal earth pressure distribution and a lateral earth pressure of 6H (kPa) can be adopted for the soil and weathered bedrock profiles.

(iii) A nominal rectangular lateral earth pressure of 10kPa should be adopted for the Class III or better shale and sandstone profiles.

(iv) Refer to attached Figure 10 for typical recommended lateral design pressures for full depth anchored or propped retaining walls.

(v) Any surcharge affecting the walls (eg. immediately adjacent building footings, construction loads, inclined backfill, etc.) should be allowed in the design using an ‘at rest’ earth pressure coefficient (K_0) of 0.55 for the soil and upper weathered bedrock profiles, assuming a horizontal backfill surface.

(vi) A bulk unit weight of 20kN/m^3 should be adopted for the soil and Class V and IV bedrock profiles.

(vii) The retaining walls should be designed as fully drained with measures undertaken to induce complete and permanent drainage of the ground behind the walls. Strip drains must be provided mid-length between soldier piles and/or rock bolts. If large spans are proposed between soldier piles or rock bolts, then additional strip drains may be required. The drainage for the contiguous pile walls should comprise a row of weepholes made up of, say, 50mm PVC pipes which are grouted into gaps or holes between adjacent piles at say, 1.5m horizontal spacing and located about 0.3m above the proposed basement floor slab. The embedded end of such weepholes must be covered by a non-woven geotextile filter fabric (such as Bidim A34 or similar). All drainage water should be piped to the stormwater system.

(viii) For perimeter piles embedded at least 0.5m into Class III or better quality sandstone or shale bedrock below bulk excavation level (including nearby footings, service trenches and lift pits), a maximum allowable lateral toe resistance of 350kPa may be adopted. The above design value assumes excavation is not carried out within the zone of influence of the wall toe. The upper 0.2m depth of the socket should not be taken into account to allow for tolerance effects and possible disturbance during excavation.

Typical K_a and K_p values for the soil and rock types encountered are presented below:

Material Type		K_a (assumes horizontal backfill surface)	K_p (assumes horizontal ground in front of the wall)
Sandy Fill		0.35	2.8
Residual Clayey Sand		0.33 (loose) 0.3 (medium dense)	3.0 (loose) 3.3 (medium dense)
Silty Clay (Hard)		0.35	2.8
Sandstone Bedrock	Class V	0.3	3.3
	Class IV	0.2	5.0
	Class III (or better)	N/A (Self-supporting, subject to geotechnical inspection)	N/A (An allowable lateral toe resistance of 350kPa may be adopted)
Shale Bedrock	Class V	0.35	2.8
	Class IV	0.3	3.3
	Class III (or better)	N/A (Self-supporting, subject to geotechnical inspection)	N/A (An allowable lateral toe resistance of 350kPa may be adopted)

Also, the modulus of subgrade reaction for the different material types and a selection of pile diameters which can be used for retention design is presented below:

Material Type	Modulus of Subgrade Reaction (kPa/mm) Applicable to Pile Diameter		
	300mm	450mm	600mm
Sandy Fill	20	14	10
Residual Clayey Sand (loose)	31	20	15
Residual Clayey Sand (medium dense)	81	54	41
Residual Clay	81	54	41
Shale: Class V	102	68	51
Class IV	198	132	99
Class III	395	263	198
Class II	1,350	900	675
Sandstone: Class V	102	68	51
Class IV	204	136	102
Class III	691	461	346
Class II	1,736	1,157	868
Class I	3,858	2,572	1,929

5.4.4 Rock Anchors/Rock Bolts

If rock anchors or bolts are to extend outside the site boundaries, then permission must be sought from the respective neighbouring property owners and Council prior to installation. Our experience has shown that this process can take time and therefore should be completed as early as possible.

Temporary rock anchors should be bonded at least 3m into bedrock, with the bond length being fully beyond a line drawn up at 45˚ from the base of the zone being retained. The temporary anchors may be designed on the basis of a maximum allowable bond stress of 250kPa, provided the rock is of at least low strength.

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. The testing may allow an upgrading of the above bond stress. We recommend that only experienced contractors be considered for the anchor installations.

Rock bolts should be bonded behind an imaginary line which extends up at 45° from the base of the soil and/or rock unit being stabilised, and designed for an allowable bond stress of 250kPa.
We have assumed that permanent lateral support of the soldier pile and contiguous walls will be provided by the proposed new building.

If permanent rock anchors and bolts are considered, with the building structures constructed independently of the excavation, then the initial retention option (ie. full depth soldier piles referred to in Section 5.4.2) is appropriate. The anchors and rock bolts should be designed as above and also for corrosion resistance and long term durability (ie. double encapsulated/ stainless steel).

In addition, provision will need to be made to allow for future inspection of the retained face with a formal monitoring program developed. To allow for future restressing, the anchor strands must not be cut off and the anchor head must be protected with a grease pot.

5.4.5 Excavation Related Movements

We recommend that a’ trigger’ level for movement of the proposed basement excavation retention system where intervention is required during construction, be set as follows:

Green (Alert) Zone:

Movements up to 8mm. Construction can continue with ‘normal’ input from the geotechnical engineer.

Amber (Action) Zone:

Movements between 8mm and 15mm. The geotechnical engineer must be informed, the monitoring information reviewed by the geotechnical and structural engineers, and construction can continue under the advice of the geotechnical and structural engineers. Additional monitoring may be requested.

Red (Alarm) Zone:

Movements in excess of 15mm. All construction activities must immediately cease and the geotechnical and structural engineers informed immediately. Construction can only recommence following approval from the structural and geotechnical engineers and will include a risk assessment and may require some structural redesign, methodology change and/or additional/further monitoring.

The builder must outline in their Construction Monitoring Program (CMP) (refer to Section 5.11 below) their method for monitoring wall movements, including monitoring points, frequency and methodology. However, we expect that monitoring points no more than 25m apart along the capping beam and at 3m depth intervals will be nominated. The movement monitoring program may need to be reviewed depending on the retention option adopted and the presence of movement sensitive buried services in close proximity.

5.5 Footings

Based on the investigation results, variable bedrock type, strength and quality will be exposed at bulk excavation level.

For uniformity of the support and design, we recommend that all pad and strip footings founded in Class III or better quality shale or sandstone bedrock and designed for a maximum allowable bearing pressure of 3,500kPa.

Perimeter shoring piles socketed at least 0.5m into Class III or better quality shale or sandstone bedrock may be designed for a maximum allowable bearing pressure of 3,500kPa. Sockets formed below the minimum 0.5m length requirement (soldier piles only) may be designed for a maximum allowable shaft adhesion value of 350kPa (compression) on condition that the pile shaft is suitably roughened using a grooving tool fitted to the side of the auger. Due to the presence of medium and high strength bedrock at depth, as well as the expected presence of quartz gravel inclusions in the rock mass, on which slow penetration rates and high bit wear should be expected, care should be taken not to design long rock socket lengths unless large, high torque piling rigs with appropriate equipment are to be used.

The above provided allowable bearing pressures are based upon serviceability criteria of deflections at the footing base of less than 1% of the minimum footing dimension/pile diameter.

For a limit state design, the following ultimate design values are applicable:

Material		**Ultimate End Bearing Pressure (kPa)**	**Allowable End Bearing Pressure (kPa)**	**Ultimate Shaft Adhesion (kPa) (Compression only)**	**Allowable Shaft Adhesion (kPa) (Compression only)**	**Elastic Modulus (MPa)**
Sandstone Bedrock	Class V	3,000	1,000	150	100	80
	Class IV	12,000	3,500	600	350	200
	Class III	30,000	5,000	1,000	500	800
	Class II	90,000	9,000	2,000	900	1,200
	Class I	120,000	12,000	3,000	1,000	2,000
Shale Bedrock	Class V	3,000	700	100	70	50
	Class IV	4,000	1,000	200	100	200
	Class III	20,000	3,500	600	350	800
	Class II	70,000	5,000	1,000	500	1,200

For uniformity of design, for pad and strip footings founded in, or for piles socketed at least 0.5m into, Class III or better quality shale or sandstone, we recommend that an ultimate bearing pressure of 20MPa be adopted together with an ultimate shaft adhesion (compression) of 600kPa. The above ultimate values must be used in conjunction with an appropriate geotechnical strength reduction factor. The geotechnical strength reduction factor (ϕ_g) will need to be determined for the project site/designer/testing specifics but provided good design practices are adopted, there is good workmanship and quality control during footing construction and the inspections detailed below are adopted, a ϕ_g value of 0.56 is considered to be appropriate.

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 pad an strip footings designed for allowable bearing pressures up to 3,500kPa (20MPa ultimate) should be cleaned out and inspected by a geotechnical engineer immediately prior to pouring (ie. spoon testing is not required). We recommend that the bored pile drilling be inspected by a geotechnical engineer during the initial stages and then periodically during the works to confirm that a satisfactory bearing stratum has been achieved. Conventional bored piles should be cleaned out, inspected and poured on the same day as drilling. Any seepage into the open pile holes must be removed immediately prior to pouring.

5.6 Basement 2 Level Floor Slab

Based on the investigation results, the Basement 2 Level floor slab will directly overlie bedrock. We therefore 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 include a sump and pump dewatering system. The retaining wall drains should be connected into the underfloor drainage system. Groundwater seepage monitoring should be carried out during basement excavation prior to finalising the design of the pump out facility. Outlets into the stormwater system will require Council approval.

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

5.7 Earthquake Design Parameters:

Based on the investigation results and in accordance with AS1170.4–2007, a Hazard Factor (Z) of 0.08 is applicable for the site, together with a subsoil Class C_e.

If, however, the buildings are constructed independently of the excavation, and not in contact with the excavation side slopes, then a Class B_e rock applies.

5.8 Soil Aggression

Based on the soil chemistry test results, a ‘mild’ exposure classification is applicable for concrete in accordance with Table 6.4.2 (C) in AS2159-2009.

5.9 External Pavements

Based on the laboratory test results, external pavements underlain by residual silty clay or sandy fill/weathered bedrock may be designed for CBR values of 1% and 10%, respectively, or Short Term Young’s Modulus (E) of 10MPa and 40MPa, respectively.

As a guide, in areas where the weaker subgrade (ie. CBR value of 1%) is present, the inclusion of a 0.3m thick (compacted) select fill layer of CBR≥20% crushed sandstone, would increase the equivalent subgrade design CBR value to 4%.

The select fill must comprise a well graded, granular crushed sandstone (maximum particle size of 75mm) with a soaked CBR value of at least 20%. If the available sandstone is assessed by tactile examination or laboratory testing to be a borderline material (ie. achieving a CBR value of just over 20% at a compaction density ratio of 100% of SMDD), then we expect that it will break down and degrade during compaction with a heavy roller to a material with an "insitu" CBR value less than 20%. As such, we recommend that the CBR testing allow for the degradation of the crushed sandstone. The standardised RTA Specification T102 method, which attempts to replicate the degradation process by pre-treatment of the crushed sandstone with three cycles of repeated compaction, would be appropriate. All crushed sandstone select fill should be compacted in maximum 200mm thick loose layers using a large static roller to at least 100% of SMDD.

External concrete pavements should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RMS Specification D&C 3051 unbound base (eg. DGB20) and compacted to a minimum density ratio of 98% of Modified Maximum Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement so as to reduce the potential for material breakdown during compaction.

The subbase layer should be compacted in maximum 200mm thick loose layers using a large static (non-vibratory) smooth drum roller. The sub-base material will provide more uniform slab support and will also reduce 'pumping' of subgrade 'fines' at joints.

Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

For any external AC pavements, we recommend that all base course materials comprise DGB20 in accordance with RMS Specification D&C 3051 unbound base. The DGB20 material should be compacted in maximum 200mm thick loose layers using a large static smooth drum roller to at least 98% of MMDD. Adequate moisture conditioning to within 2% of MOMC should be provided during placement so as to reduce the potential for material breakdown during compaction.

We further recommend that all sub-base materials below any external AC pavements comprise DGS40 in accordance with RMS Specification D&C 3051 unbound base. Recycled materials may be used provided they conform to the specification requirements of DGS40. If the recycled materials contain brick or ceramic fragments, it is highly unlikely that they will conform to the specification requirements. The DGS40 material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 95% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement so as to reduce the potential for material breakdown during compaction.

Density tests should be regularly carried out on the granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be as per the requirements of AS3798-2007. The geotechnical testing authority (GTA) should be directly engaged by the client and not by the earthworks contractor or sub-contractors.

Subsoil drains should be provided along the perimeter of the proposed external pavement, with invert levels of at least 200mm below subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

Where the proposed basement entry ramp/s or external pavements overlie soil, we recommend the soil subgrade be proof rolled with at least six passes of a small sized (preferably at least six tonnes dead-weight) smooth drum roller. The last two passes should be under the direction of a geotechnical engineer. The objective of the proof rolling is to assist in the detection of unstable areas. Based on the investigation results, we do not expect the subgrade to heave, although soft or unstable areas may be present if the earthworks are completed during or following a period of wet weather. However, if subgrade heaving is detected during proof rolling, then the heaving areas should be locally removed down to a stable base and further geotechnical advice should be sought. Further advice and guidance on the treatment of heaving areas, if encountered, will be provided during the proof rolling inspection.

If the external pavements are designed as suspended or if bedrock is exposed, then there would be no need for proof rolling the subgrade.

5.10 **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. In this regard, we note the comment made on page 18 of Section 2.4.2.2 of the Groundwater Management Handbook (September 2008 – First Edition), that *'Generally, the sandstones are fine-to-medium-grained and cemented, making the bulk of the rock mass relatively impermeable.'*

Based on our experience with numerous similar types of projects in the vicinity of the subject site and Sydney wide, we are unware of referrals to the NSW Office of Water being made on the basis of localised groundwater seepage having been encountered in an environment of shallow bedrock.

We expect that seepage volumes, as estimated in Section 5.3.4 above, will decrease once the bulk excavation has drained the local area. Such seepage is expected to be controlled using sump and pump techniques, with the seepage collected within sumps located within the basement which are pumped out periodically. Thus, a 'drained' basement will be feasible. Continuous dewatering during construction will not be required and tanking of the basement over the long term is considered unwarranted for this project.

The proposed excavation will intersect the groundwater seepage paths, though provision for drained retraining walls which will permit groundwater through-flow and 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 in their entirety.

5.11 Construction Monitoring Program

Construction of the proposed development including excavation methods, earthworks and the construction of retaining walls to retain the proposed basement excavation cuts, will require careful sequencing. We therefore recommend that the builder submit their own Construction Monitoring Program (CMP), prior to the commencement of demolition and excavation.

The CMP must incorporate the steps outlined in the 'Geotechnical and Hydrogeological Monitoring Program' (GHMP) tabulated below. The CMP must include, but not be limited to, the proposed demolition and excavation techniques and equipment, the proposed demolition and excavation sequencing, methods for monitoring and assessing retaining wall movements, piling methodology and the various inspection intervals and/or hold points tabulated below. Each activity must be satisfactorily completed before the next one is commenced.

The geotechnical and structural engineers must review and approve the builder's CMP, prior to its implementation.

Further design decisions and discussions with the various parties involved in the project will probably be required during construction and there may be a need for revisions to the builder's CMP and/or program of geotechnical monitoring during construction. The pre-construction meeting referred to in Section 5.2 will help all parties understand the importance of the critical aspects.

The construction works are to be subject to on-going monitoring and review by the structural and geotechnical engineers. The following GHMP, which must be incorporated in the CMP prepared by the builder, is therefore intended to provide an appropriate degree of assurance that the recommended geotechnical design parameters have been reached and to check initial assumptions about subsurface conditions and possible variations that may occur between borehole locations.

GEOTECHNICAL AND HYDROGEOLOGICAL MONITORING PROGRAM			
Item	**Action**	**Action By**	**Date Completed**
1	Detailed dilapidation reports be compiled on the monuments/memorial structures located within McKillop Park to the east, the on-grade AC car park to the east, the adjoining road surfaces and the neighbouring residential properties located on the western and southern sides of Carrington Parade and Evans Street.	JK/SE	
2	Excavation of test pits in the presence of a geotechnical engineer to assess the lateral extent of the contiguous piled wall	BLD/JK	
3	Prepare structural and hydraulic drawings in light of the groundwater, retention, footing and basement slab-on-grade requirements as outlined in this (and any subsequent) geotechnical investigation reports.	SE	
4	Geotechnical review of structural and hydraulic drawings.	JK/SE	
5	Builder to prepare Construction Method Program (CMP).	BLD	
6	Geotechnical and structural review and approval of CMP.	JK/SE	
7	Review of dilapidation reports so that suitable vibration limits can be assessed (refer Section 5.3.1).	JK/SE	
8	Quantitative vibration monitoring during all rock excavation on site when using hydraulic rock hammers (refer Section 5.3.3).	BLD	
9	Builder to arrange for survey monitoring of the basement excavation (refer Section 5.4.3).	BLD	
10	Builder is responsible for coordinating all necessary inspections, and ensuring all approvals are given before proceeding to the next stage of work.	BLD	
11	Geotechnical Engineer to witness the initial stage of drilling shoring piles.	BLD/JK	
12	Builder and Geotechnical Engineer to monitor groundwater seepage (volumes, locations etc) into the excavation.	BLD/JK	
13	Geotechnical Engineer to inspect cut faces between and below, if appropriate, soldier piles at 1.5m depth intervals.	JK	
14	Geotechnical Engineer to inspect the base of pad and strip footing excavations for bearing capacity.	JK	
15	Geotechnical Engineer to carry out a proof rolling inspection of the subgrade for any proposed external pavements.	JK	

NOTES: **JK:** JK Geotechnics **SE:** Structural Engineer **BLD:** Builder

6 GENERAL COMMENTS

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

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

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

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. Copyright in this report is the property of JK Geotechnics. If there is any change in the proposed development described in this report then all recommendations should be reviewed. 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.

Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A
MOISTURE CONTENT TEST REPORT

Client: JK Geotechnics
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW

Ref No: 25077ZH2
Report: A
Report Date: 26/09/2012
Page 1 of 1

AS 1289 **TEST METHOD** **2.1.1**

BOREHOLE NUMBER	DEPTH	MOISTURE CONTENT
	m	%
101	0.70-0.90	2.8
103	0.80-1.00	2.4
104	4.00-4.40	8.4
104	5.10-5.40	5.7

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TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client: JK Geotechnics
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW

Ref No: 25077ZH2
Report: B
Report Date: 26/09/2012
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BOREHOLE NUMBER	104
DEPTH (m)	0.80 - 1.50
Surcharge (kg)	9.0
Maximum Dry Density (t/m^3)	1.75 STD
Optimum Moisture Content (%)	14.3
Moulded Dry Density (t/m^3)	1.72
Sample Density Ratio (%)	98
Sample Moisture Ratio (%)	99
Moisture Contents	
Insitu (%)	16.5
Moulded (%)	14.2
After soaking and After Test, Top 30mm(%)	30.4
Remaining Depth (%)	20.3
Material Retained on 19mm Sieve (%)	0
Swell (%)	7.5
C.B.R. value: @5.0mm penetration	1.0

NOTES:

- Refer to appropriate notes for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample: 17/09/2012

NATA
NATA Accredited Laboratory
Number:1327

Accredited for compliance with ISO/IEC 17025.
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Authorised Signature / Date
(A. Tatikonda) 26/9/12

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TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Redevelopment
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Ref No: 25077ZH
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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	1.36-1.39	1.0	20
	1.73-1.76	2.1	42
	2.00-2.04	1.3	26
	2.61-2.65	5.5	110
	3.49-3.54	1.3	26
	3.96-4.00	0.09	2
	4.13-4.17	2.9	58
	4.87-4.91	2.7	54
	5.22-5.25	1.1	22
	5.77-5.80	1.2	24
	6.15-6.19	1.7	34
	6.84-6.87	1.7	34
	7.16-7.19	2.2	44
	7.71-7.75	1.2	24
	8.35-8.38	0.6	12
	9.00-9.02	0.3	6
	9.31-9.34	0.1	2
	10.51-10.56	1.0	20
	10.86-10.90	1.6	32
	11.26-11.29	0.5	10
	11.73-11.76	0.9	18
	12.17-12.20	0.5	10
	12.45-12.49	2.7	54
	12.82-12.85	1.3	26
102	4.87-4.90	0.09	2
	5.11-5.15	0.1	2
	5.69-5.72	0.4	8
	6.18-6.20	0.05	1
	6.49-6.51	0.04	1
	7.16-7.20	0.06	1
	7.35-7.39	0.2	4

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
102	7.74-7.77	1.3	26
	8.24-8.27	1.1	22
	8.69-8.72	0.8	16
	9.33-9.36	1.2	24
	9.80-9.83	0.7	14
	10.20-10.24	1.8	36
	10.52-10.56	0.4	8
	11.06-11.09	0.4	8
103	1.34-1.38	1.0	20
	1.70-1.74	1.4	28
	2.10-2.14	0.7	14
	2.89-2.92	0.6	12
	3.25-3.30	0.5	10
	4.33-4.37	0.4	8
	4.86-4.90	0.3	6
	5.25-5.28	0.3	6
	5.84-5.87	0.09	2
	6.25-6.27	0.2	4
	6.73-6.76	0.1	2
	7.22-7.26	0.2	4
	7.89-7.91	0.06	1
	8.67-8.70	0.7	14
	9.20-9.24	2.4	48
	9.84-9.88	0.4	8
	10.26-10.29	1.6	32
	10.84-10.86	1.2	24
	11.41-11.43	1.6	32
	11.97-12.00	1.0	20
	12.38-12.40	2.2	44
	12.94-12.96	1.7	34
	13.44-13.47	0.5	10

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
103	13.94-13.98	0.9	18
	14.41-14.44	0.9	18
	14.97-15.00	1.5	30
104	5.76-5.80	0.6	12
	6.11-6.16	0.5	10
	6.46-6.50	0.5	10
	7.00-7.03	0.8	16
	7.55-7.58	0.7	14
	8.00-8.02	1.0	20
	8.49-8.53	1.0	20
	9.00-9.03	0.9	18
	9.43-9.46	1.0	20
105	3.21-3.24	0.1	2
	3.84-3.87	0.3	6
	4.27-4.30	0.1	2
	4.75-4.79	0.1	2
	5.76-5.79	0.07	1
	6.46-6.49	0.05	1
	6.87-6.90	0.05	1
	7.40-7.43	1.0	20
	7.83-7.86	0.5	10
	8.68-8.72	0.05	1
	9.16-9.20	0.1	2
	9.64-9.68	0.3	6
	10.44-10.47	1.2	24
	10.97-11.00	1.1	22
	11.20-11.23	0.7	14
	11.79-11.83	0.1	2
	12.37-12.41	0.6	12
	12.77-12.80	0.9	18
	13.32-13.35	0.7	14

