Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on Geotechnical Investigation

Proposed Residential Development
124 – 128 Killeaton Street
St Ives

Prepared for
Ausprospect Pty Ltd

Project 84348.00
October 2014

Douglas Partners
Geotechnics / Environment / Groundwater

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
Author		17-10-2014
Reviewer		17.10.2014

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Report on Geotechnical Investigation
Proposed Residential Development
124 – 128 Killeaton Street, St Ives

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed residential development at 124 – 128 Killeaton Street, St Ives. The investigation was commissioned by Develotek Property Group Pty Ltd on behalf of Ausprospect Pty Ltd.

It is understood that the development will include the construction of two 4-storey buildings with a common two level basement. The investigation was carried out to provide information on the subsurface profile for design and planning purposes.

The investigation included the drilling of three rock cored boreholes and the installation of two groundwater monitoring wells for measurement of water levels. Laboratory testing of selected soil and rock core samples was undertaken. Details of the field and laboratory testing are given in this report, together with comments on design and construction issues.

2. Site Description

The site is irregular in shape and covers an area of approximately 4575 m^2 with a northern frontage Killeaton Road and Mona Vale Road. The site slopes gently downwards towards the west from approximately RL157.0 m to RL153.5 m relative to Australian height datum (AHD).

At the time of the investigation the site was occupied by three 2-storey houses with swimming pools and surrounding landscaped and paved areas and scattered mature trees.

On the adjacent properties to the east and south-west are one to two-storey brick houses which are set back approximately 3 m to 8 m from the common boundary.

Open grassed areas of the Corpus Christi Catholic Primary School are located on the property to the south of the site.

3. Geology

Reference to the Sydney 1:100 000 Series Geological Sheet (9130) indicates the site is underlain by Ashfield Shale and is also close to a boundary with Hawkesbury Sandstone. Ashfield Shale typically comprises black to dark grey shale and laminite (interbedded shale, siltstone and fine grained sandstone) and typically weathers to form clays of medium to high plasticity. Hawkesbury Sandstone typically comprises medium to coarse grained quartz sandstone with some shale bands or lenses.

The Mittagong Formation is a transitional unit that can be encountered between Ashfield Shale and Hawkesbury Sandstone and generally comprises interbedded fine grained sandstone and laminite. The results of the investigation indicate that the site is underlain by the Mittagong and Hawkesbury Sandstone formations.

4. Field Work Methods

The field work for the current investigation included three boreholes (BH1 to BH3) which were drilled using a Bobcat-mounted rig on 18 June 2014. The borehole locations are shown on Drawing 1 in Appendix B.

The boreholes were drilled into the weathered rock to depths of 3.4 m to 3.6 m using solid flight augers and rotary drilling methods, and then continued to depths of 9.2 m to 10.5 m using diamond core drilling equipment to obtain continuous core samples of the bedrock.

The boreholes were logged and sampled by a geotechnical engineer. The rock cores recovered from the boreholes were photographed, followed by Point Load Strength Index (Is_{50}) testing on selected samples.

Groundwater monitoring wells (slotted PVC) were installed in BH1 and BH2 and water levels were measured on 19 June 2014.

The ground surface levels at the borehole locations were interpolated from surface levels shown on the survey plan by StrataSurv Pty Ltd (Drawing 360DT-01, Rev A, Dated 4 June 2014)) and are understood to be relative to AHD.

5. Field Work Results

Details of the subsurface conditions encountered in the investigation are given in the borehole logs in Appendix C, together with colour photographs of the rock core and notes defining classification methods and descriptive terms.

The sequence of subsurface materials encountered within the boreholes, in increasing depth order, may be summarised as follows:

Topsoil/Filling: typically silty sand and silty clay to depths of 0.2 m to 0.6 m;

Clay: generally stiff to very stiff clay becoming very stiff to hard silty clay/shaly clay to depths of 3.2 m to 3.5 m;

Laminite: extremely low and very low strength laminite, with medium and high strength ironstone bands, to depths of approximately 5.5 m to 6.2 m.

Sandstone: Generally medium strength sandstone with some extremely low strength bands to depths of 7.2 m to 8.2 m over medium to high strength sandstone.

Free groundwater was not observed during augering of the boreholes to depths of 2.5 m. Groundwater was pumped out of the wells and the water levels then measured in the monitoring wells on 19 June 2014 at depths of 3.4 m (RL 152.9) and 2.7 m (RL 151.7) in BH1 and BH2, respectively.

6. Laboratory Testing

Two samples of clay were provided to DP's laboratory for plasticity and linear shrinkage testing. The results are summarised in Table 1 and indicate the clay is of medium to high plasticity.

Table 1: Summary of Laboratory Classification Test Results

Borehole No.	Sample Depth (m)	Description	Liquid Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
BH1	1.0	Silty clay: very stiff, grey/red brown	47	29	14.5
BH2	2.0	Shaly clay: very stiff, light grey	63	44	20.5

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the relevant bore logs at the appropriate depth. The Is_{50} values for the rock ranged from 0.4 MPa to 2.2 MPa, indicating that the rock tested was of medium and high strength.

7. Geotechnical Model

Two geotechnical cross-sections (Sections A-A and B-B) showing the interpreted subsurface profile are presented as Drawings 2 and 3 in Appendix B. The sections show interpreted geotechnical divisions of underlying soil and rock together with the proposed basement floor level.

It should be noted that the interpreted boundaries shown on the sections are accurate at the borehole locations only and layers shown diagrammatically on these drawings are based on inferred strata boundaries. Reference should be made to the borehole logs for more detailed information and descriptions of the soil and rock.

Groundwater was measured in the monitoring wells at depths of 2.7 m to 3.4 m, which was close to the rock surface. The measured water levels are likely to be associated with perched water flowing along the top of the rock and through fractures and seams in the rock. On-going monitoring of groundwater levels, particularly after heavy rainfall, should continue in order to obtain more information on fluctuations in groundwater levels.

8. Proposed Development

Based on the preliminary architectural drawings by Marchese Partners International Pty Ltd (dated 23/09/2014) it is understood that the proposed development includes two 4-storey residential buildings and a common basement. The lowest basement level varies from RL148.8 m to RL149.3 m and is expected to require excavation to depths of 5.5 m to 7.5 m, with the depth of excavation reducing towards the west. Details of structural loads were not provided at the time of preparing this report.

9. Comments

9.1 Site Preparation and Earthworks

9.1.1 Excavation Conditions

It is expected that the basement will require the excavation of mostly soils and extremely low to very low strength rock with some medium and high strength sandstone in the deeper parts of the excavation. Some bands of high strength ironstone are also expected within the upper weathered rock profile.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of any medium to high strength ironstone bands. It is anticipated that excavation of medium to high strength rock may require moderate to heavy ripping with a large bulldozer. Medium to high strength or stronger rock will probably require hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material.

9.1.2 Dilapidation Surveys

Dilapidation surveys should be carried out on surrounding buildings and pavements that may be affected by the basement construction. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

9.1.3 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of building structure (e.g. reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 "Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)" indicates an acceptable day time limit of 8 mm/s PPVi for human comfort.

Based on the experience of DP and with reference to AS2670, it is suggested that a maximum PPVi of 8 mm/s (applicable at the foundation level of existing buildings) be adopted at this site for both architectural and human comfort considerations, although this vibration limit may need to be reduced if there are sensitive buildings or equipment in the area.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

9.1.4 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (DECCW, 2009). This includes filling and natural materials that may be removed from the site. Accordingly, environmental testing will need to be carried out to classify spoil prior to transport from the site.

9.2 Excavation Support

The most appropriate type of excavation support will vary with subsurface conditions, depth of excavation, set back from site boundaries, and proximity of existing buildings and infrastructure. The proposed basement is generally set back 7 m or more from the site boundaries, with the exception of some sections on the south-western corner which are set back about 4 m from the site boundary. Temporary batters may be possible for most sides of the basement excavation to allow construction of retaining walls, although some shoring will probably be required on the south-western corner of the site.

9.2.1 Batter Slopes and Vertical Rock Excavations

Suggested temporary and permanent batter slopes for unsupported excavations are shown in Table 2. These batters are subject to assessment of jointing in the rock by a geotechnical engineer. If adverse jointing is present in the rock then flatter batters or stabilisation may be required. Also, if surcharge loads (i.e. footings) are applied near the crest of the slope then further geotechnical review and probably flatter batters or stabilisation using rock bolts or soil nails may be required.