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
105	13.76-13.78	0.7	14
	14.31-14.34	0.8	16
	14.79-14.82	0.6	12
	15.32-15.35	1.0	20
201	0.50-0.54	0.4	8
	0.79-0.83	0.3	6
	1.30-1.33	0.5	10
	1.84-1.86	0.6	12
	2.24-2.28	0.5	10
	2.62-2.65	0.3	6
	3.15-3.19	0.03	1
	3.70-3.75	0.2	4
	4.36-4.40	0.5	10
	4.70-4.73	0.2	4
	5.23-5.26	0.3	6
	6.33-6.36	1.0	20
	6.66-6.69	0.2	4
	6.88-6.91	0.5	10
	7.10-7.14	0.1	2
	7.80-7.84	0.2	4
	8.18-8.22	1.0	20
	8.71-8.75	4.8	96
	9.28-9.32	0.7	14
	9.75-9.79	1.4	28
	10.20-10.24	1.7	34
	10.84-10.88	2.6	52
	11.30-11.34	1.3	26
	11.80-11.83	0.9	18
	12.11-12.13	1.3	26
	12.61-12.64	2.1	42
	13.30-13.33	0.6	12

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BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
201	13.66-13.69	0.6	12
	14.30-14.33	1.1	22
	14.85-14.88	0.5	10
202	1.13-1.16	1.5	30
	1.75-1.79	0.9	18
	2.38-2.42	0.7	14
	2.81-2.84	1.9	38
	3.21-3.24	1.7	34
	3.75-3.78	1.0	20
	4.17-4.21	1.5	30
	4.70-4.74	1.8	36
	5.33-5.37	0.9	18
	5.70-5.74	0.5	10
	6.26-6.30	0.3	6
	6.76-6.79	0.3	6
	7.30-7.34	0.3	6
	8.10-8.14	0.1	2
	8.81-8.84	0.05	1
	9.36-9.39	0.4	8
	9.82-9.85	1.0	20
	10.32-10.35	0.9	18
	10.77-10.80	5.4	108
	11.17-11.21	1.3	26
	11.80-11.83	2.3	46
	12.24-12.28	1.4	28
	12.75-12.79	1.4	28
	13.22-13.25	0.9	18
	13.70-13.73	1.2	24
	14.25-14.29	1.5	30
	14.87-14.90	1.4	28
	15.25-15.29	0.8	16

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BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
202	15.75-15.79	1.1	22
	16.15-16.18	1.4	28
	16.79-16.81	1.7	34
	17.16-17.19	0.6	12
	17.67-17.70	1.0	20
	18.12-18.15	0.3	6
	18.77-18.80	1.4	28
	18.80-18.82	0.5	10
203	0.96-1.01	0.5	10
	1.26-1.29	0.3	6
	1.56-1.59	0.4	8
	2.82-2.86	0.4	8
	3.26-3.30	0.2	4
	3.88-3.92	0.9	18
	4.40-4.43	0.1	2
	4.60-4.62	0.2	4
	5.20-5.23	0.2	4
	5.75-5.78	0.5	10
	6.22-6.25	0.9	18
	6.80-6.83	0.4	8
	7.36-7.40	1.0	20
203A	4.24-4.29	0.2	4
	4.86-4.90	0.07	1
	5.30-5.34	0.05	1
	5.75-5.78	0.4	8
	6.20-6.24	0.9	18
	6.75-6.78	0.3	6
	7.18-7.21	0.6	12
	7.86-7.89	0.7	14
	8.25-8.28	0.7	14
	8.62-8.65	2.3	46

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203A	9.57-9.61	0.8	16
	10.25-10.28	2.4	48
	10.69-10.73	0.9	18
	11.45-11.50	1.7	34
	12.21-12.25	1.1	22
	12.64-12.67	0.6	12
	13.19-13.21	0.2	4
	13.78-13.82	0.5	10
	14.11-14.14	1.0	20
	14.72-14.75	1.4	28
204	4.27-4.31	0.2	4
	4.42-4.45	0.2	4
	4.59-4.63	0.2	4
	5.20-5.23	0.4	8
	5.75-5.78	0.7	14
	6.17-6.20	0.4	8
	6.80-6.83	0.7	14
	7.17-7.20	0.9	18
	7.65-7.68	0.7	14
	8.22-8.24	1.0	20
	8.78-8.82	0.9	18
	9.17-9.20	1.3	26
	9.66-9.69	1.3	26
	10.16-10.19	0.6	12
	10.45-10.48	0.6	12
	10.83-10.86	1.5	30
	11.06-11.08	0.2	4
	11.42-11.45	0.4	8
	11.83-11.86	0.4	8
	12.64-12.68	0.7	14
	13.00-13.04	1.4	28

NOTES: See Page 9 of 9

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	25077ZH
Project:	Proposed Redevelopment	**Report:**	C
Location:	80 Evans Street, Freshwater, NSW	**Report Date:**	12/03/2015
		Page 8 of 9	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
204	13.35-13.39	1.7	34
205	0.30-0.33	0.6	12
	0.61-0.64	0.3	6
	1.49-1.52	0.8	16
	2.00-2.04	0.5	10
	3.25-3.29	1.0	20
	3.74-3.78	1.1	22
	4.05-4.09	0.9	18
	5.15-5.19	1.4	28
	5.67-5.70	0.4	8
	6.31-6.35	0.2	4
	6.75-6.78	0.1	2
	7.13-7.16	0.09	2
	7.28-7.31	0.2	4
	7.59-7.63	0.4	8
	8.00-8.03	0.6	12
	8.56-8.60	0.8	16
	8.95-8.99	0.4	8
	9.14-9.18	0.6	12
	9.86-9.90	0.7	14
206	2.25-2.28	1.4	28
	2.79-2.82	0.8	16
	3.21-3.24	2.2	44
	3.74-3.77	1.6	32
	4.22-4.25	1.9	38
	4.72-4.75	3.1	62
	5.22-5.24	2.8	56
	5.72-5.75	3.4	68
	6.21-6.24	2.7	54
	6.76-6.78	2.5	50

NOTES: See Page 9 of 9

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: JK Geotechnics
Project: Proposed Redevelopment
Location: 80 Evans Street, Freshwater, NSW
Ref No: 25077ZH
Report: C
Report Date: 12/03/2015
Page 9 of 9

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
206	7.43-7.46	2.8	56
	8.18-8.22	3.0	60
	8.70-8.73	2.1	42
	9.40-9.43	1.8	36
	9.86-9.90	0.5	10
	10.17-10.21	1.0	20
	10.80-10.84	1.2	24
	11.67-11.70	2.3	46
	12.16-12.19	0.3	6
	12.50-12.53	0.6	12
	13.30-13.33	0.5	10
	13.80-13.82	0.7	14
	14.46-14.50	1.1	22
	15.25-15.29	1.6	32
	15.71-15.73	0.7	14
	16.60-16.63	0.5	10
	17.21-17.24	1.1	22
	18.19-18.22	0.6	12
	18.70-18.73	0.7	14
	19.29-19.32	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: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. 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 :

$$\text{U.C.S.} = 20\, I_{S(50)}$$

TABLE D
SUMMARY OF LABORATORY RESULTS
SOIL CHEMISTRY - pH, SULPHATE & CHLORIDES

Borehole Number	Sample Depth (m)	Sample Description	pH Units	Sulphate (mg/kg)	Chloride (mg/kg)
BH102	0.50 - 0.95	Fill: Silty sand	8.8	8.0	6.0
BH102	1.50 - 1.95	Fill: Silty sand	8.1	240.0	16.0
BH104	1.50 - 1.95	Residual Silty Clay	5.0	97.0	210.0

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
101
1/3

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB

Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 22.0m

Date: 11-9-12
Datum: AHD

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

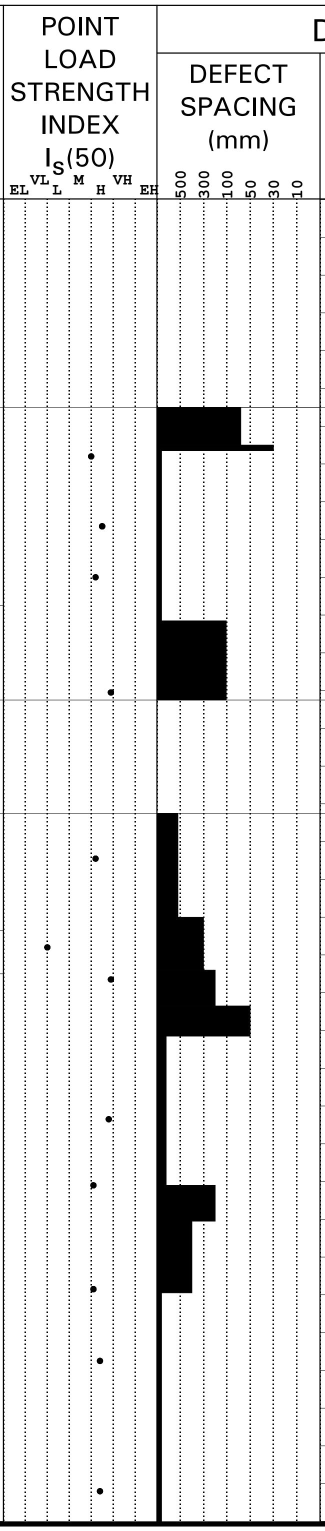
Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING	■					0			FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained ironstone and sandstone gravel and root fibres.	M			GRASS COVER
				■	SPT 4/100mm REFUSAL			SC	CLAYEY SAND: fine to medium grained, orange brown, trace of fine grained ironstone gravel and root fibres.	M	MD		RESIDUAL
	■			■				-		DW	L		LOW 'TC' BIT RESISTANCE
						1			SANDSTONE: fine to medium grained, orange brown and light grey.		H		MODERATE RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						2							
						3							
						4							
						5							
						6							
						7							

CORED BOREHOLE LOG

Borehole No.
101

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2	**Core Size:** NMLC	**R.L. Surface:** ≈ 22.0m
Date: 11-9-12	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: JK300	**Bearing:** -	**Logged/Checked by:** J.D./A.J.H.



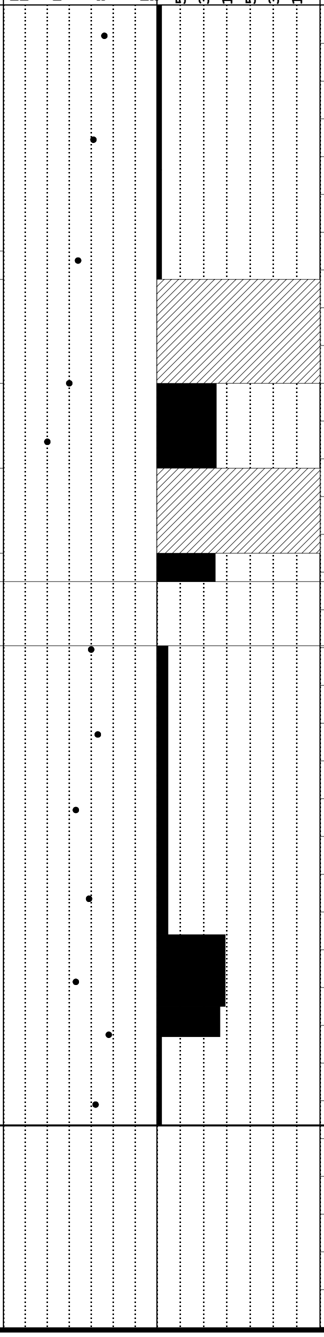
Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0						
		1		START CORING AT 1.10m				
				SANDSTONE: fine to medium grained, light orange brown, with light grey and red brown bands, bedded at 0-10°.	DW	H	- J, 70-90°, Un, R, IS - Be, 0°, P, S, IS - J, 90°, P, S, IS	
		2					-	
						VH	- J, 75°, P, R, IS - - J, 75°, P, S, IS - J, 55°, P, S, IS - 2xJ, 65°, P, S, IS	
				CORE LOSS 0.60m				
		3						
▼ ON COMPLETION OF CORING				SANDSTONE: fine to medium grained, light brown, with red brown and light grey bands, bedded at 0-20°.	DW	H	- J, 45°, P, R	
		4				VL		
						H	- HEALED J, 85° - Be, 10°, P, R, IS - Be, 0°, P, R, IS - Be, 0°, P, S, IS - HEALED J, 80°, P	
		5		as above, but fine to coarse grained.			- Be, 0-20°, Un, R, IS - Be, 5°, P, R, IS - Be, 0°, P, S, IS	
		6						
		7						

CORED BOREHOLE LOG

Borehole No.
101
3/3

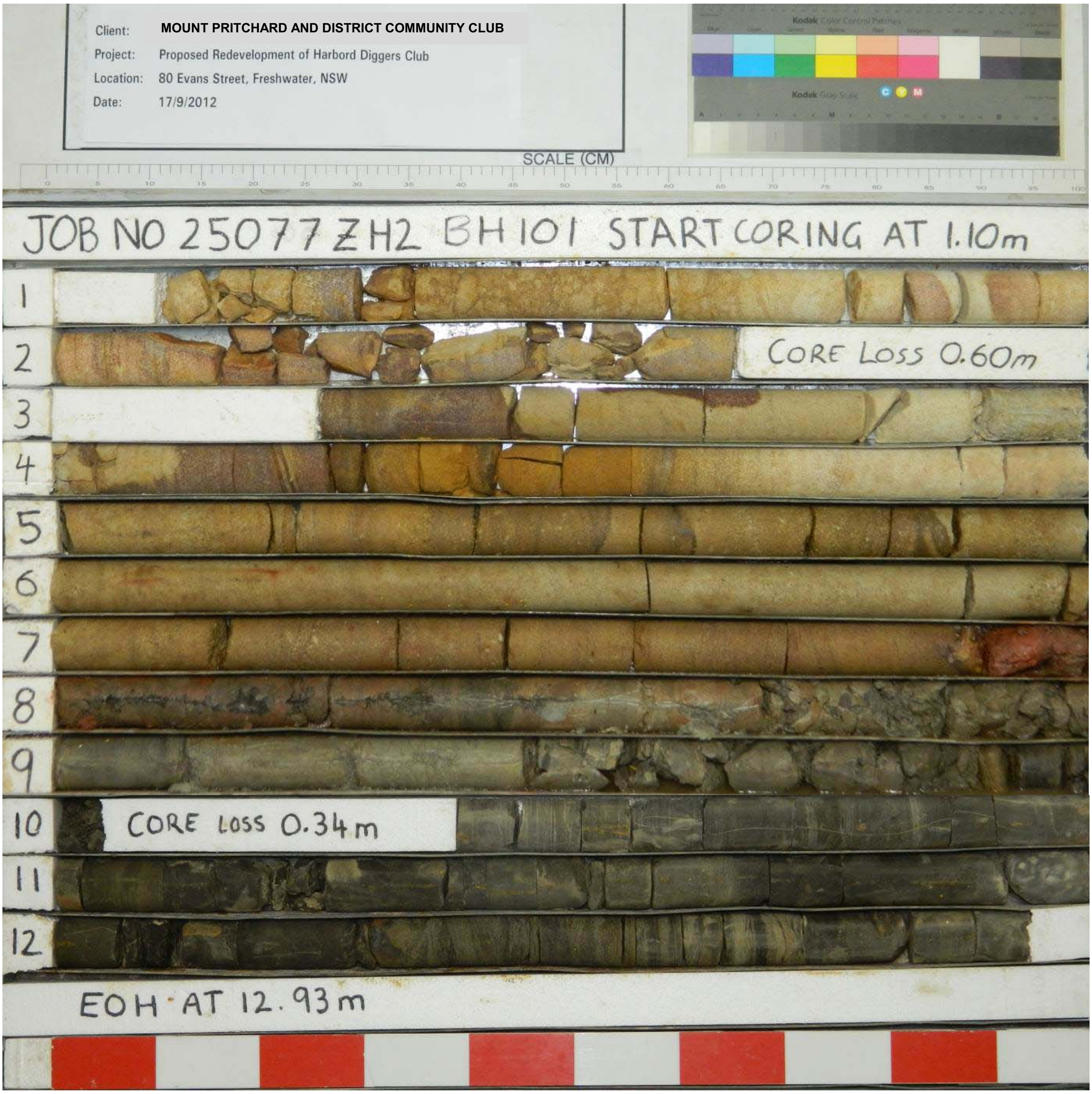
Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 22.0m
Date: 11-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.D./A.J.H.



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RET-URN				SANDSTONE: fine to coarse grained, light brown and red brown, bedded at 0-20°.	DW	H		
		8		as above, but dark grey and red brown.				
						L-M		
					XW	EL		
		9		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-5°.	DW	VL-L	- XWS, 0°, 30mm.t - XWS, 0°, 20mm.t	
					XW	EL	- J, 70-90°, Un, S, CLAY COATED - J, 50°, P, S, CLAY INFILL	
					DW	M	- J, 80-90°, Un, R, CLAY INFILL	
		10		CORE LOSS 0.34m				
				SHALE: dark grey, with fine grained light grey sandstone laminae, bedded at 0-10°.	SW-FR	M-H	- PROPOSED BASEMENT 2 FFL, RL 11.4m	
		11					- J, 70°, P, S	
		12					- J, 50°, P, S, - 3xBe, 0°, P, S, - Be, 0-20°, Un, R, IS - 2xBe, 0°, P, S, IS	
		13		END OF BOREHOLE AT 12.93m				
		14						

Ref: 25077ZH2 Borehole 101



Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW
Date: 17/9/2012

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
102
1/3

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 19.5m
Date: 11-9-12
Datum: AHD
Logged/Checked by: J.D./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
	■					0	▩		FILL: Silty sand topsoil, fine to medium grained, dark brown, with roots.	M			GRASS COVER
				■	N = 18 5,9,9	1	▩		FILL: Silty sand, fine to medium grained, light brown, trace of fine to medium grained ironstone and sandstone gravel.				APPEARS WELL COMPACTED
	■			■	N = 22 11,9,13		▩		as above, but light brown and grey, trace of clay fines.				
				■		2	▩		FILL: Silty sand, fine to medium grained, light grey and dark grey.				
ON COMPLETION OF CORING ▼	■			■	N = 9 4,2,7	3	▨	SC	CLAYEY SAND: fine to medium grained, light grey and orange brown, trace of fine grained ironstone gravel.	M	L		RESIDUAL
▼ ON COMPLETION OF AUGERING						4	⁞	-	SANDSTONE: fine to medium grained, light grey and orange brown.	DW	L-M		LOW TO MODERATE 'TC' BIT RESISTANCE
						5 6 7			REFER TO CORED BOREHOLE LOG				

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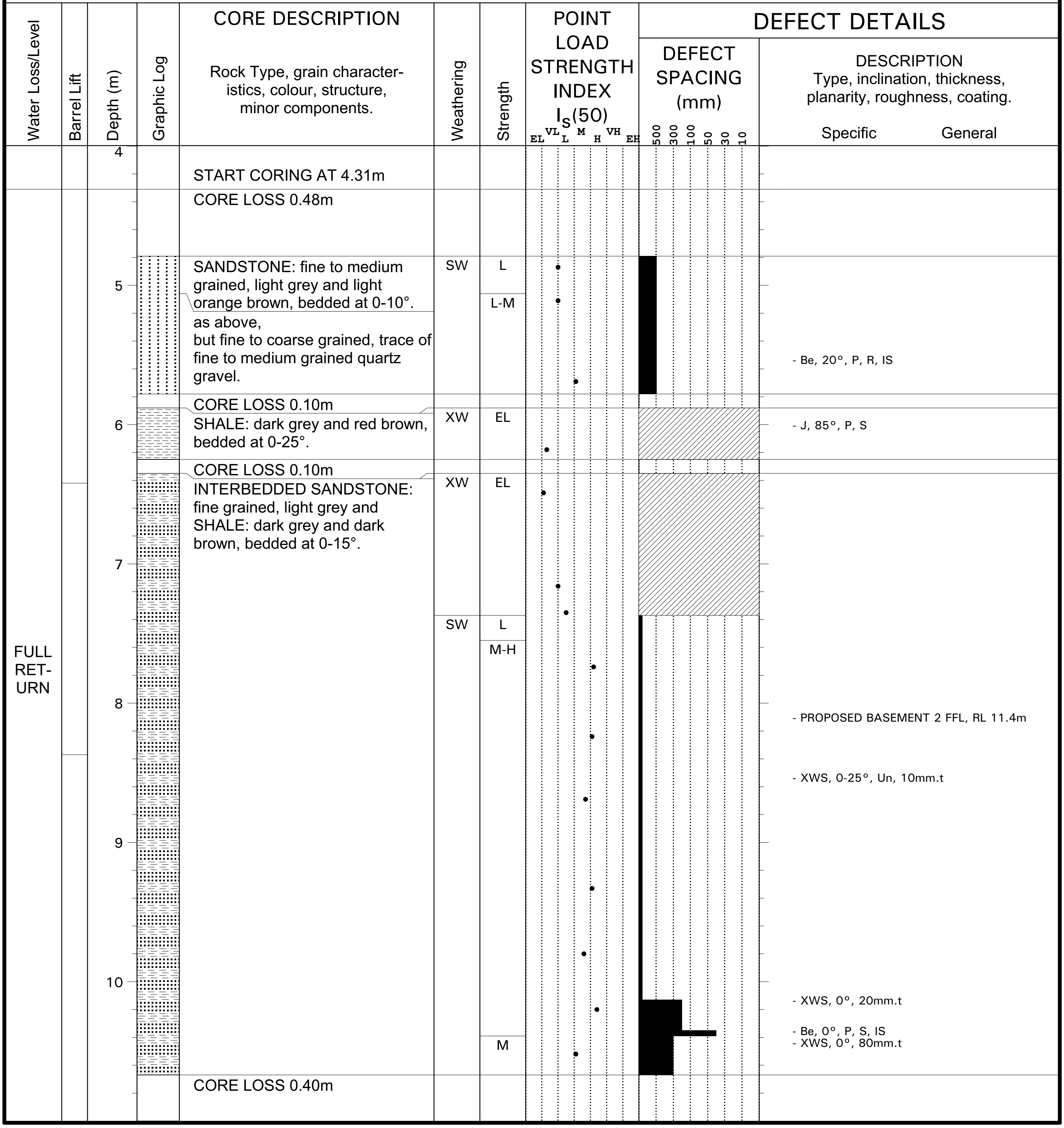
CORED BOREHOLE LOG

Borehole No.
102
2/3

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Date: 11-9-12
Drill Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 19.5m
Datum: AHD
Logged/Checked by: J.D./A.J.H.