Table 2: Recommended Safe Batter Slopes for Exposed Material

Exposed Material	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
Filling / Residual Soils	1.5 : 1	2 : 1
Extremely low to low strength laminite	1 : 1	1.5 : 1
Medium strength or stronger sandstone	Vertical*	Vertical*

Note: * Subject to jointing assessment by experienced Geotechnical Engineer/Engineering Geologist

Extremely low to low strength rock is expected to deteriorate and break down if left exposed to weather. It is therefore recommended that any soil and weathered rock faces that are to be exposed over the long term should be covered with mesh reinforced shotcrete which is pinned to the face with steel dowels. A minimum shotcrete thickness of 80 mm should be adopted unless stability issues dictate a greater thickness is required.

Excavations in medium strength or stronger sandstone will generally be self-supporting (subject to joint orientation) and may be cut vertically. The excavation should be completed in 1.5 m (maximum) drops to allow inspection of the vertical rock faces by a geotechnical engineer, to check for adversely inclined joints and to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcrete of fractured or highly weathered zones or rock bolts/anchors where adverse joints form potentially unstable wedges of rock. Some allowance for shotcrete and stabilisation of extremely low strength bands within the upper medium strength rock profile should be made (as encountered between 6 m to 8.2 m depth in BH2).

9.2.2 Retaining/Shoring Walls

Where required, vertical excavations within the soils and extremely low to low strength rock will require both temporary and permanent lateral support during and after excavation. A bored soldier pile shoring wall with shotcrete infill panels would be suitable where there are no movement sensitive structures in close proximity to the excavation. Typically, soldier piles are installed at a centre to centre spacing of approximately 2.5 m, however, closer spaced piles may be required to reduce wall movements, or prevent collapse of infill materials, where pavements, structures or services are located in close proximity to the excavation.

Preferably, shoring piles should be founded at least 1.0 m below the base of the bulk excavation level (or any perimeter drainage trenches or footings) in order to provide lateral restraint at the base of the excavation and to avoid the risk of adversely inclined joints or wedges undermining the bases of the piles. However, this will require the drilling of piles through medium and high strength sandstone. It may be possible to terminate the shoring piles within unsupported medium to high strength or stronger sandstone above the bulk excavation level, however, it will be important for a geotechnical engineer to assess the stability of the rock directly beneath each pile. The toe of the piles which terminate above bulk excavation level will also need to be restrained with rock bolts or anchors.

It is anticipated that at least one row of anchors will be required to provide lateral restraint to shoring piles for the excavation, particularly in areas where deeper soil is encountered and wall movements must be reduced.

9.2.3 Earth Pressure Design

Design for lateral earth pressures may be based on the parameters given in Table 3. For situations where only minor lateral movements are acceptable, such as the support of sensitive structures or services, an increased pressure based on “at-rest” conditions should be adopted, depending on the level of restraint required.

Table 3: Recommended Design Parameters for Shoring Systems

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (K_a)	At Rest (K_o)		
Filling / Residual Soils	20	0.3	0.5	5	20
Extremely low to low strength laminite	21	0.2	0.3	10	25
Medium strength or stronger sandstone	22	0*	0*	20	38

Note: * Subject to jointing assessment by experienced Geotechnical Engineer/Engineering Geologist

All surcharge loads should be allowed for in the shoring design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 1 m to 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to the stormwater drainage system.

9.2.4 Passive Resistance

Passive resistance for piles founded below the base of the bulk excavation (including allowance for services or footings) may be based on the ultimate passive restraint values provided in Table 4. These ultimate values will need to incorporate a factor of safety to limit the wall movement that is required to mobilise the full passive resistance. The top 0.5 m of the socket should be ignored due to possible disturbance (e.g. over-excavation) and tolerance effects. The passive restraint adopted in the design must not exceed the shear capacity of the pile.

Table 4: Passive Resistance Values

Foundation Stratum	Ultimate Passive Pressure (kPa)
Medium strength sandstone with extremely low strength bands	3,000
Medium to high strength or stronger sandstone	6,000

9.2.5 Ground Anchors

The design of temporary and permanent ground anchors for the support of excavations and/or shoring systems may be carried out on the basis of the maximum allowable bond stresses given in Table 5.

Table 5: Allowable Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Extremely low to very low strength laminite	50	100
Medium strength sandstone with extremely low strength bands	200	400
Medium to high strength or stronger sandstone	500	800

The parameters given in Table 5 assume that the drilled holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring, and "lift-off" tests should be carried out to confirm the anchor capacities. It is suggested that ground anchors should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load.

It is anticipated that the building will support the basement excavation over the long term and therefore the ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection including full column grouting and the use of an internal corrugated sheathing over the full length of the anchor. A detailed specification would need to be prepared for the installation and stressing of permanent anchors.