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 $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		4		START CORING AT 4.31m						
FULL RET-URN				CORE LOSS 0.48m						
		5		SANDSTONE: fine to medium grained, light grey and light orange brown, bedded at 0-10°.	SW	L L-M				
				as above, but fine to coarse grained, trace of fine to medium grained quartz gravel.					- Be, 20°, P, R, IS	
				CORE LOSS 0.10m						
		6		SHALE: dark grey and red brown, bedded at 0-25°.	XW	EL			- J, 85°, P, S	
				CORE LOSS 0.10m						
		7 8 9 10		INTERBEDDED SANDSTONE: fine grained, light grey and SHALE: dark grey and dark brown, bedded at 0-15°.	XW SW	EL L M-H M			- PROPOSED BASEMENT 2 FFL, RL 11.4m - XWS, 0-25°, Un, 10mm.t - XWS, 0°, 20mm.t - Be, 0°, P, S, IS - XWS, 0°, 80mm.t	
				CORE LOSS 0.40m						

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
102
3/3

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB

Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 19.5m

Date: 11-9-12
Inclination: VERTICAL
Datum: AHD

Drill Type: JK300
Bearing: -
Logged/Checked by: J.D./A.J.H.

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 DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
				SANDSTONE: fine grained, light brown.	DW	M			- CS, 0°, 10mm.t
				END OF BOREHOLE AT 11.20m					
		12							
		13							
		14							
		15							
		16							
		17							

Ref: 25077ZH2 Borehole 102

JK Geotechnics

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW
Date: 17/9/2012

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
103
1/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Date: 14-9-12
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 24.1m
Datum: AHD
Logged/Checked by: J.D./A.J.H.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING	■			■		0	▨	-	ASPHALTIC CONCRETE: 25mm.t FILL: Silty sand, fine to medium grained, light brown and dark brown, trace of fine to medium grained sandstone and ironstone gravel.	M			
	■			■		1	⁝	-	SANDSTONE: fine to medium grained, light grey and orange brown.	DW	VL		VERY LOW 'TC' BIT RESISTANCE
						2 3 4 5 6 7			REFER TO CORED BOREHOLE LOG				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

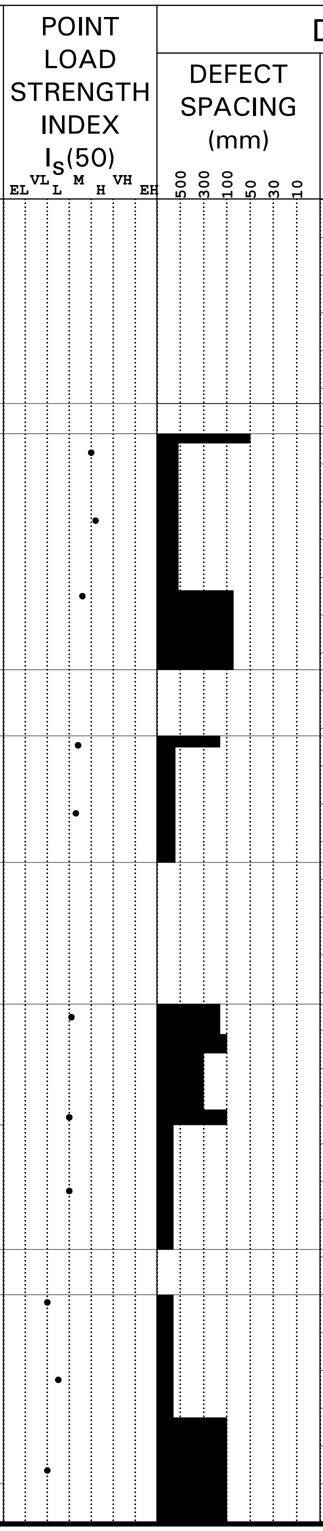
CORED BOREHOLE LOG

Borehole No.
103
2/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 24.1m
Date: 14-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.D./A.J.H.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
	0				
	1	START CORING AT 1.08m			
		CORE LOSS 0.16m			
	2	SANDSTONE: fine to medium grained, orange brown, with light grey bands, bedded at 0-15°.	DW	M-H	- 4xBe, 0-15°, Un, S, IS - Be, 10°, P, R, IS - 2x J, 35°, P, S, IS - 2x J, 35°& 45°, P, S, IS
		CORE LOSS 0.35m			
▼ ON COMPLETION OF CORING	3	SANDSTONE: fine to medium grained, light grey and red brown, bedded at 15°, with occasional cross beds up to 30°.	DW	M	- Be, 15°, P, S, IS
	4	CORE LOSS 0.75m			
		SANDSTONE: fine to medium grained, red brown, with light grey and orange brown bands, bedded at 0-20°.	DW	L-M	- J, 80°, P, R, IS - J, 60°, P, R, IS - J, 50°, P, R, IS - Be, 0-5°, Un, R, IS
	5	as above, but fine to coarse grained, orange brown, bedded at 0-10°.		L	
		CORE LOSS 0.24m			
	6	SANDSTONE: fine to coarse grained, light grey, bedded at 0-20°.	SW	L	- XWS, 0°, 30mm.t - Be, 20° P, S, IS - Be, 0-15°, Un, R, IS
	7	as above, but with orange brown and red brown seams.		VL-L	- XWS, 0°, 40mm.t - XWS, 0°, 50mm.t - J, 50°, P, R, CLAY INFILL



CORED BOREHOLE LOG

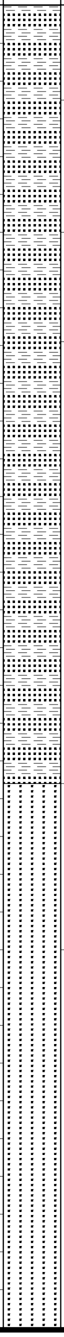
Borehole No.
103
3/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

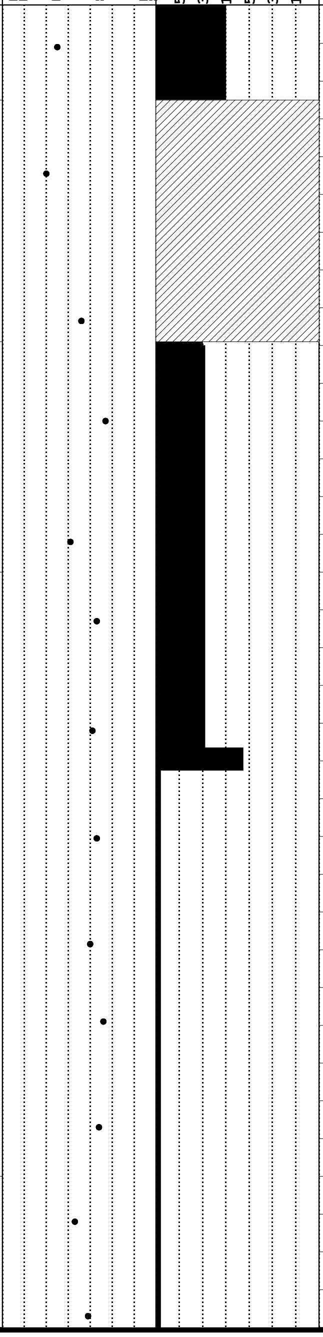
Job No. 25077ZH2
Date: 14-9-12
Drill Type: JK300
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 24.1m
Datum: AHD
Logged/Checked by: J.D./A.J.H.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating.
		INTERBEDDED SANDSTONE: fine grained, light grey, and SHALE: dark grey, bedded at 0-10°.	DW	VL-L	- XWS, 0°, 10mm.t - XWS, 0°, 10m.t - XWS, 0°, 10mm.t
FULL RETURN	8	as above, but with very low strength seams.	XW	EL	- J, 50°, P, R, CLAY INFILL
		as above, but with occasional DW, M strength seams (up to 80mm.t).			- 2x J, IS, 50°&75°, P, R, CLAY INFILL
	9	INTERBEDDED SANDSTONE: fine to medium grained, light grey, and SHALE: dark grey, bedded at 0-15°	SW	M-H	- Be, 0-10°, P, R - XWS, 0°, 50mm.t - Be, 10°, P, R - J, 55°, P, R - J, 60°, P, S
	10			H	- XWS, 0°, 10mm.t - J, 45-90°, Un, R, CLAY INFILL - J, 45-90°, Un, R, CLAY INFILL
	11				- XWS, 0°, 100mm.t - 9x Be, 0-5°, P, S, IS
		SANDSTONE: fine to medium grained, light grey, with dark grey shale laminae, bedded at 0-20°.			
	12	as above, but bedded at 0-15°			- Be, 0°, P, R, CLAY INFILL 10mm.t
			SW-FR		- PROPOSED BASEMENT 2 FFL, RL 11.4m
	13			M-H	- J, 75°, P, R
	14				

Graphic Log



POINT LOAD STRENGTH INDEX $I_s(50)$ / DEFECT SPACING (mm)



CORED BOREHOLE LOG

Borehole No.
103
4/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED

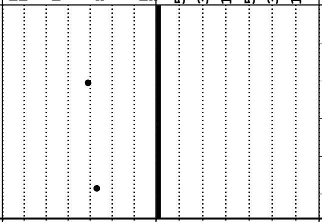
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB

Location: 80 EVANS STREET, FRESHWATER, NSW

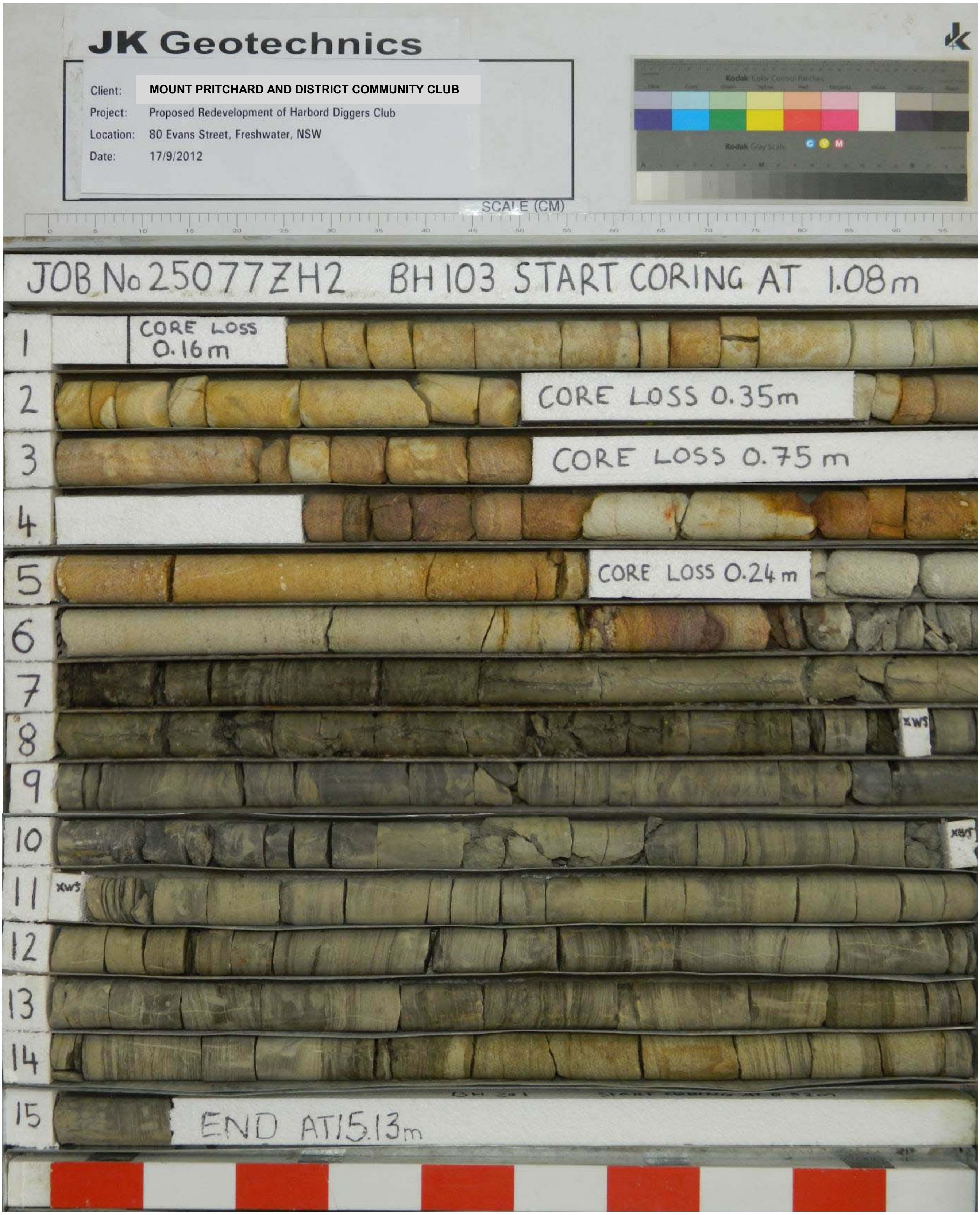
Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 24.1m

Date: 14-9-12
Inclination: VERTICAL
Datum: AHD

Drill Type: JK300
Bearing: -
Logged/Checked by: J.D./A.J.H.

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 DETAILS: DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS: DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		15		SANDSTONE: fine to medium grained, light grey, with dark grey shale laminae, bedded at 0-10°.	SW-FR	M-H			- XWS, 0°, 90mm.t - Be, 5°, P, R, IS - Be, 0° P, R, IS	
		16 17 18 19 20		END OF BOREHOLE AT 15.13m						

Ref: 25077ZH2 Borehole 103



JK Geotechnics

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW
Date: 17/9/2012

BOREHOLE LOG

Borehole No.
104
1/2

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Date: 12-9-12
Method: SPIRAL AUGER
JK500
R.L. Surface: ≈ 18.0m
Datum: AHD
Logged/Checked by: J.D./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 OF AUGERING						0			FILL: Silty sand, fine to medium grained, dark brown and grey, trace of clay, roots fibres, fine to medium grained sandstone and ironstone gravel.	M			GRASS COVER
					N = 15 5,6,9			CH	SILTY CLAY: high plasticity, light grey, with occasional ironstone bands, trace of root fibres.	MC≈PL	H	>600	RESIDUAL
						1							
					N = 23 5,10,13								
						2							
▼ ON COMPLETION OF CORING									as above, but light grey and dark grey, trace of fine to medium grained sand.	MC<PL			
					SPT 15/150mm REFUSAL	3							
								-	INTERBEDDED SHALE: dark grey and SANDSTONE: fine to medium grained, light grey, with occasional iron indurated bands.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
						4							
					N = 28 3,10,18								
						5		-	SANDSTONE: fine to medium grained, orange brown.	DW	M		LOW RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						6							
						7							

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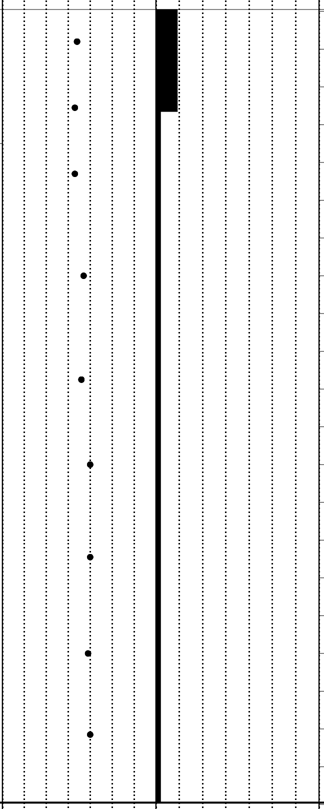
CORED BOREHOLE LOG

Borehole No.
104
2/2

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 18.0m
Date: 12-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK300
Bearing: -
Logged/Checked by: J.D./

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		START CORING AT 5.59m						
FULL RET-URN		6		SANDSTONE: fine to medium grained, orange brown, bedded at 0-15°.	DW	M			- Be, 15°, P, S, IS	
		7 8		as above, but fine grained, light grey, with dark grey shale laminae, bedded at 0-10°.	FR	M-H			- PROPOSED BASEMENT 2 FFL, RL 11.4m - CS, 5°, 15mm.t	
		9		SHALE: dark grey, with fine grained sandstone laminae and seams, bedded at 0-5°.						
		10 11		END OF BOREHOLE AT 9.79m						



Ref: 25077ZH2 Borehole 104

JK Geotechnics

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW
Date: 17/9/2012

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
105
1/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Date: 12-9-12
Method: SPIRAL AUGER
JK500
R.L. Surface: ≈ 23.5m
Datum: AHD
Logged/Checked by: J.D./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 COMPLETION OF AUGERING						0			FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained igneous and ironstone gravel, concrete, ceramic and glass fragments and root fibres.	M			GRASS COVER APPEARS POORLY COMPACTED
	■			■	N = 6 5,3,3								
	■					1							
ON COMPLETION OF CORING				■	SPT 4/150mm REFUSAL								
	■							-	SANDSTONE: fine to medium grained, light grey and light brown.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
▼						2				DW	VL-L		LOW RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						3							
						4							
						5							
						6							
						7							

CORED BOREHOLE LOG

Borehole No.
105
2/4

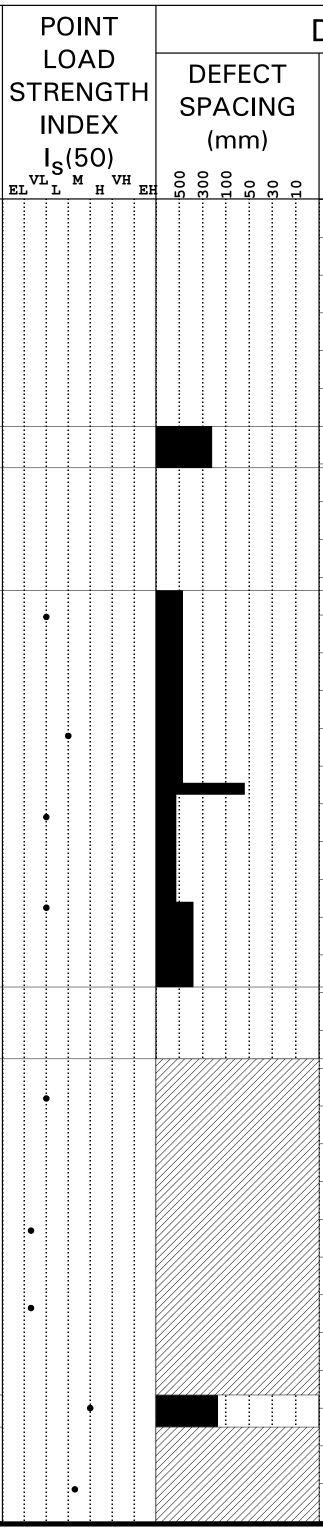
Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED

Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB

Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 23.5m
Date: 12-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK500
Bearing: -
Logged/Checked by: J.D./A.J.H.

Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
1				
2	START CORING AT 2.20m			
	SANDSTONE: fine to medium grained, light brown and orange brown, bedded at 0-10°.	DW	VL-L	
	CORE LOSS 0.65m			
3	SANDSTONE: fine to coarse grained, light grey and orange brown, bedded at 0-5°.	DW	VL-L	- XWS, 0°, 80mm.t
	as above but light grey, bedded at 0-20°.			- XWS, 0°, 70mm.t
4				- XWS, 10°, 70mm.t - XWS, 0°, 50mm.t
				- Be, 20°, P, S
5				- XWS, 0°, 50mm.t
	CORE LOSS 0.38m			
	SHALE: dark grey, with fine grained sandstone laminae and seams, bedded at 0-20°.	XW	EL	
6				
7		DW	M	
		XW	EL	- J, 60°, P
8				- 2x J, 50°, P, S



CORED BOREHOLE LOG

Borehole No.
105
3/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 23.5m
Date: 12-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK500
Bearing: -
Logged/Checked by: J.D./A.J.H.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		SHALE: dark grey, with fine grained sandstone laminae, bedded at 0-10°.	XW	EL		
		CORE LOSS 0.30m				
FULL RET-URN		SHALE: dark grey, with fine grained sandstone laminae, bedded at 0-10°.	XW	EL	- J, 50°, P, S, IS	
	9		DW	VL-L	- CS, 0°, 20mm.t - CS, 0°, 10mm.t	
			XW	EL	- 2x J, 45°, P, S, IS - J, 60-90°, Un, R, IS	
	10		DW	VL	- Be, 0-15°, Un, R, IS - J, 45°, P, S - Be, 5°, P, R, CLAY INFILL - XWS, 0°, 40mm.t	
			SW	M-H		
	11	INTERBEDDED SANDSTONE: fine grained, light grey and SHALE: dark grey.			- CS, 0°, 10mm.t	
	12	SANDSTONE: fine to medium grained, light grey, with dark grey shale laminae, bedded at 0-10°.	DW	VL	- PROPOSED BASEMENT 2 FFL, RL 11.4m	
			SW	M-H	- XWS, 0°, 75mm.t	
	13		FR			
	14	as above, but bedded at 0-25°.				

Barrel Lift | Graphic Log

POINT LOAD STRENGTH INDEX $I_s(50)$: EL VL L M H VH EH

DEFECT SPACING (mm): 500 300 100 50 30 10

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

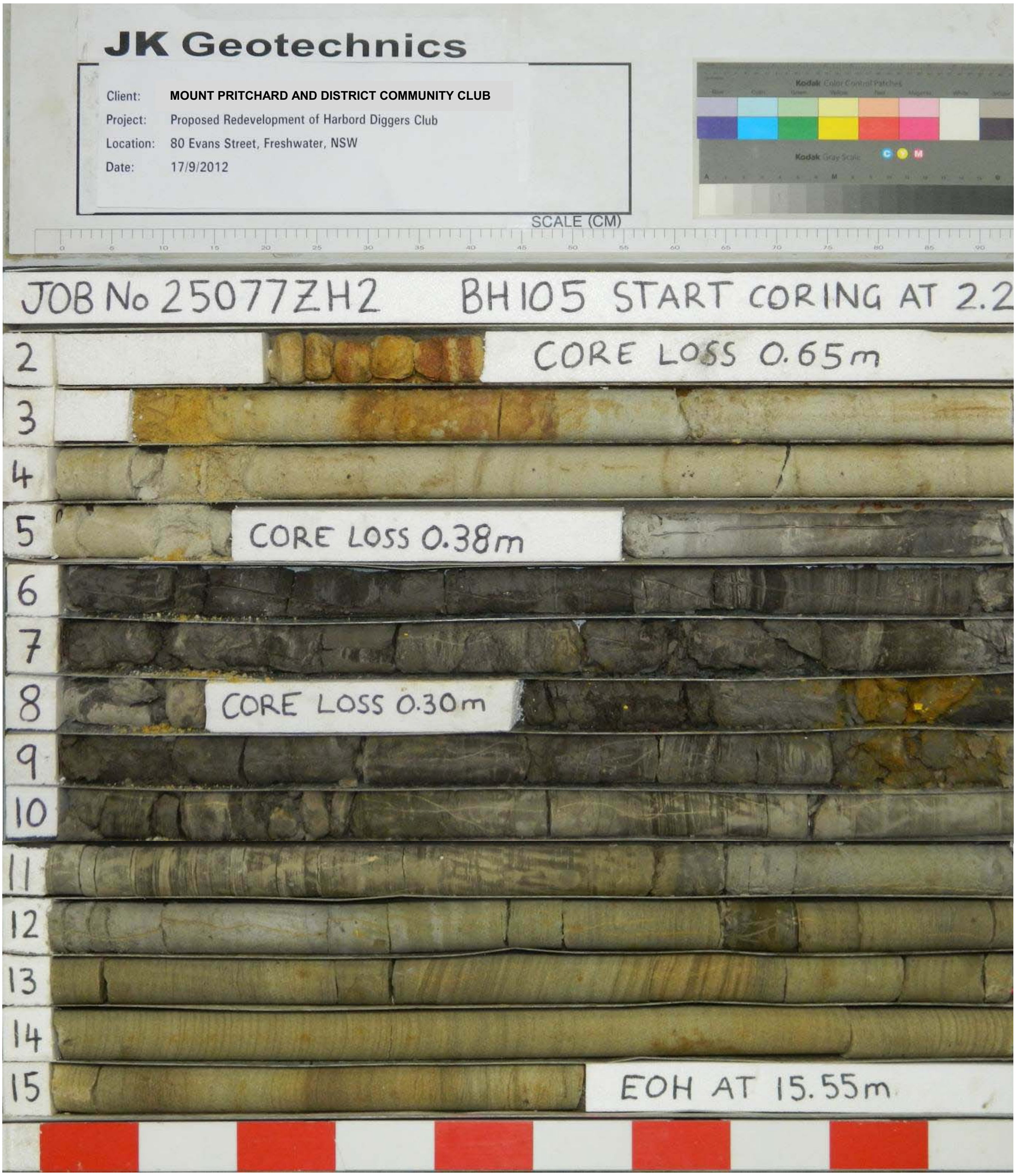
CORED BOREHOLE LOG

Borehole No.
105
4/4

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB LIMITED
Project: PROPOSED REDEVELOPMENT OF HARBORD DIGGERS CLUB
Location: 80 EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH2
Core Size: NMLC
R.L. Surface: ≈ 23.5m
Date: 12-9-12
Inclination: VERTICAL
Datum: AHD
Drill Type: JK500
Bearing: -
Logged/Checked by: J.D./A.J.H.

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
				SANDSTONE: fine to medium grained, ligth grey, with dark grey laminae, bedded at 0-25°.	FR	M-H				
				END OF BOREHOLE AT 15.55m						
		16								
		17								
		18								
		19								
		20								
		21								



JK Geotechnics

Client: MOUNT PRITCHARD AND DISTRICT COMMUNITY CLUB
Project: Proposed Redevelopment of Harbord Diggers Club
Location: 80 Evans Street, Freshwater, NSW
Date: 17/9/2012

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
201
1/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 19-2-15
Method: SPIRAL AUGER
JK305
R.L. Surface: ≈ 20.7m
Datum: AHD
Logged/Checked by: D.A.F./

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING						0			FILL: Silty sand, fine to medium grained, brown and light grey.	M			APPEARS POORLY COMPACTED
								SP	SAND: fine to medium grained, light orange brown.	M			RESIDUAL
								-	SANDSTONE: fine to medium grained, orange brown and light grey.	DW	M		MODERATE TO HIGH 'TC' BIT RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						1							
						2							
						3							
						4							
						5							
						6							
						7							

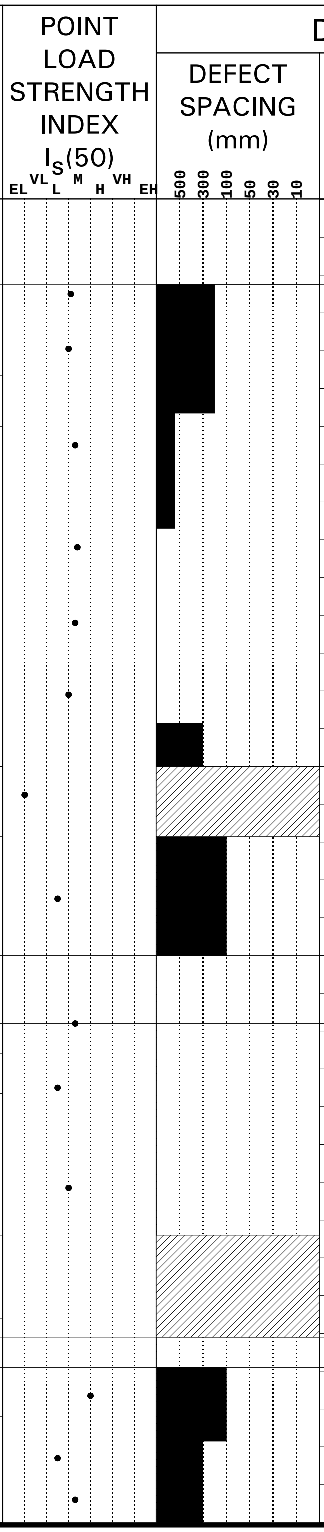
CORED BOREHOLE LOG

Borehole No.
201
2/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 19-2-15
Drill Type: JK305
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 20.7m
Datum: AHD
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
	0	START CORING AT 0.45m			
FULL RET-URN		SANDSTONE: fine to medium grained, orange brown and light grey.	DW	M	- XWS, 0°, 10mm.t - XWS, 0°, 30mm.t
	1			VL	- XWS, 0°, 30mm.t
				L-M	- J, 10°, P, R
	2				- J, 50°, P, R
	3	as above, but red brown and orange brown.	XW	EL	
			DW	L	- XWS, 20mm.t - Be, 10° - XWS, 10mm.t - Be, 10° - Be, 10° - Be, 0°
	4	CORE LOSS 0.36m			
		SANDSTONE: fine to medium grained, red brown and orange brown.	DW	VL L M	
	5	SANDSTONE: fine to coarse grained, red brown and orange brown.	XW	EL	
	6	SILTY CLAY: light grey and orange brown.	MC≈PL	(H)	
		CORE LOSS 0.16m			
		SANDSTONE: fine to coarse grained, orange brown and red brown.	DW	M L	- Be, 0° - CS, 40mm.t - J, 70°, Un, R
	7	SANDSTONE: fine grained, dark grey, with grey laminae, bedded at			

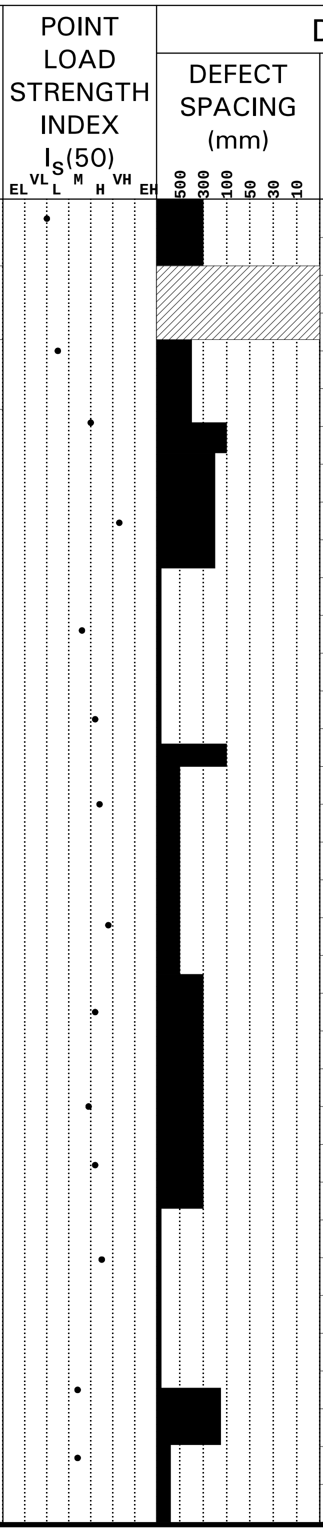


CORED BOREHOLE LOG

Borehole No.
201
3/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 20.7m
Date: 19-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.



Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RETURN			0-10°. SANDSTONE: fine grained, dark grey, with grey laminae, bedded at 0-10°.	DW	VL	- CS, 0°, 70mm.t	
				XW-DW	EL-VL		
				DW	VL		
▼		8					
AFTER 3 DAYS				SW	M-H	- Be, 0° - XWS, 2mm.t - XWS, 2mm.t - XWS, 2mm.t - XWS, 60mm.t - J, SUBVERTICAL, P, R	
		9					
						= J, 45°, Un, R - XWS, 100mm.t	
		10	as above, but grey, with dark grey laminae, bedded at 0-15°.			- Be, 10°	
		11				- Be, 0° - Be, 0° - XWS, 15mm.t - J, SUBVERTICAL, Un, R - Be, 5°	
50% RETURN		12				- Be, 5°	
		13				- Be, 0° - J, 50°, P, R - Cr, 40mm.t	
		14					

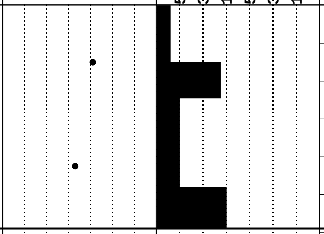
CORED BOREHOLE LOG

Borehole No.
201
4/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 20.7m
Date: 19-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

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
		15		SANDSTONE: fine to medium grained, grey, with dark grey laminae, bedded at 0-15°.	SW	M-H			- XWS, 30mm.t - XWS, 50mm.t - XWS, 10mm.t - Be, 0°	
		16 17 18 19 20		END OF BOREHOLE AT 15.18m						MACHINE SLOTTED PVC STANDPIPE INSTALLED TO 15.18m DEPTH, SLOTTED FROM 15.18m TO 1.68m, CASING 1.68m TO SURFACE, BACKFILLED WITH 2mm FILTER SAND FROM 15.18m TO 1.5m, BENTONITE CLAY FROM 1.5m TO 0.2m, QUICKSET CONCRETE 0.2m TO SURFACE, COMPLETED WITH CAST IRON GATIC COVER AT SURFACE




JK Geotechnics
25077ZH BH201 START CORING AT 0.45m
CORE LOSS 0.36m
CORE LOSS 0.16m
1
2
3
4
5
6
7
8
9


JK Geotechnics
10
11
12
13
14
15
END OF BH201 AT 15.18m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
202
1/4

Client: HARBORD DIGGERS

Project: PROPOSED REDEVEOPMENT

Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH

Date: 24-2-15

Method: SPIRAL AUGER
JK305

R.L. Surface: ≈ 23.5m

Datum: AHD

Logged/Checked by: D.A.F./A.Z.

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 COMPLETION OF AUGERING						0			FILL: Silty sand, fine to medium grained, dark brown.	M			GRASS COVER APPEARS POORLY COMPACTED
				■	N > 4 6,4/100mm REFUSAL				as above, but with brick fragments, trace of sandstone gravel and cobbles.				
				■		1		-	SANDSTONE: fine to medium grained, red brown.	SW	H		HIGH 'TC' BIT RESISTANCE
						2 3 4 5 6 7			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL

CORED BOREHOLE LOG

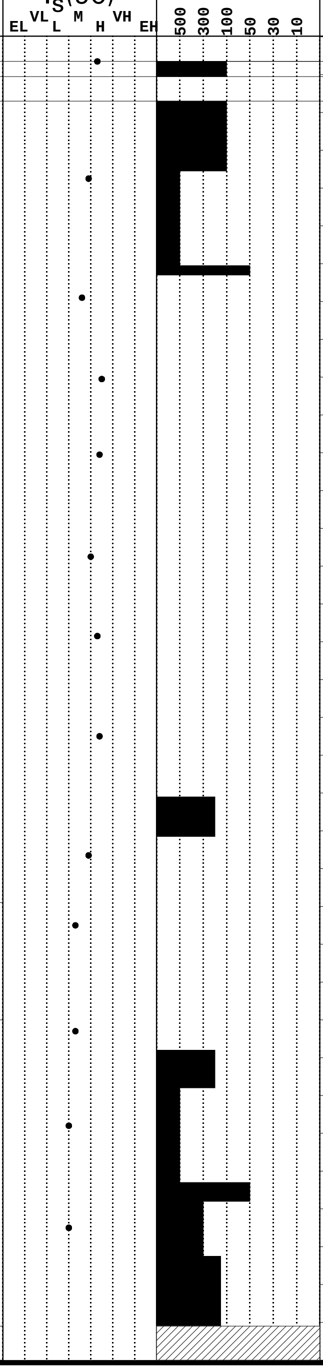
Borehole No.
202
2/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 23.5m
Date: 24-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
	1	START CORING AT 1.13m			
FULL RET-URN		SANDSTONE: fine to medium grained, red brown.	SW	H	
		CORE LOSS 0.13m			
	2	SANDSTONE: fine to medium grained, red brown and orange brown.	SW	M-H	- J, 5°, P, R - Be, 0° - Cr, 5°, 10mm.t - J, 40°, P, R
		as above, but light grey, orange brown and red brown.			
	3 4	SANDSTONE: fine to medium grained, orange brown, bedded at 45°.			
	5	SANDSTONE: fine to coarse grained, red brown.			- Be, 5° - Be, 5°
	6	SANDSTONE: fine to medium grained, orange brown.		M	
		SANDSTONE: fine to coarse grained, orange brown.	DW	L-M	- J, 15°, P, R - J, 15°, P, R
	7	as above, but light grey and orange brown.			- XWS, 0°, 5mm.t - XWS, 0°, 5mm.t - XWS, 0°, 5mm.t - Be, 0° - XWS, 0°, 15mm.t - Be, 5° - Be, 0°
	8	SHALE: grey.	XW-	EL-VL	

POINT LOAD STRENGTH INDEX $I_s(50)$ / DEFECT SPACING (mm)



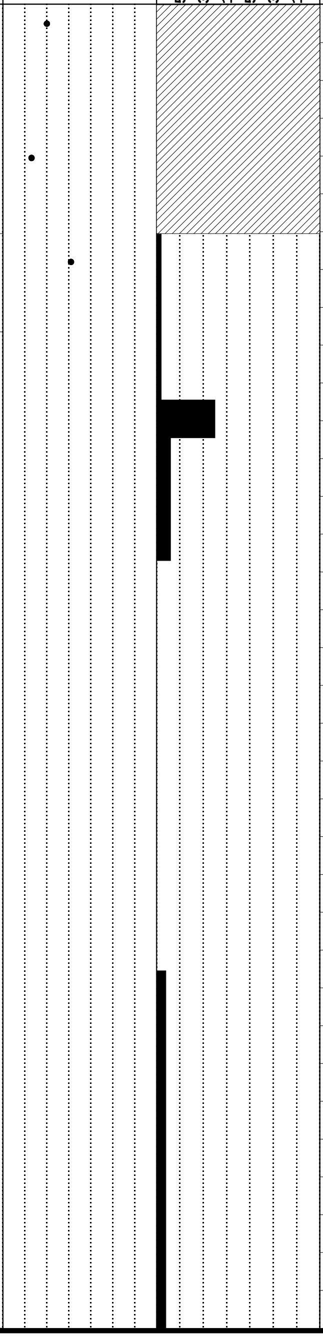
CORED BOREHOLE LOG

Borehole No.
202
3/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 24-2-15
Drill Type: JK305
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 23.5m
Datum: AHD
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
			SHALE: grey.	DW XW-DW	EL-VL		
		9					
				DW	M		
			SANDSTONE: fine grained, grey.				
			SHALE: light grey and grey.	FR	H		
		10					
						- Be, 5°	
						- J, 0°, P, R	
						- J, 30°, Un, R	
		11					
FULL RET-URN							
		12					
			INTERBEDDED SANDSTONE: fine grained, grey, AND SHALE: dark grey and grey.				
		13					
						- Be, 5°	
						- Cr, 10mm.t	
		14					
						- J, 30°, P, R	



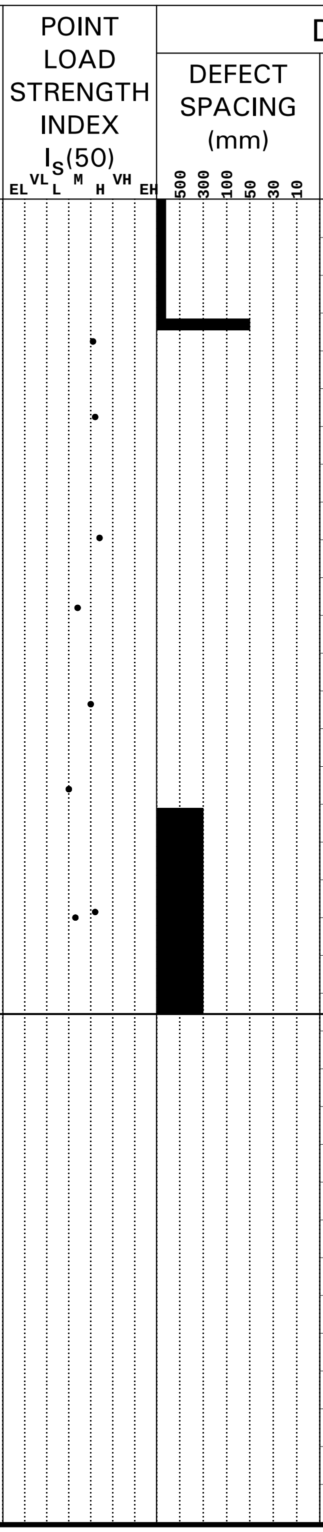
CORED BOREHOLE LOG

Borehole No.
202
4/4

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 23.5m
Date: 24-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
			INTERBEDDED SANDSTONE: fine grained, grey, AND SHALE: dark grey and grey.	FR	H		
						- XWS, 2mm.t - XWS, 2mm.t	
		16					
FULL RET-URN		17					
			SANDSTONE: fine to medium grained, grey.			- CS, 5°, 10mm.t	
		18					
						- CS, 0°, 30mm.t	
						- Be, 0°	
		19				- XWS, 0°, 25mm.t	
						- CS, 0°, 50mm.t	
			END OF BOREHOLE AT 19.31m				
		20					
		21					




JK Geotechnics
25077ZH BH202 START CORING AT 1.13m
CORE LOSS 0.13m
1
2
3
4
5
6
7
8
9


JK Geotechnics
10
11
12
13
14
15
16
17
18
19
END OF BH202 AT 19.31m

BOREHOLE LOG

Borehole No.
203
1/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 20-2-15
Method: SPIRAL AUGER
JK305
R.L. Surface: ≈ 20.2m
Datum: AHD
Logged/Checked by: D.A.F./A.Z.

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 COMPLETION OF AUGERING						0			CONCRETE: 220mm.t				8mm DIA. REINFORCEMENT, 100mm TOP COVER
	■							-	FILL: Silty sand, fine to medium grained, light orange brown, with clay and fine to coarse grained sandstone gravel.	M			APPEARS POORLY COMPACTED
	■							-	SANDSTONE: fine to coarse grained, red brown.	SW	H		MODERATE 'TC' BIT RESISTANCE
						1			REFER TO CORED BOREHOLE LOG				
						2							
						3							
						4							
						5							
						6							
						7							

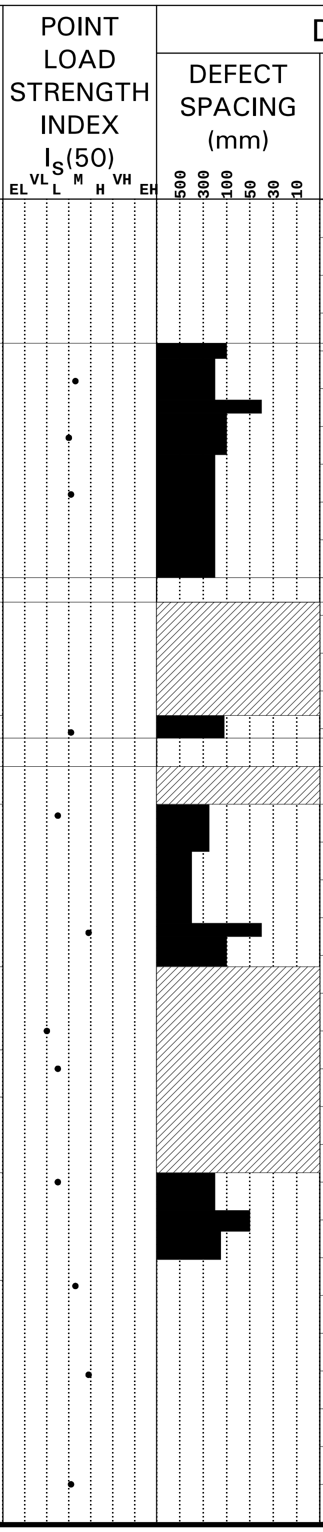
CORED BOREHOLE LOG

Borehole No.
203

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 20.2m
Date: 20-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
	0	START CORING AT 0.60m				
FULL RET-URN	1	SANDSTONE: fine to coarse grained, red brown.	SW	M	- J, 50°, P, HEALED - Be, 0° - Be, 0° - Be, 0°	
		as above, but light grey.			- Be, 0° - J, 30°, P, R	
		SANDSTONE: fine to coarse grained, orange brown.			- Be, 15° - Be, 5°	
	2	CORE LOSS 0.13m				
		SANDSTONE: fine to coarse grained, red brown and orange brown.	XW	EL		
		as above, but light grey.	SW	M		
	3	CORE LOSS 0.15m				
		SANDSTONE: fine to coarse grained, light grey.	XW DW	EL L-M	- Be, 0°	
	4	as above, but brown.			- Be, 0° - Be, 0° - Be, 0° - CS, 30mm.t - Be, 0°	
		SILTY CLAY: high plasticity, grey.	MC<PL	(H)		
		SHALE: dark grey.	XW-DW	EL-VL	- CS, 30mm.t	
	5	SILTY CLAY: high plasticity, grey.	MC>PL	H	HP 500,400,420	
		SHALE: dark grey.	DW	L	- CS, 0°, 10mm.t - Cr, 0°, 10mm.t - XWS, 0°, 2mm.t - XWS, 10mm.t	
	6 7		SW	M-H	- J, SUBVERTICAL, Un, R	



JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
203
3/3

Client: HARBORD DIGGERS

Project: PROPOSED REDEVEOPMENT

Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 20.2m

Date: 20-2-15
Inclination: VERTICAL
Datum: AHD

Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

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 DETAILS DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific General
FULL RET-URN			▤	SHALE: dark grey.	SW	M-H	●		
				END OF BOREHOLE AT 7.39m					
		8							
		9							
		10							
		11							
		12							
		13							
		14							


JK Geotechnics
25077ZH BH203 START CORING AT 0.76m
0
1
CORE LOSS 0.15m
2
CORE LOSS 0.13m
3
4
5
6
7
END OF BH203 AT 7.39m

BOREHOLE LOG

Borehole No.
203A
1/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 27-2-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: D.A.F./A.Z.
R.L. Surface: ≈ 20.2m
Datum: AHD

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 OF AUGERING						0			CONCRETE: 240mm.t				
								-	FILL: Silty sand, fine to medium grained, light orange brown, with clay and fine to coarse grained sandstone gravel.	M			
						1		-	SANDSTONE: fine to medium grained, orange brown and red brown.	SW	M-H		HIGH 'TC' BIT RESISTANCE
						2 3 4 5 6 7			REFER TO CORED BOREHOLE LOG				

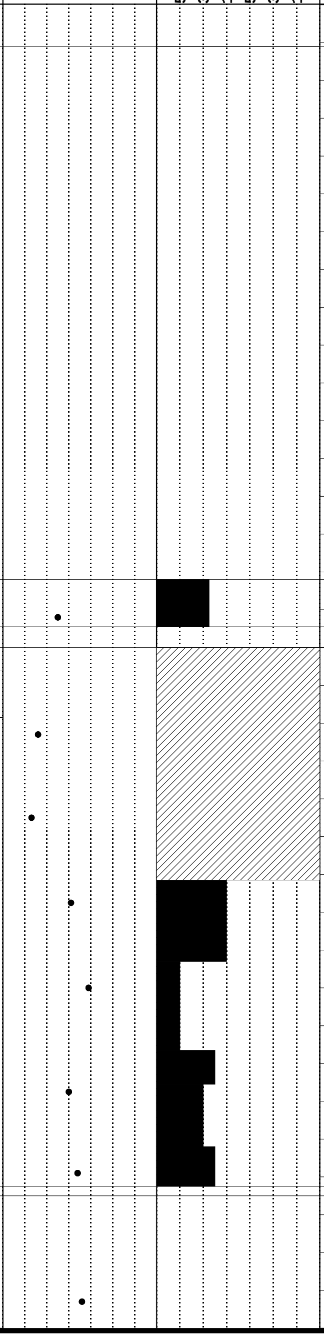
CORED BOREHOLE LOG

Borehole No.
203A
2/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 20.2m
Date: 27-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific
	1	START CORING AT 1.22m			
	2, 3, 4	CORE LOSS 2.82m			
FULL RET-URN		SANDSTONE: fine to coarse grained, orange brown.	DW	M	
		CORE LOSS 0.11m			
		SHALE: dark grey.	DW	VL-L	
		SILTY CLAY: medium to high plasticity, grey and dark grey.	MC≈PL	VSt-H	
	5	SHALE: grey and dark grey.	XW-DW	EL-VL	
	6, 7	INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW	M-H	- J, SUBVERTICAL, P, R - Cr, 0°, 40mm.t - J, SUBVERTICAL, P, R - J, 70°, Un, R - Cr, 0°, 10mm.t - Be, 0° - Be, 0°
		CORE LOSS 0.05m			
	8	INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW	M-H	



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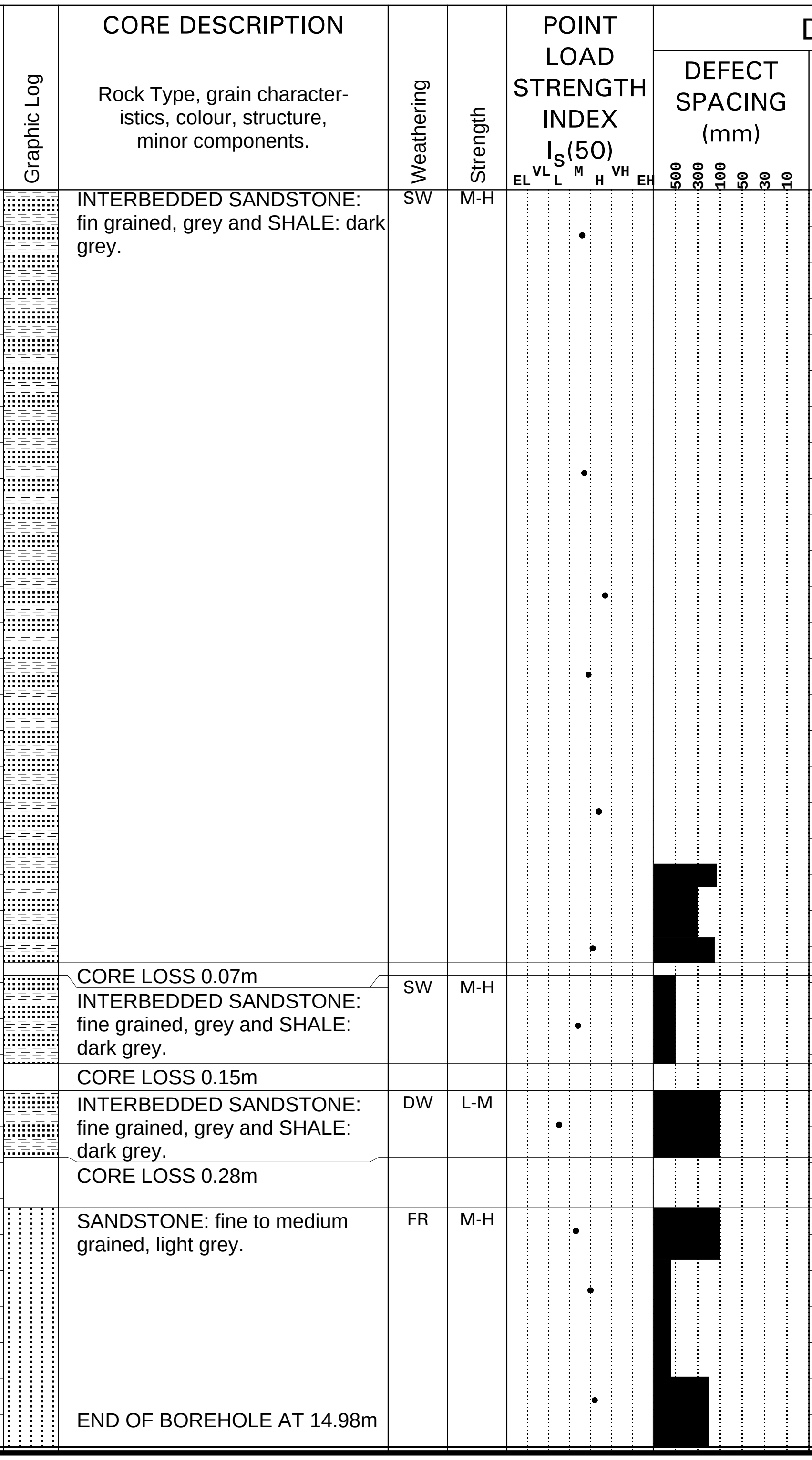
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
203A
3/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 27-2-15
Drill Type: JK305
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 20.2m
Datum: AHD
Logged/Checked by: D.A.F./A.Z.



Water Loss/Level	Barrel Lift	Depth (m)	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RET-URN		9 10 11 12	INTERBEDDED SANDSTONE: fin grained, grey and SHALE: dark grey.	SW	M-H	- Be, 0° - J, 40°, P, R - XWS, 0°, 10mm.t - CS, 0°, 5mm.t	
			CORE LOSS 0.07m				
			INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW	M-H	- J, 70°, P, R	
			CORE LOSS 0.15m				
		13	INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	DW	L-M	- Be, 0°, - CS, 0°, 10mm.t - Cr, 0°, 110mm.t	
			CORE LOSS 0.28m				
		14	SANDSTONE: fine to medium grained, light grey.	FR	M-H	- Be, 0° - Be, 5° - Be, 10° - Be, 5°	
			END OF BOREHOLE AT 14.98m				


JK Geotechnics
25077ZH BH203A START CORING AT 1.23m
CORE LOSS 2.82m
CORE LOSS 0.11m
5
6
7
CORE LOSS 0.05m
8
CORE LOSS 0.20m
9
10
11
12
CORE LOSS 0.15m
CORE LOSS 0.07m
13
CORE LOSS 0.28m
14
END OF BH203 AT 14.98m

BOREHOLE LOG

Borehole No.
204
1/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 25-2-15
Method: SPIRAL AUGER
JK305
Logged/Checked by: D.A.F./A.Z.
R.L. Surface: ≈ 17.8m
Datum: AHD

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 OF AUGERING						0			FILL: Silty sand, fine to medium grained, brown, trace of brick fragments.				GRASS COVER
				■	N = 8 4,4,4	1		CL-CH	SILTY CLAY: medium to high plasticity, grey and dark grey, with sand and fine grained quartz gravel.	MC>PL MC<PL	St		RESIDUAL
				■	N > 36 10,17, 19/120mm REFUSAL	2		-	SHALE: light grey and grey.	XW	EL		VERY LOW 'TC' BIT RESISTANCE
▼ AFTER 3 DAYS				■	N = 36 13,16,20	3							
						4		-	SANDSTONE: fine to medium grained, red brown, orange brown and grey.	DW	VL-L		VERY LOW TO LOW RESISTANCE
						5 6 7			REFER TO CORED BOREHOLE LOG				

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
204
2/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 17.8m
Date: 25-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

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 4.27m						
FULL RET-URN				SANDSTONE: fine to medium grained, red brown, orange brown and grey.	DW	VL-L	●			
						L-M	●			
				as above, but light grey, red brown and light orange brown.			●			
		5					●			
				SHALE: grey.	XW	EL		▨		
				SANDSTONE: fine to medium grained, light grey, red brown and light orange brown.	DW	L-M	●			
		6							- Be, 5mm.t	
						M-H	●			
		7					●			
		8					●			
		9					●			
		10		INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW		●		- CS, 5°, 15mm.t	

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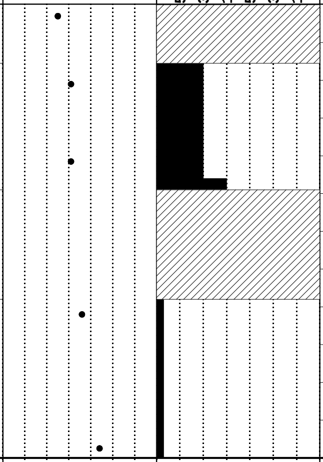
CORED BOREHOLE LOG

Borehole No.
204
3/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 17.8m
Date: 25-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.



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 $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
FULL RETURN				SANDSTONE: fine to medium grained, grey, with occasional shale lenses.	XW-DW	EL-VL				
					DW	M				
		12			XW	EL				
		13		SANDSTONE: fine to coarse grained, red brown.	SW	M-H				
		14		END OF BOREHOLE AT 13.40m						MACHINE SLOTTED PVC STANDPIPE INSTALLED TO 13.2m DEPTH, SLOTTED FROM 13.2m TO 2.7m, CASING 2.7m TO SURFACE, BACKFILLED WITH 2mm FILTER SAND FROM 13.2m TO 2.2m, BENTONITE CLAY FROM 2.2m TO 0.2m, QUICKSET CONCRETE 0.2m TO SURFACE, COMPLETED WITH CAST IRON GATIC COVER AT SURFACE
		15								
		16								
		17								


JK Geotechnics
25077ZH BH204 START CORING AT 4.27m
4
5
6
7
8
9
10
11
12
13
END OF BH204 AT 13.40m

BOREHOLE LOG

Borehole No.
205
1/3

Client: HARBORD DIGGERS

Project: PROPOSED REDEVEOPMENT

Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH

Date: 23-2-15

Method: SPIRAL AUGER
JK305

R.L. Surface: ≈ 22.7m

Datum: AHD

Logged/Checked by: D.A.F./A.Z.

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION OF AUGERING						0			CONCRETE: 190mm.t				8mm DIA. REINFORCMENT, 100mm TOP COVER
								-	FILL: Gravel, medium grained, igneous.	W			
									REFER TO CORED BOREHOLE LOG				
						1							
						2							
						3							
						4							
						5							
						6							
						7							

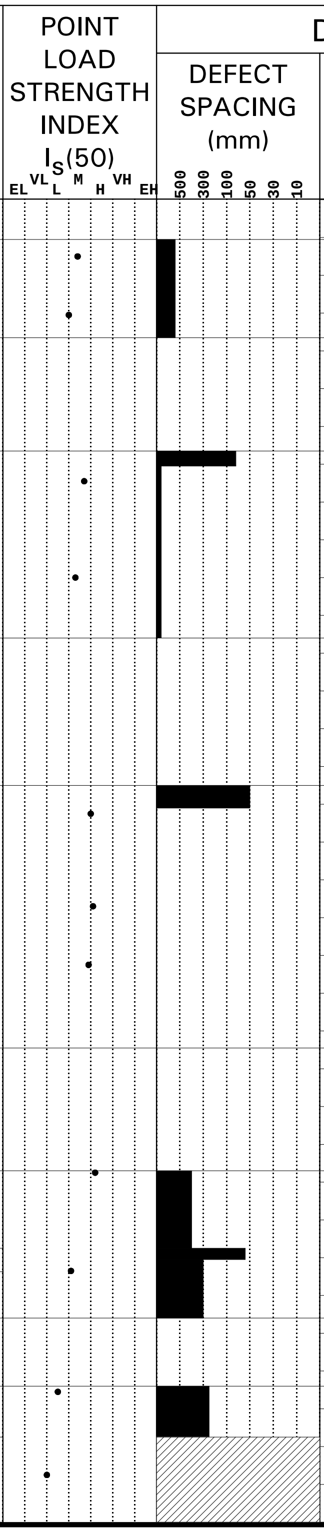
CORED BOREHOLE LOG

Borehole No.
205
2/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 22.7m
Date: 23-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0		START CORING AT 0.21m				
FULL RET-URN				SANDSTONE: fine to medium grained, red brown and orange brown.	DW	M		
		1		CORE LOSS 0.60m				
				SANDSTONE: fine to medium grained, red brown.	DW	M	- CS, 0°, 10mm.t	
		2						
				CORE LOSS 0.78m				
		3						
				SANDSTONE: fine to coarse grained, light orange brown.	DW-SW	M-H	- XWS, 0°, 40mm.t - XWS, 0°, 30mm.t	
NO RET-URN		4						
				CORE LOSS 0.65m				
		5						
				SANDSTONE: fine to medium grained, grey and red brown.	DW	H		
				as above, but light grey.		M H	- Be - J, 20°, P, R	
				as above, but dark red brown.				
ON COMPLETION OF CORING ▼		6		CORE LOSS 0.36m				
				INTERBEDDED SANDSTONE: fine grained, grey, and SHALE: dark grey.	DW XW-DW	L EL-VL		
		7						



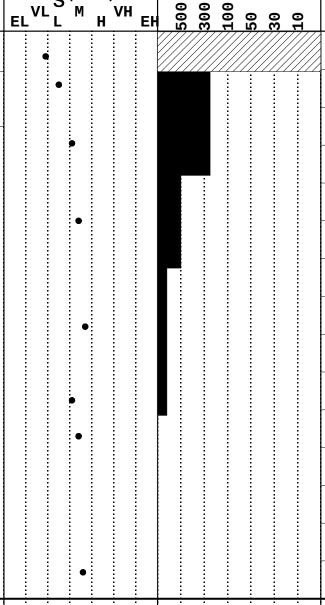
CORED BOREHOLE LOG

Borehole No.
205
3/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 22.7m
Date: 23-2-15
Inclination: VERTICAL
Datum: AHD
Drill Type: JK305
Bearing: -
Logged/Checked by: D.A.F./A.Z.

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
NO RET-URN				INTERBEDDED SANDSTONE: fine grained, grey, and SHALE: dark grey.	XW	EL				
					DW	L-M				
					SW	M			- J, 60°, P, R	
									- Cr, 0°, 20mm.t	
		8								
									- J, SUBVERTICAL, Un, R	
		9							- J, 40°, Un, R	
		10		END OF BOREHOLE AT 10.00m						
		11								
		12								
		13								
		14								




JK Geotechnics
25077ZH BH205 START CORING AT 0.21m
CORE LOSS 0.60m
CORE LOSS 0.78m
CORE LOSS 0.65m
CORE LOSS 0.36m
END OF BH205 AT 10.00m
0
1
2
3
4
5
6
7
8
9

CORED BOREHOLE LOG

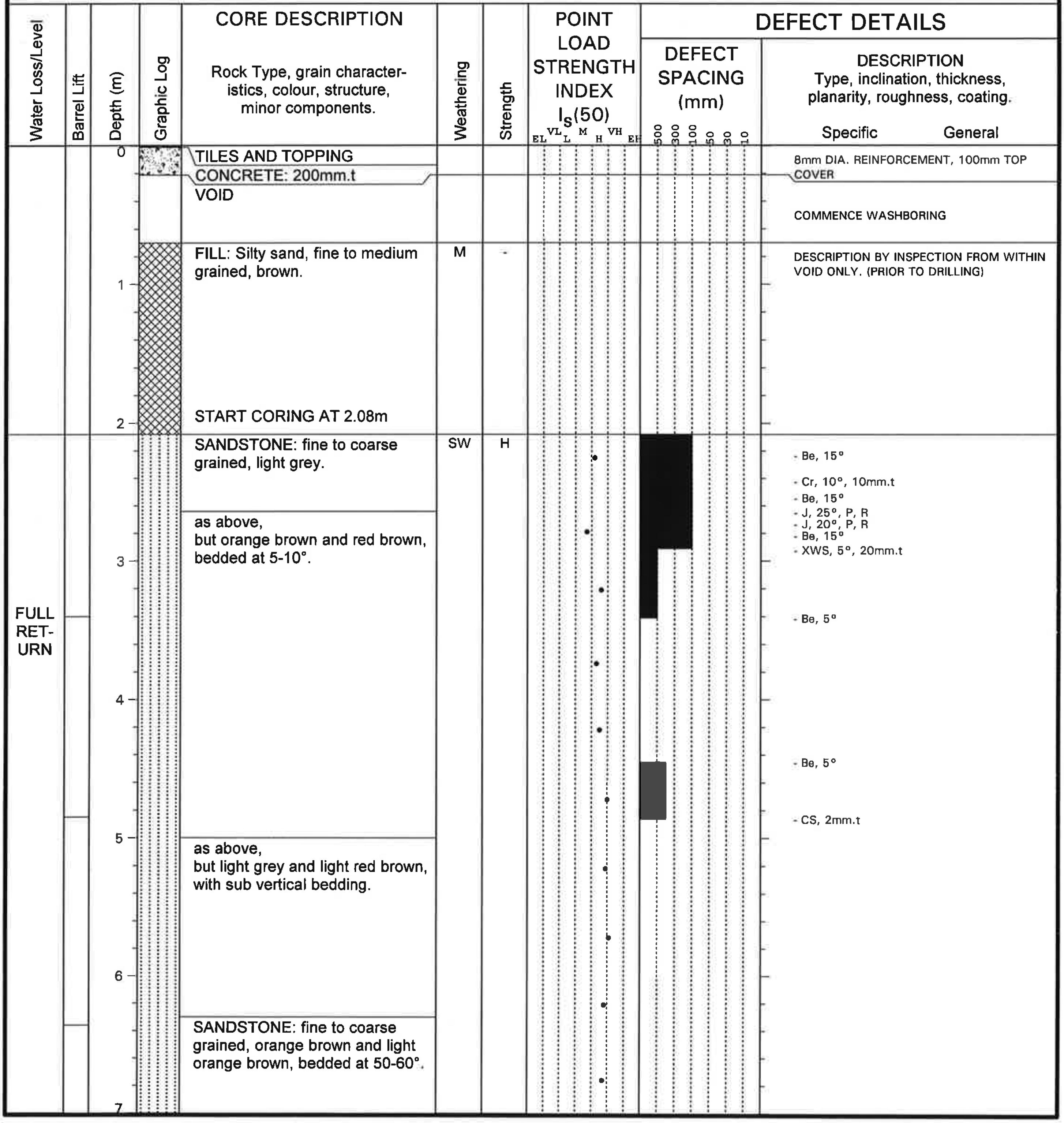
Borehole No.
206
1/3

Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Date: 10-3-15
Drill Type: MELVELLE

Core Size: NMLC
Inclination: VERTICAL
Bearing: -

R.L. Surface: ≈ 25.1m
Datum: AHD
Logged/Checked by: D.A.F./A.Z.



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 $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		0		TILES AND TOPPING						
				CONCRETE: 200mm.t					8mm DIA. REINFORCEMENT, 100mm TOP COVER	
				VOID					COMMENCE WASHBORING	
		1		FILL: Silty sand, fine to medium grained, brown.	M	-			DESCRIPTION BY INSPECTION FROM WITHIN VOID ONLY. (PRIOR TO DRILLING)	
		2		START CORING AT 2.08m						
FULL RET-URN				SANDSTONE: fine to coarse grained, light grey.	SW	H			- Be, 15° - Cr, 10°, 10mm.t - Be, 15°	
		3		as above, but orange brown and red brown, bedded at 5-10°.					- J, 25°, P, R - J, 20°, P, R - Be, 15° - XWS, 5°, 20mm.t - Be, 5° - Be, 5° - CS, 2mm.t	
		4								
		5		as above, but light grey and light red brown, with sub vertical bedding.						
		6		SANDSTONE: fine to coarse grained, orange brown and light orange brown, bedded at 50-60°.						
		7								

CORED BOREHOLE LOG

Borehole No.
206
2/3

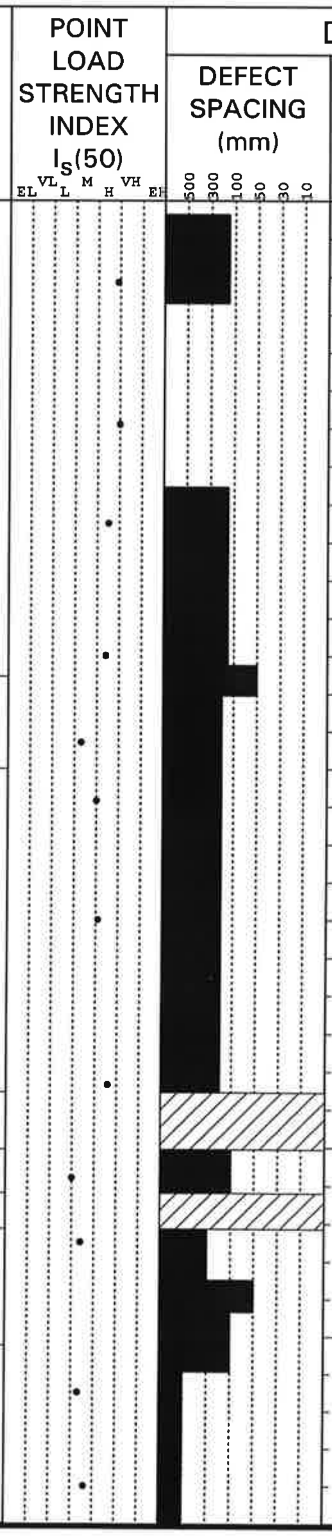
Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH	**Core Size:** NMLC	**R.L. Surface:** ≈ 25.1m
Date: 10-3-15	**Inclination:** VERTICAL	**Datum:** AHD
Drill Type: MELVELLE	**Bearing:** -	**Logged/Checked by:** D.A.F./A.Z.

Water Loss/Level: FULL RET URN

Depth (m): 8, 9, 10, 11, 12, 13, 14

CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength
SANDSTONE: fine to coarse grained, orange brown and light orange brown, bedded at 50-60°.	SW	H
as above, but bedded at 10-15°		
SANDSTONE: fine to coarse grained, red brown and light grey.		
as above, but orange brown and light grey.	DW	M
	SW	H
SANDSTONE: fine to coarse grained, light grey.		
SANDSTONE: fine to coarse grained, orange brown, red brown and light grey.		
SANDSTONE: fine to medium grained, light grey and grey.	XW	EL
	DW	L
	XW	EL
	DW	L-M
INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW	M-H



DEFECT DETAILS
DESCRIPTION
Type, inclination, thickness, planarity, roughness, coating.
Specific General

- XWS, 5mm.t
- Be, 10°
- Be, 15°
- J, 15°, P, R
- Be, 10°
- Be, 0°
- Be, 5°
- Be, 5°
- CS
- J, 20°, Un, R
- Be, 5°
- Be, 10°
- Be, 10°
- Be, 10°
- J, SUBVERTICAL, Un, CL
- Be, 10°
- XWS, 5°, 20mm.t
- CS, 30mm.t
- Be, 0°
- Be, 0°
- Be, 0°
- Be, 0°
- XWS, 0°, 5mm.t
- CS, 0°, 5mm.t
- Cr, 20°, 15mm.t
- Be, 0°
- XWS, 0°, 40mm.t
- CS, 0°, 60mm.t
- Be, 0°
- Be, 0°V
- Be, 0°
- XWS, 0°, 10mm.t
- Be, 5°
- Be, 0°

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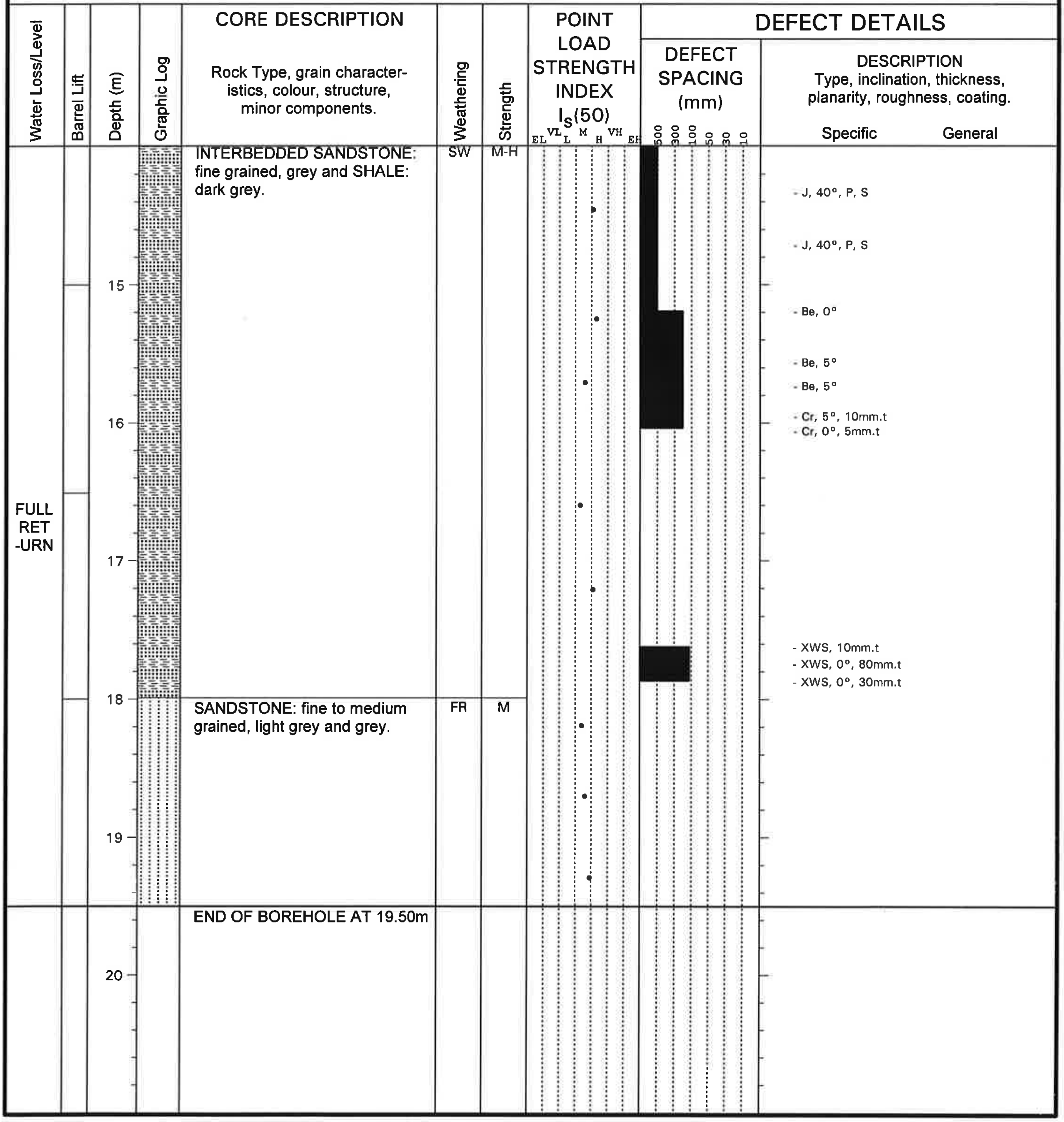
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
206
3/3

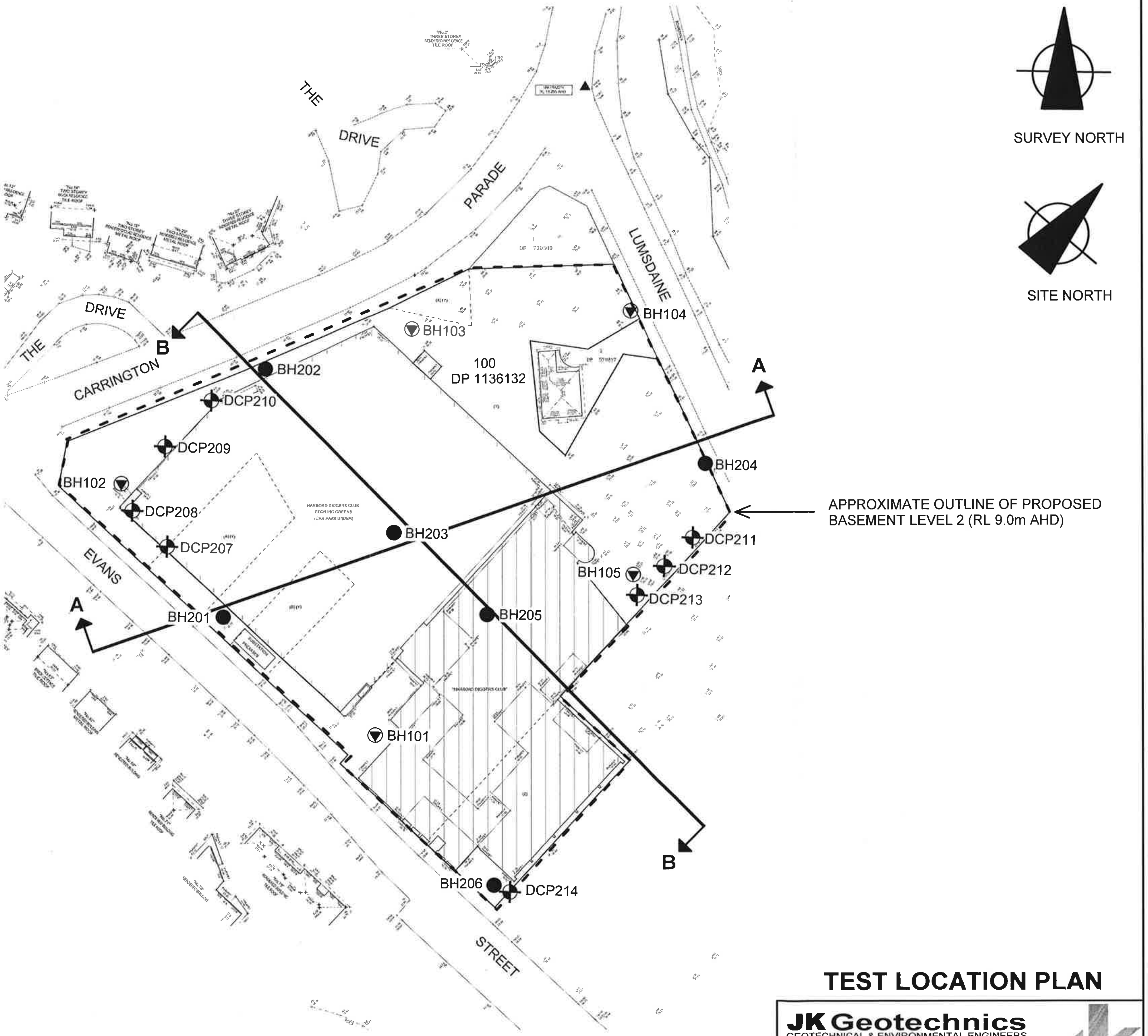
Client: HARBORD DIGGERS
Project: PROPOSED REDEVEOPMENT
Location: EVANS STREET, FRESHWATER, NSW

Job No. 25077ZH
Core Size: NMLC
R.L. Surface: ≈ 25.1m
Date: 10-3-15
Inclination: VERTICAL
Datum: AHD
Drill Type: MELVELLE
Bearing: -
Logged/Checked by: D.A.F./A.Z.



Water Loss/Level	Depth (m)	Core Description: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	Defect Details: Description (Type, inclination, thickness, planarity, roughness, coating.) Specific	General
FULL RET -URN		INTERBEDDED SANDSTONE: fine grained, grey and SHALE: dark grey.	SW	M-H	- J, 40°, P, S - J, 40°, P, S	
	15				- Be, 0° - Be, 5° - Be, 5° - Cr, 5°, 10mm.t - Cr, 0°, 5mm.t	
	16					
	17				- XWS, 10mm.t - XWS, 0°, 80mm.t - XWS, 0°, 30mm.t	
	18	SANDSTONE: fine to medium grained, light grey and grey.	FR	M		
	19					
		END OF BOREHOLE AT 19.50m				
	20					


JK Geotechnics
25077ZH BH206 START CORING AT 2.08m
2
3
4
5
6
7
8
9
10



LEGEND

- ● BOREHOLE (STAGE 2 INVESTIGATION, FEB 2015)
- ⊕ DYNAMIC CONE PENETROMETER (DCP) TEST (FEB 2015)
- ▼ BOREHOLE (STAGE 1 INVESTIGATION, SEPT 2012)

SCALE (m)
0 50

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TEST LOCATION PLAN

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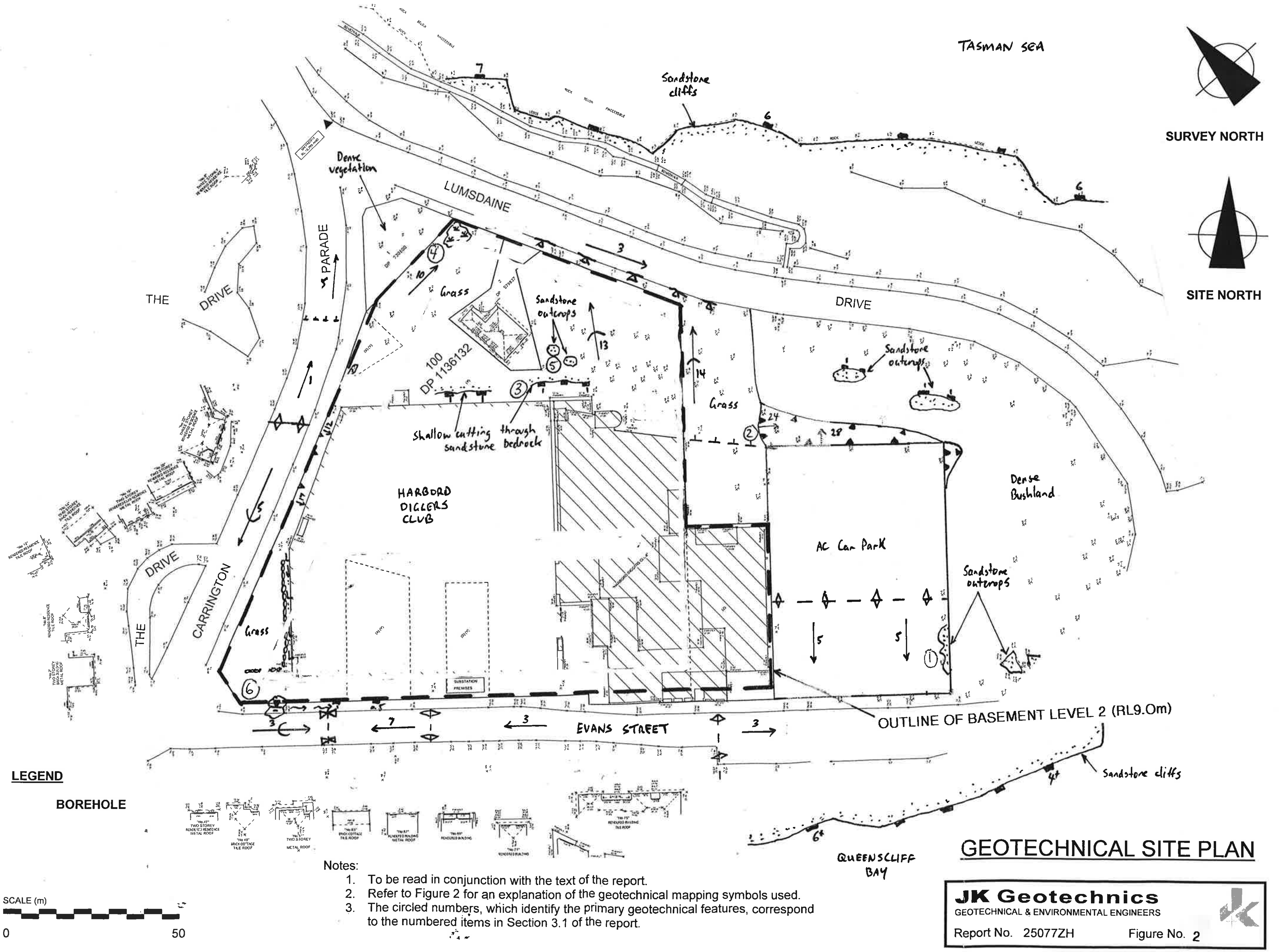
Report No. 25077ZH Figure No. 1



LEGEND

BOREHOLE

Notes:

1. To be read in conjunction with the text of the report.
2. Refer to Figure 2 for an explanation of the geotechnical mapping symbols used.
3. The circled numbers, which identify the primary geotechnical features, correspond to the numbered items in Section 3.1 of the report.

GEOTECHNICAL SITE PLAN

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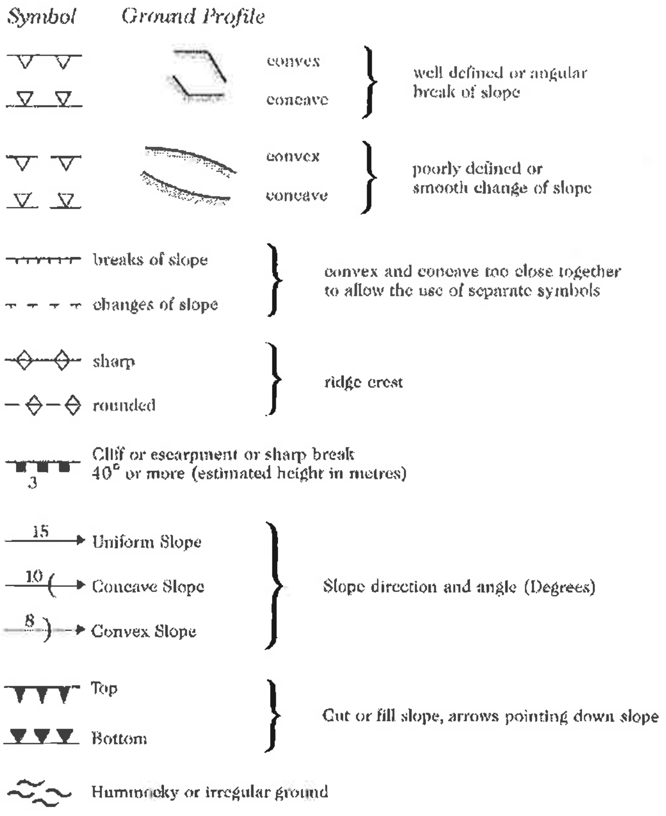
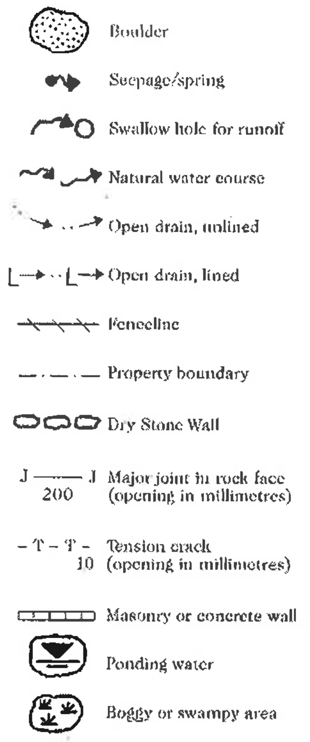
Report No. 25077ZH

Figure No. 2

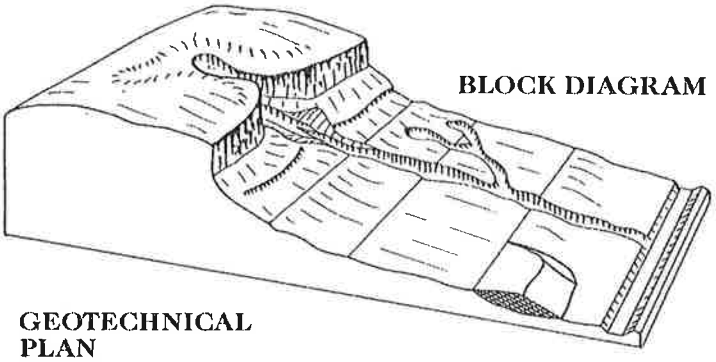
TOPOGRAPHY



OTHER FEATURES

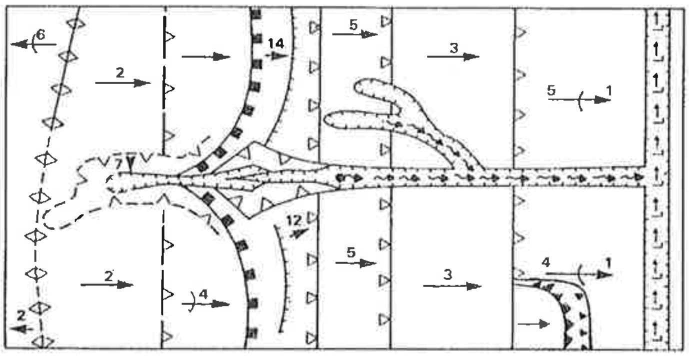


EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



BLOCK DIAGRAM

GEOTECHNICAL PLAN



(After Gardiner, V & Dackombe, R.V. (1983), Geomorphological Field Manual; George Allen & Unwin).

GEOTECHNICAL MAPPING SYMBOLS

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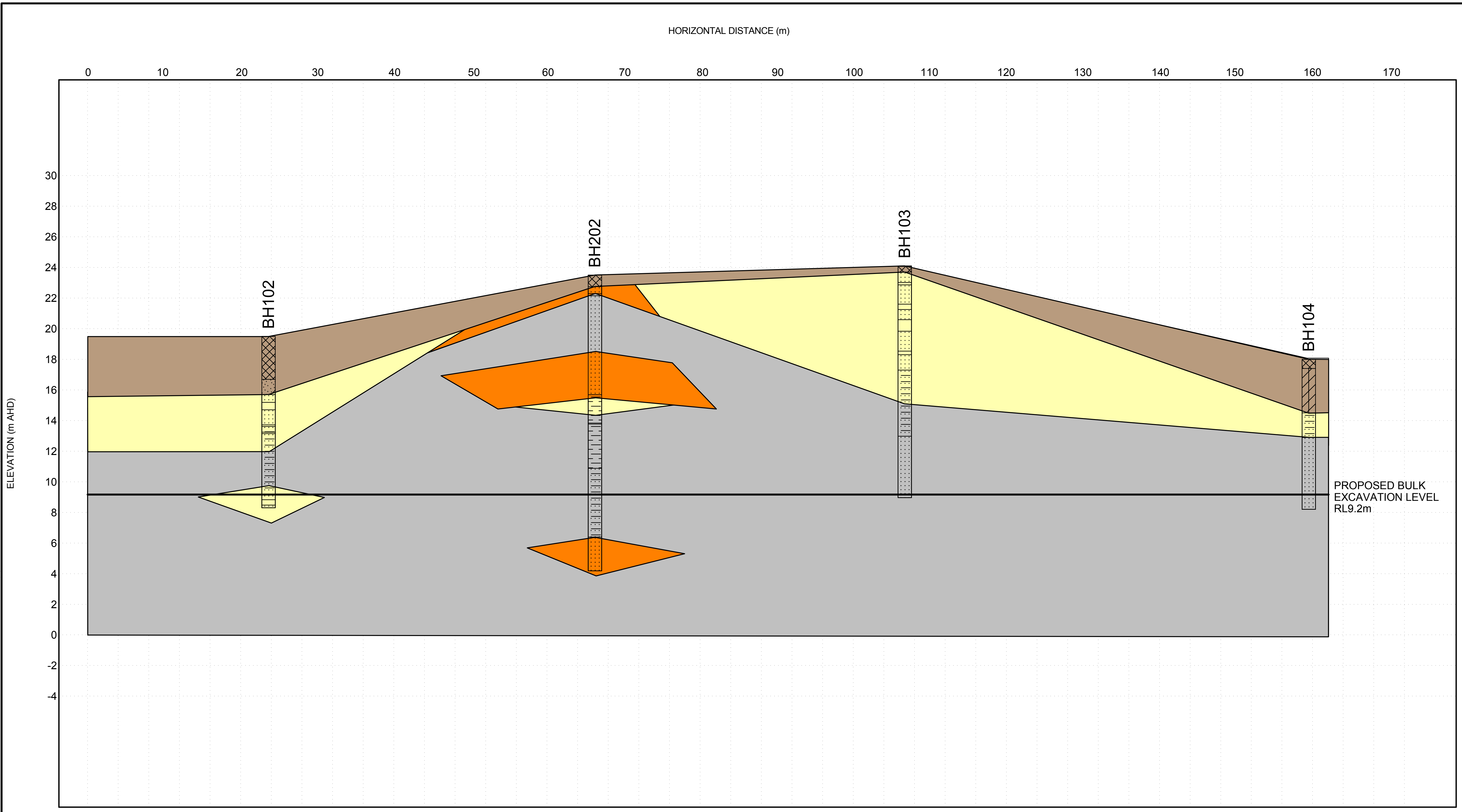
Report No. 25077ZH3 Figure No. 3



LEGEND

- ■ ASPHALTIC CONCRETE
- ▨ CLAYEY SAND
- ☐ CORE LOSS
- ▨ SILTY CLAY
- ☒ FILL
- ▨ SANDSTONE
- ▤ SHALE
- ■ SOIL PROFILE
- ■ CLASS V BEDROCK
- ■ CLASS IV BEDROCK
- ■ CLASS III OR BETTER BEDROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

No.	Amendment Description	Initials	Date
C	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
B	CLASS V UNIT BOUNDARIES	A.P.C	20/04/15
A	INITIAL DRAFT	A.P.C	29/03/15

A3 Original This sheet may be prepared using colour and may be incomplete if copied

Coordinate System: MGA94 Zone 56

Height Datum: AHD

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ENVIRONMENTAL
ENGINEERS

DESIGNED: A.P.C

REVIEWED: A.Z

JK Geotechnics

HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMMARY - WEST FACE

SCALES

H 1:500

V 1:250

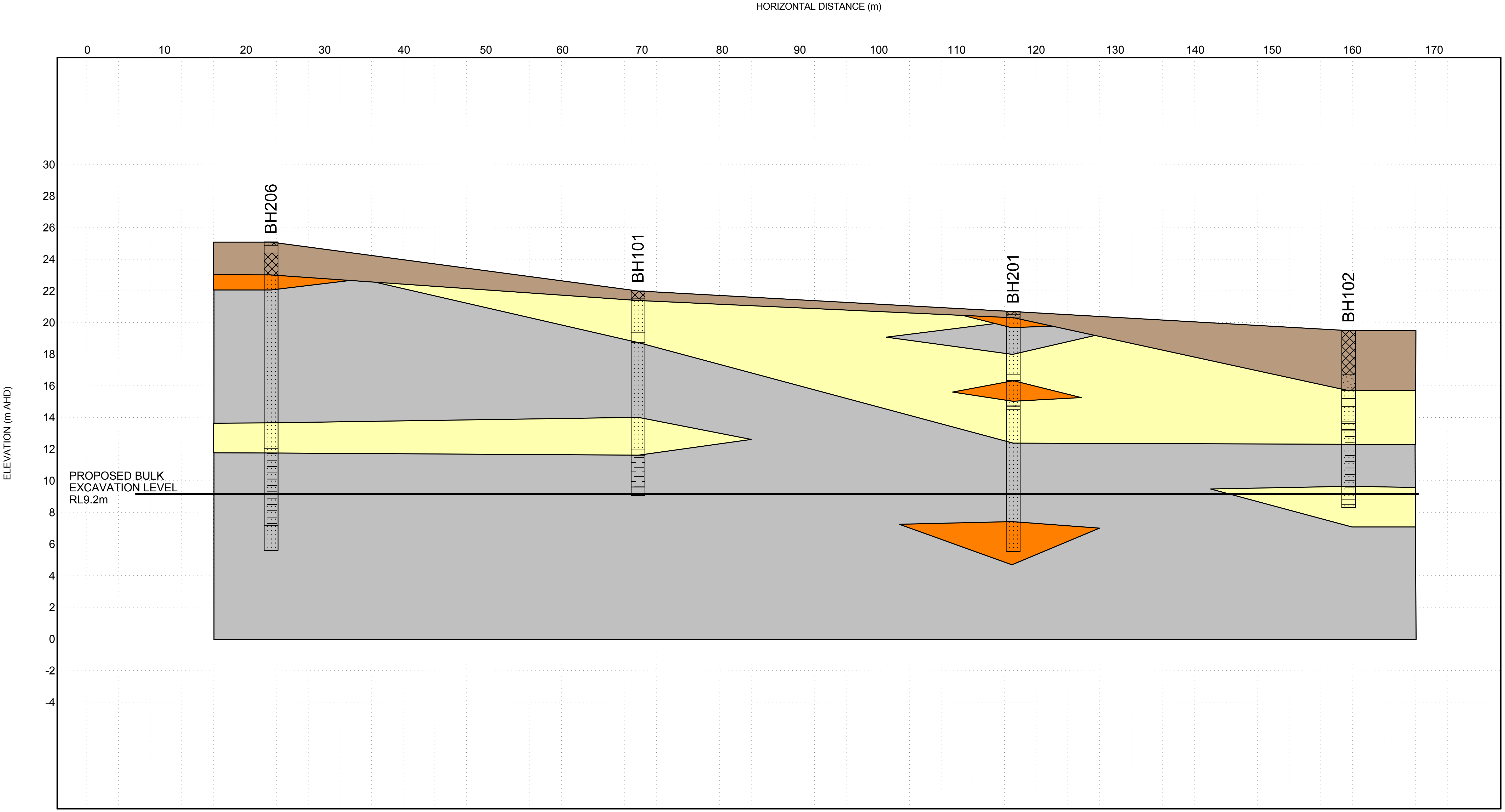
JOB NUMBER

25077ZH

FIGURE NUMBER

4

JK_LIB_CURRENT - V7.3.GLB Fence FENCE A3L NO PLAN 25077ZH FRESHWATER.GPJ 25077ZH FIG4 WEST FACE REVC.GDW 12/03/2015 08:28 Produced by gINT Professional, Developed by Datgel



LEGEND

- ☐ BLANK
- ▨ CLAYEY SAND
- ☐ CORE LOSS
- ▨ SAND
- ▨ SILTY CLAY
- ▨ CONCRETE
- ▨ FILL
- ▨ SANDSTONE
- ▨ SHALE
- 🟫 SOIL PROFILE
- 🟨 CLASS V BEDROCK
- 🟧 CLASS IV BEDROCK
- ⬜ CLASS III OR BETTER BEBROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

No.	Amendment Description	Initials	Date
B	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
A	INITIAL DRAFT	A.P.C	29/03/15

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DESIGNED: A.P.C
REVIEWED: A.Z

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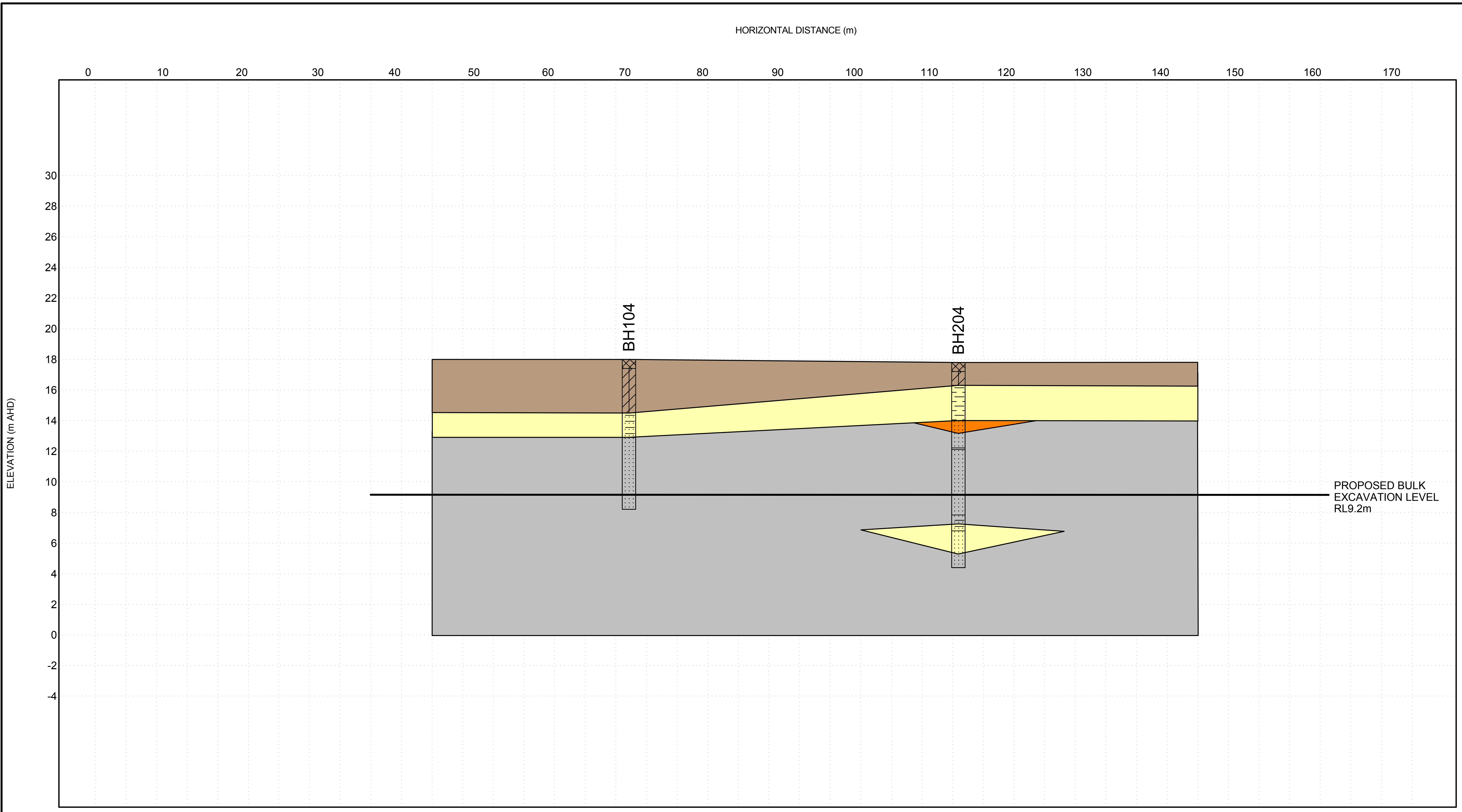
HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMMARY - SOUTH FACE

SCALES
H 1:500
V 1:250

JOB NUMBER
25077ZH

FIGURE NUMBER
5



LEGEND

- ▨ SILTY CLAY
- ☒ FILL
- ⁙ SANDSTONE
- ☰ SHALE
- 🟫 SOIL PROFILE
- 🟨 CLASS V BEDROCK
- 🟧 CLASS IV BEDROCK
- ⬜ CLASS III OR BETTER BEBROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

JK_LIB_CURRENT - V7.3.GLB Fence FENCE A3L NO PLAN 25077ZH FRESHWATER.GPJ 25077ZH FIG6 NORTH FACE REVB.GDW 12/03/2015 08:32 Produced by gINT Professional, Developed by Datgel

No.	Amendment Description	Initials	Date
B	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
A	INITIAL DRAFT	A.P.C	29/03/15

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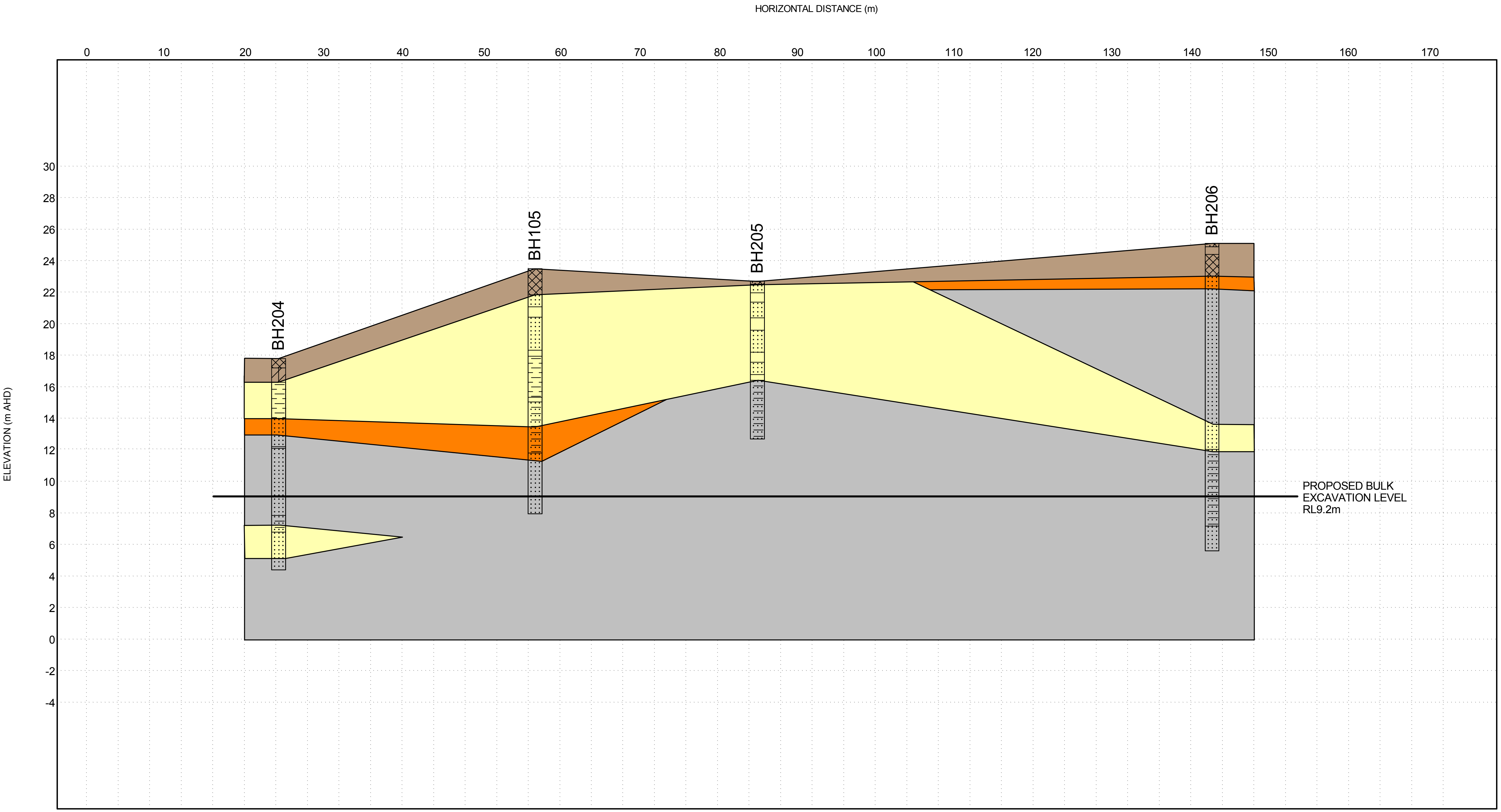
HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMMARY - NORTH FACE

SCALES
H 1:500
V 1:250

JOB NUMBER
25077ZH

FIGURE NUMBER
6



LEGEND

- ☐ BLANK
- ☐ CORE LOSS
- ▨ SILTY CLAY
- ⁘ CONCRETE
- ☒ FILL
- ⁘ SANDSTONE
- ☰ SHALE
- 🟫 SOIL PROFILE
- 🟨 CLASS V BEDROCK
- 🟧 CLASS IV BEDROCK
- ⬜ CLASS III OR BETTER BEBROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

B	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
A	INITIAL DRAFT	A.P.C	29/03/15
No.	Amendment Description	Initials	Date

A3 Original

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REVIEWED: A.Z

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HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMAMRY - EAST FACE

SCALES

H 1:500

V 1:250

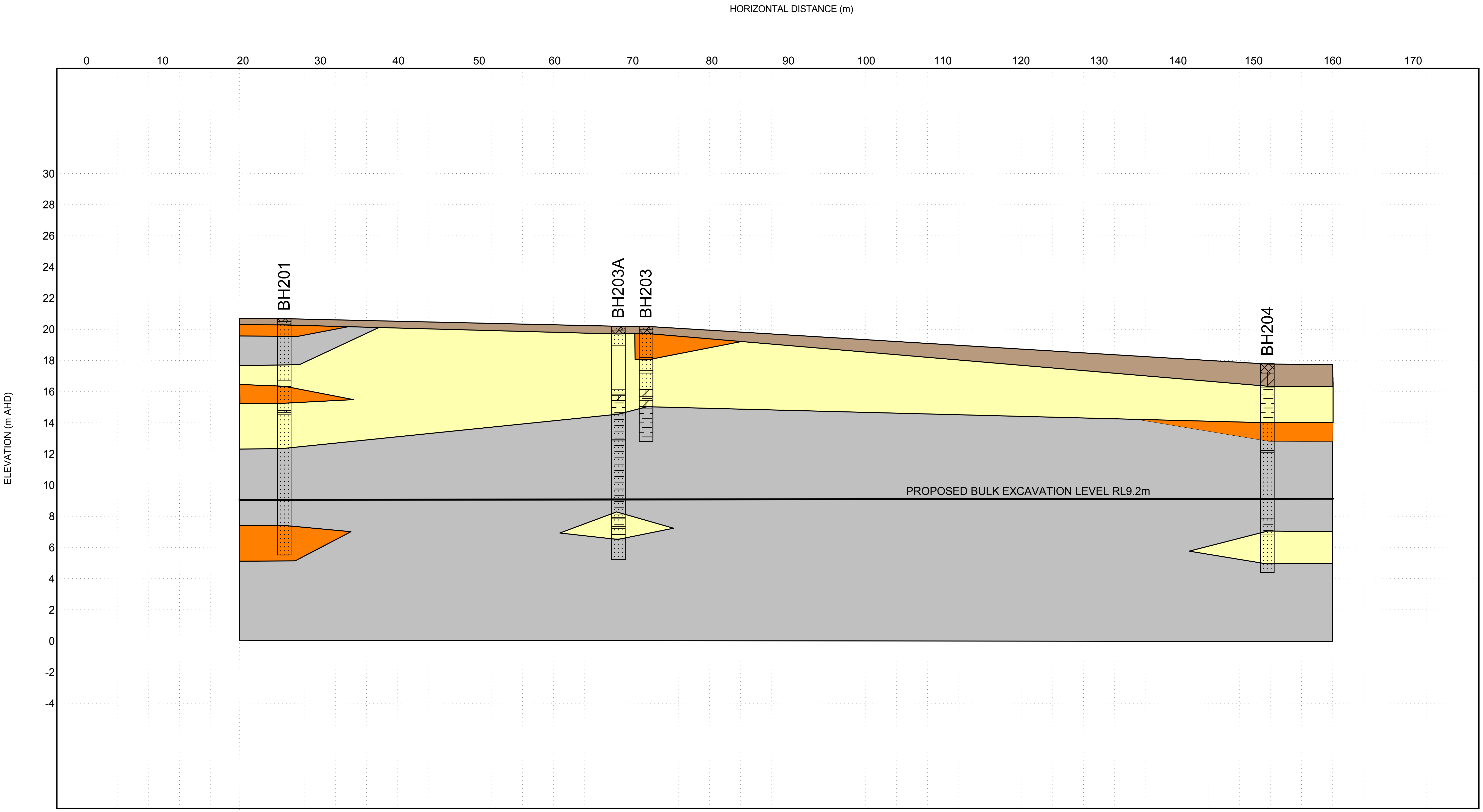
JOB NUMBER

25077ZH

FIGURE NUMBER

7

JK_LIB_CURRENT - V7.3.GLB Fence FENCE A3L NO PLAN 25077ZH FRESHWATER.GPJ 25077ZH FIG7 SOUTH EAST FACE REVB.GDW 12/03/2015 17:17 Produced by gINT Professional, Developed by Datgel



LEGEND

- ☐ CORE LOSS
- ▨ SAND
- ▨ SILTY CLAY
- ▨ CONCRETE
- ☒ FILL
- ▨ SANDSTONE
- ▤ SHALE
- 🟨 CLASS V BEDROCK
- 🟧 CLASS IV BEDROCK
- ⬜ CLASS III OR BETTER BEBROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

No.	Amendment Description	Initials	Date
C	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
B	BH201 UNIT BOUNDARIES	A.P.C	20/04/15
A	INITIAL DRAFT	A.P.C	29/03/15

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DESIGNED: A.P.C

REVIEWED: A.Z

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HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMMARY - SECTION A-A

SCALES

H 1:500

V 1:250

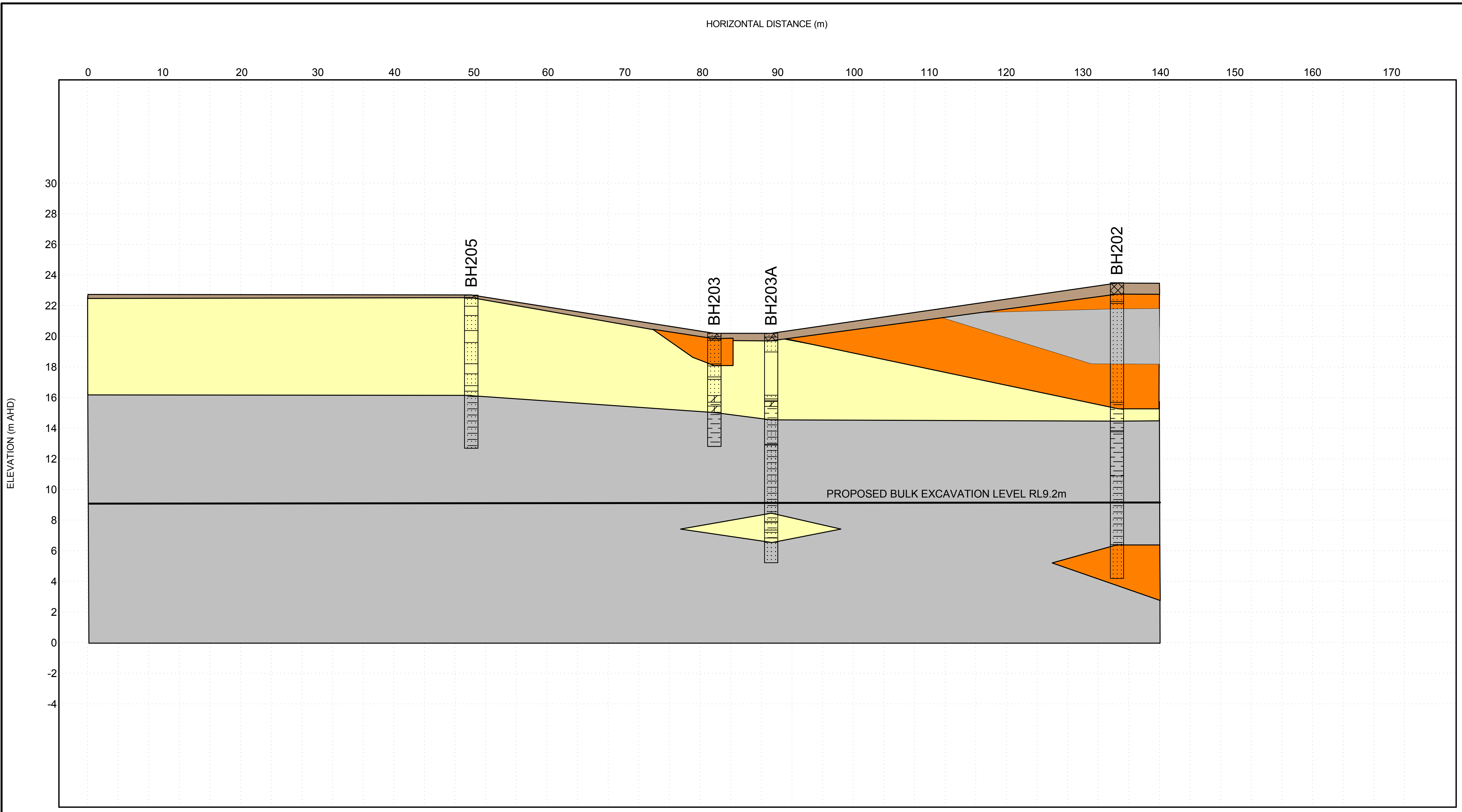
JOB NUMBER

25077ZH

FIGURE NUMBER

8

JK_LIB_CURRENT - V7.3.GLB Fence FENCE A3L NO PLAN 25077ZH FRESHWATER.GPJ 25077ZH3 FIG 8 SECTION A-A REV C.GDW 12/03/2015 12:14 Produced by gINT Professional, Developed by Datgel



LEGEND

- ☐ CORE LOSS
- ☒ FILL
- 🟫 SOIL PROFILE
- ⬜ CLASS III OR BETTER BEBROCK
- ▨ SILTY CLAY
- ⁙ SANDSTONE
- 🟨 CLASS V BEDROCK
- ⁘ CONCRETE
- ☰ SHALE
- 🟧 CLASS IV BEDROCK

NOTES

- UNIT BOUNDARIES ARE APPROXIMATE
- TO BE READ IN CONJUCTION WITH TEXT OF REPORT
- EXAGGERATED VERTICAL SCALE

No.	Amendment Description	Initials	Date
C	AMMDTS TO EXCAVATION LEVEL	A.P.C	17/06/15
B	BH202 UNIT BOUNDARIES	A.P.C	20/04/15
A	INITIAL DRAFT	A.P.C	29/03/15

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REVIEWED: A.Z

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HARBORD DIGGERS
PROPOSED REDEVELOPMENT
EVANS STREET, FRESHWATER, NSW
GRAPHICAL BOREHOLE SUMMARY - SECTION B-B

SCALES

H 1:500

V 1:250

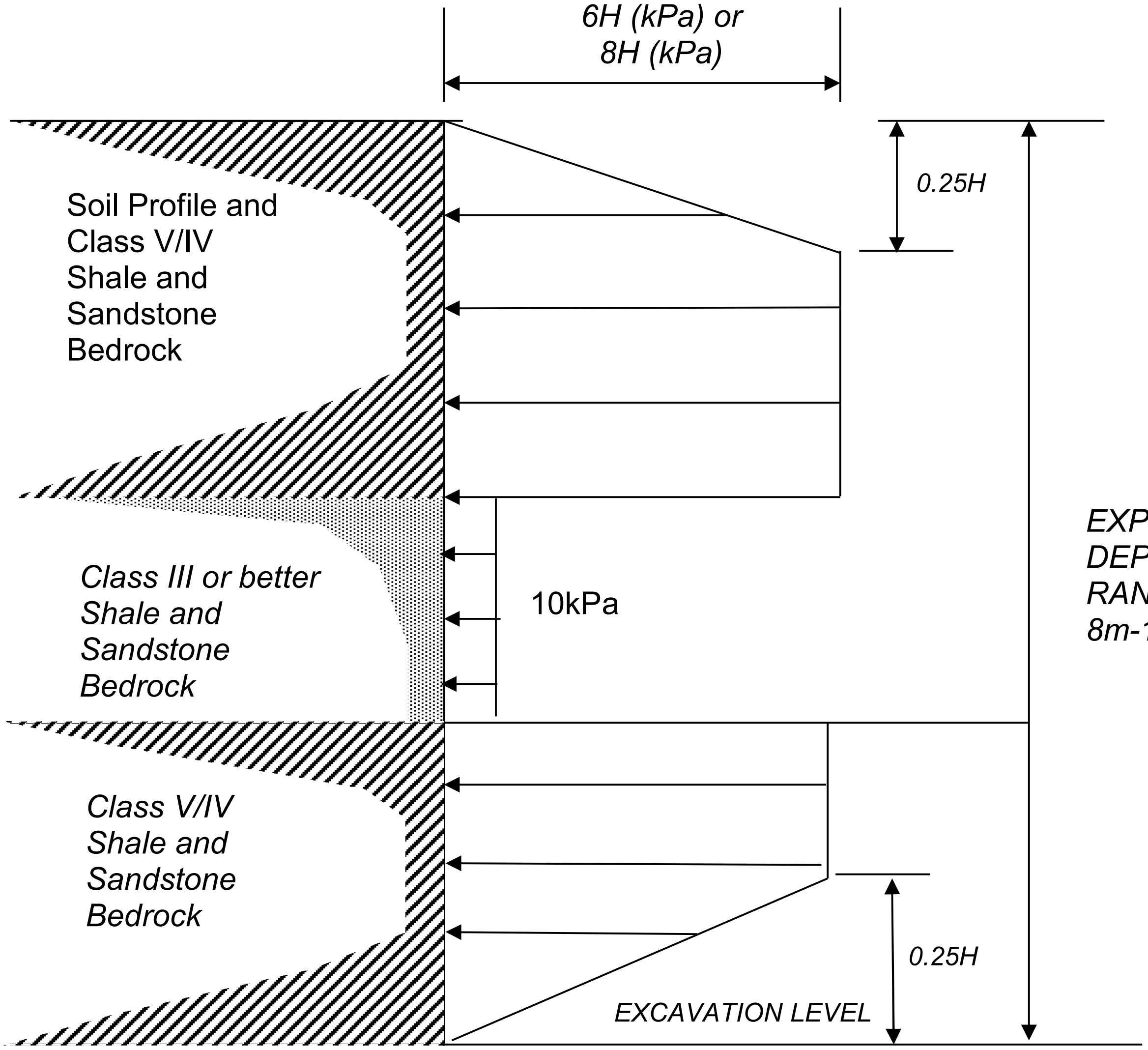
JOB NUMBER

25077ZH

FIGURE NUMBER

9

JK_LIB_CURRENT - V7.3.GLB Fence FENCE A3L NO PLAN 25077ZH FRESHWATER.GPJ 25077ZH3 FIG 9 SECTION B-B REVC.GDW 12/03/2015 14:03 Produced by gINT Professional, Developed by Datgel



NOTES:

1. USE 6H FOR DESIGN WHERE NO MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
2. USE 8H FOR DESIGN WHERE MOVEMENT SENSITIVE STRUCTURES OR SERVICES ARE LOCATED WITHIN H FROM LINE OF EXCAVATION.
3. SURCHARGE AND GROUNDWATER PRESSURES MUST BE ADDED TO THE ABOVE IF APPLICABLE.
4. REFER TO TEXT OF REPORT

RECOMMENDED DESIGN PRESSURES FOR ANCHORED OR PROPPED RETAINING WALLS

Figure 10

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structures.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

NOTE: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

-

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 subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

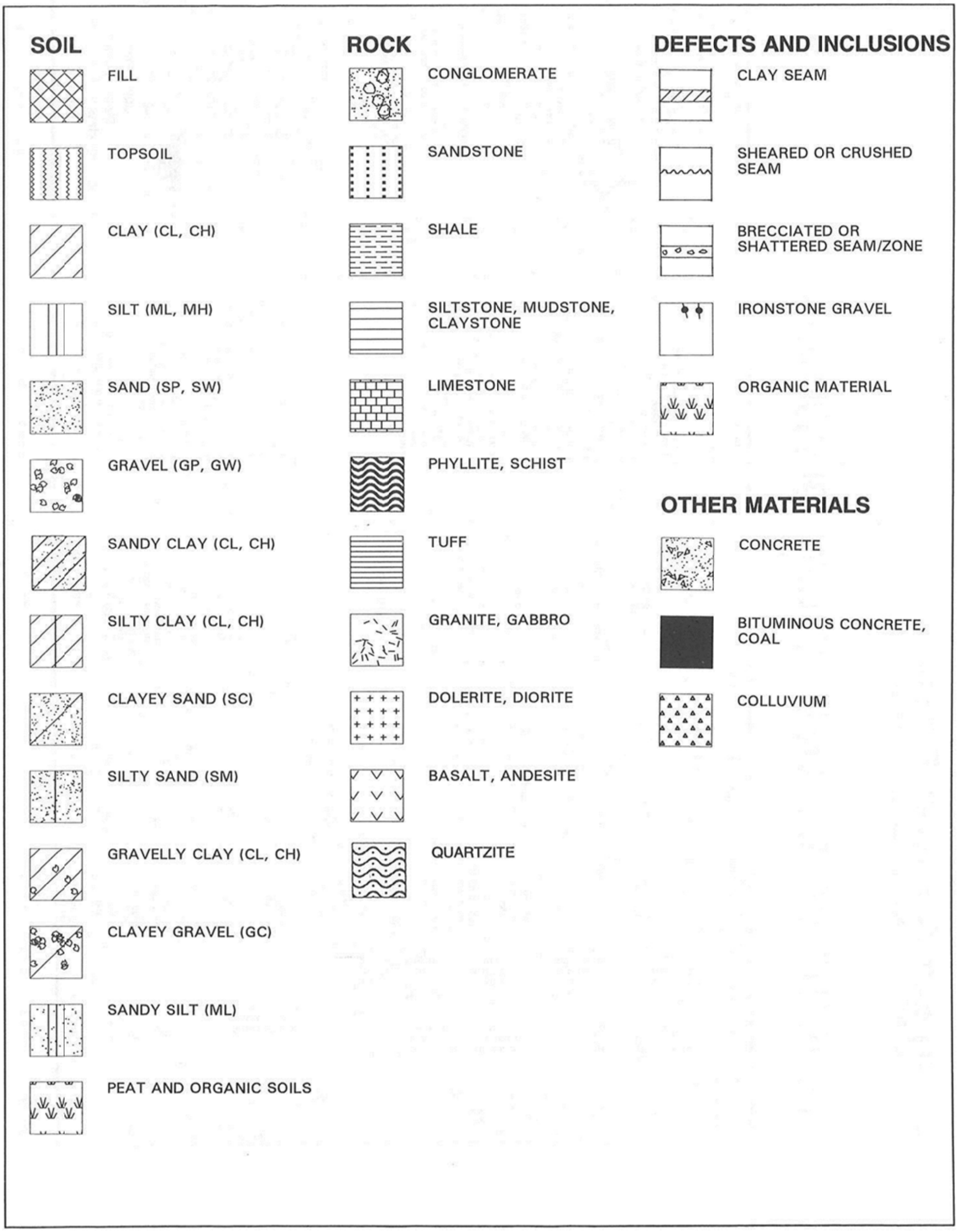
SITE INSPECTION

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

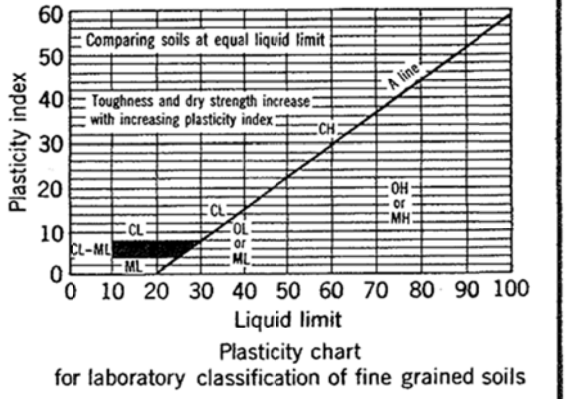
Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)						Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria		
Coarse-grained soils More than half of material is *larger* than 75 µm sieve size[b] (The 75 µm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for *GW*	
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures			Atterberg limits above "A" line, with *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines			$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for *SW*	
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits below "A" line with *PI* greater than 7	
	Identification Procedures on Fraction Smaller than 380 µm Sieve Size										
			Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)						
Fine-grained soils More than half of material is *smaller* than 75 µm sieve size	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)	 Plasticity chart for laboratory classification of fine grained soils		
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays				
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity				
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays				
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity				
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils				

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	—▼— —C— ▶—	Standing water level. Time delay following completion of drilling may be shown. 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 PID = 100	Vane shear reading in kPa of Undrained Shear Strength. Photoionisation detector reading in ppm (Soil 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 'TC' bit T_{60}	Hardened steel 'V' shaped bit. Tungsten carbide wing bit. 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	

APPENDIX A

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 79002

Client:

Jeffery & Katauskas Pty Ltd
PO Box 976
North Ryde BC
NSW 1670

Attention: J Dalberger

Sample log in details:

Your Reference:	**25007ZH2, Freshwater**		
No. of samples:	3 Soils		
Date samples received / completed instructions received	18/09/2012	/	18/09/2012

Analysis Details:

Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	25/09/12	/	21/09/12
Date of Preliminary Report:	Not Issued		

NATA accreditation number 2901. This document shall not be reproduced except in full.
Accredited for compliance with ISO/IEC 17025. **Tests not covered by NATA are denoted with *.**

Results Approved By:

Priya Samarawickrama
Senior Chemist

ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil Our Reference: Your Reference Depth Date Sampled Type of sample	UNITS ------------- ------------	79002-1 BH102 0.5-0.95 11/09/2012 Soil	79002-2 BH104 1.5-1.95 12/09/2012 Soil	79002-3 BH102 1.5-1.95 11/09/2012 Soil
Date prepared	-	20/09/2012	20/09/2012	20/09/2012
Date analysed	-	20/09/2012	20/09/2012	20/09/2012
pH 1:5 soil:water	pH Units	8.8	5.0	8.1
Chloride, Cl 1:5 soil:water	mg/kg	6	210	16
Sulphate, SO4 1:5 soil:water	mg/kg	8	97	240

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			20/09/2012	[NT]	[NT]	LCS-1	20/09/2012
Date analysed	-			20/09/2012	[NT]	[NT]	LCS-1	20/09/2012
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	92%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	[NT]	[NT]	LCS-1	100%

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NA: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batched of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable.

APPENDIX B

LANDSLIDE RISK
MANAGEMENT
TERMINOLOGY

APPENDIX B
LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

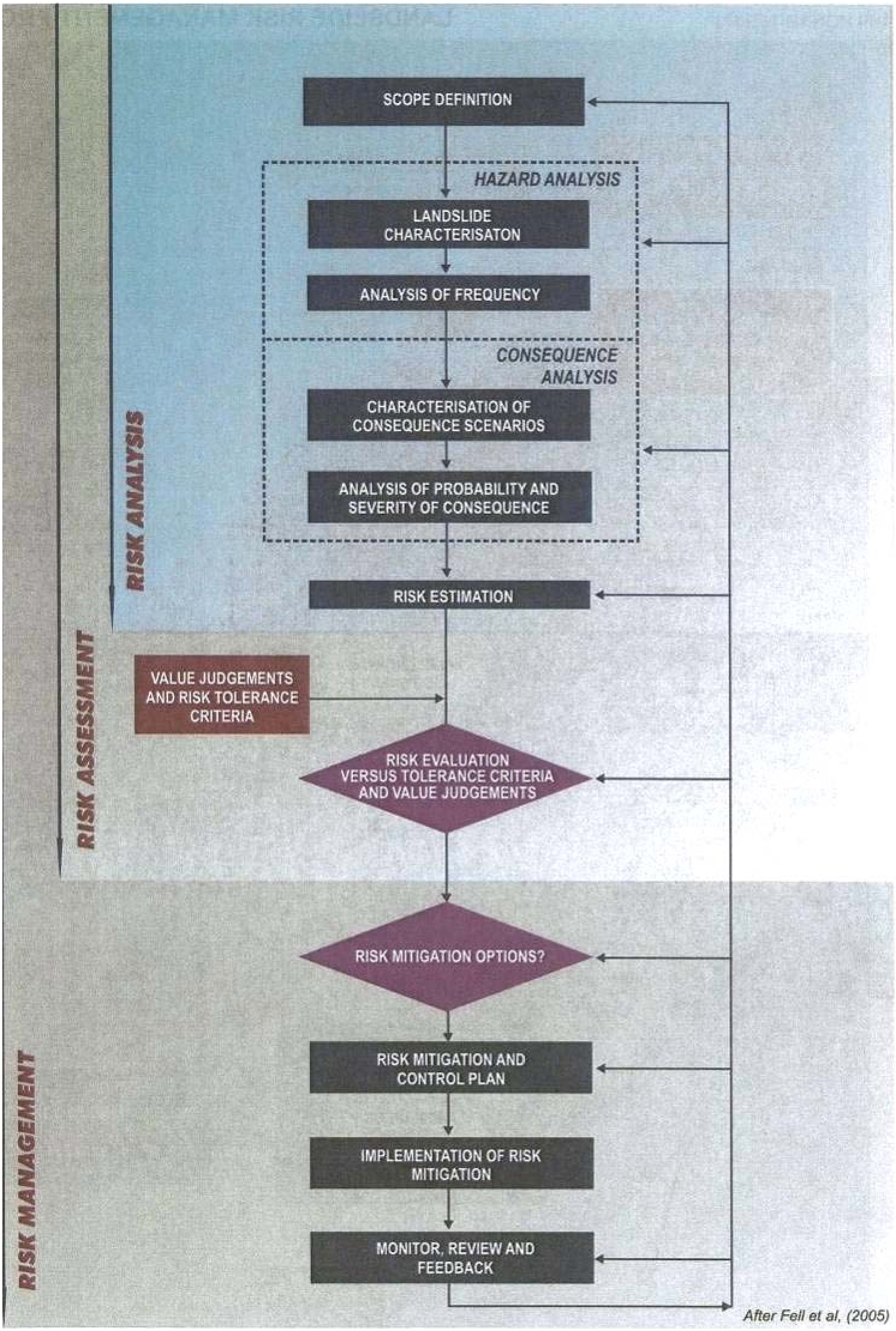
Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
Consequence	The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
Hazard	A condition with the potential for causing an undesirable consequence (the landslide). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.
Individual Risk to Life	The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
Landslide Intensity	A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. These are two main interpretations: (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability ***(continued)***	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

NOTE: Reference should be made to Figure B1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.



After Fell et al, (2005)

FIGURE B1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE B1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	**Notional Boundary**			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE B1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (continued)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		**CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)**				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a “landslide”. Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book “Guideline Document Landslide Hazards” published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board’s website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both “potential” and “actual” landslides must be taken very seriously. The present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with series consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can “run-out” from above, “regress” from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else’s land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

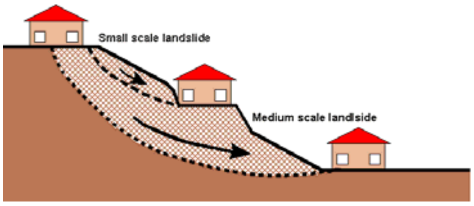


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

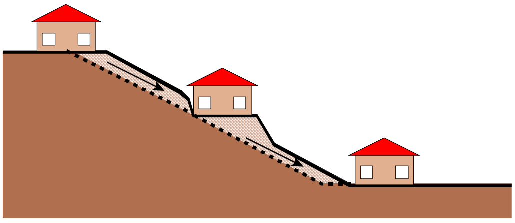


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

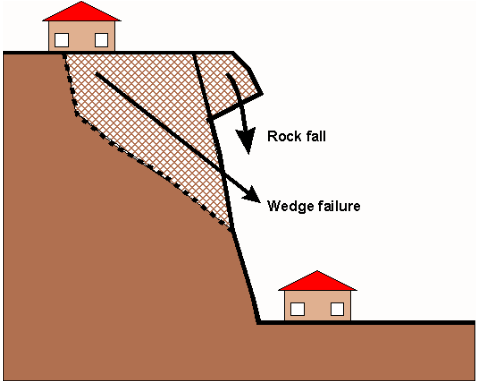


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

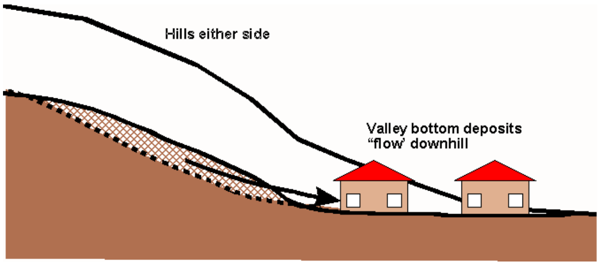


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council. If you have any concern that you could be dealing with a landslide hazard that your local council is not aware of you should seek advice from a geotechnical practitioner.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical

investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction inevitably lacks precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. Likelihood is the chance of it happening in any one year, as indicated in Table 2. Consequences are related to the cost of the repairs and perhaps temporary loss of use. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	**Unacceptable** without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**May be tolerated** in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

TABLE 2 – LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others. Some local councils and planning authorities stipulate a maximum tolerable risk level. This may be lower than you feel is reasonable for your block but it is, nonetheless, a pre-requisite for development. Reasons for this include the fact that a landslide on your block may pose a risk to neighbours and passers-by and that , should you sell, subsequent owners of the block may be more risk averse than you.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding , ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other AUSTRALIAN GEOGUIDES:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
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