9.3 Groundwater

It is expected that the groundwater table would be well below the proposed bulk excavation on the site. Seepage should, however, be expected along the top of the rock and through fractures and beddings in the rock, particularly after periods of wet weather.

During construction and in the long term, it is anticipated that seepage into the excavation should be readily controlled by perimeter drains connected to a "sump-and-pump" system. A drained basement will require permanent subfloor drainage below the basement floor slab to direct seepage to the stormwater drainage system.

It is not possible to provide a reliable estimate of the seepage quantity that may be expected within the basement based on the available data. A more usual approach is to monitor the seepage rates during the excavation to assess pumping rates over the longer term.

9.4 Foundations

It is expected that bulk excavation for the basement will expose medium and medium to high strength sandstone over most of the basement footprint and shallow pad footings founded on sandstone should be appropriate.

Recommended maximum pressures for the various rock strata are presented in Table 6. For piles, shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the values for compression. If thick bands of extremely low to very low strength rock are encountered close to the underside of the pad footing excavations then footings will need to be either deepened to found below these weak layers or footings redesigned for reduced bearing pressures in the order of 1000 kPa. This will be subject to advice by a geotechnical engineer at the time of inspection.

Table 6: Recommended Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure	
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)
Medium strength sandstone with extremely low strength bands	2,000	200	5,000	400
Medium to high strength sandstone	3,500	350	20,000	600

Higher allowable bearing pressures of up to 6,000 kPa may be possible on high strength sandstone. Additional rock cored boreholes would be required to confirm the depth to consistent high strength sandstone.

Foundations proportioned on the basis of the allowable bearing pressures in Table 6 would be expected to experience total settlements of less than 1% of the footing width / pile diameter under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

All footing/pile excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing should be carried out in at least one third of the footings which are designed for an allowable end bearing pressure of greater than 3500 kPa. Spoon testing generally involves drilling a 50 mm diameter hole below the base of the footing, to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak/clay bands. If weak seams are detected then footings may need to be taken deeper to reach suitable foundation material.

10. Limitations

Douglas Partners (DP) has prepared this report for the Proposed Residential Development at 124 – 128 Killeaton Street, St Ives, in accordance with DP's proposal dated 4 June 2014 and approval from Develotek Property Group Pty Ltd on behalf of Ausprospect Pty Ltd. The report is provided for the exclusive use of Ausprospect Pty Ltd for this project only and for the purpose(s) described in the report. It should not be used for other projects or by a third party. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions only at the specific sampling or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of anthropogenic influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion given in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instruction for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP.

Douglas Partners Pty Ltd

Appendix A

About this Report

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

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

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- 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 first be washed out of the hole if water measurements are to be made.

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

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

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

About this Report

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

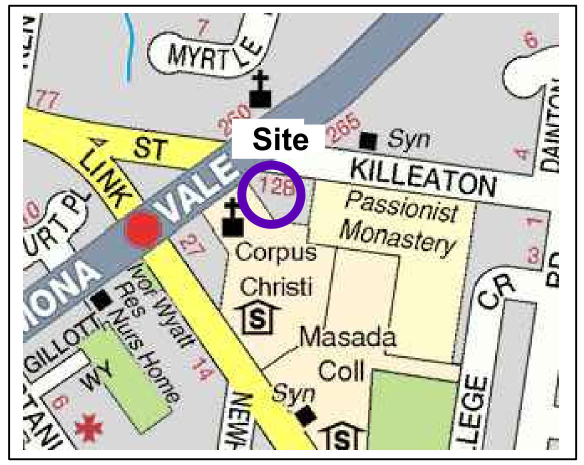
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

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

Appendix B

Drawings



Locality Plan

NOTE: Base drawing from StrataSurv Pty Ltd
(Drawing 3604DT-01, Rev A, dated 4.6.2014)

0 2 4 6 8 10 15 20 30 40m

1:400 @ A3

LEGEND

- Borehole Location
- A — A' Geotechnical Cross Section A-A'
- Basement Outline

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ausprospect Pty Ltd		TITLE: **Location of Boreholes**
OFFICE: Sydney	DRAWN BY: PSCH	**Proposed Residential Development**
SCALE: As shown	DATE: 16.10.2014	**124-128 Killeaton Street, ST IVES**

PROJECT No:	84348
DRAWING No:	1
REVISION:	0

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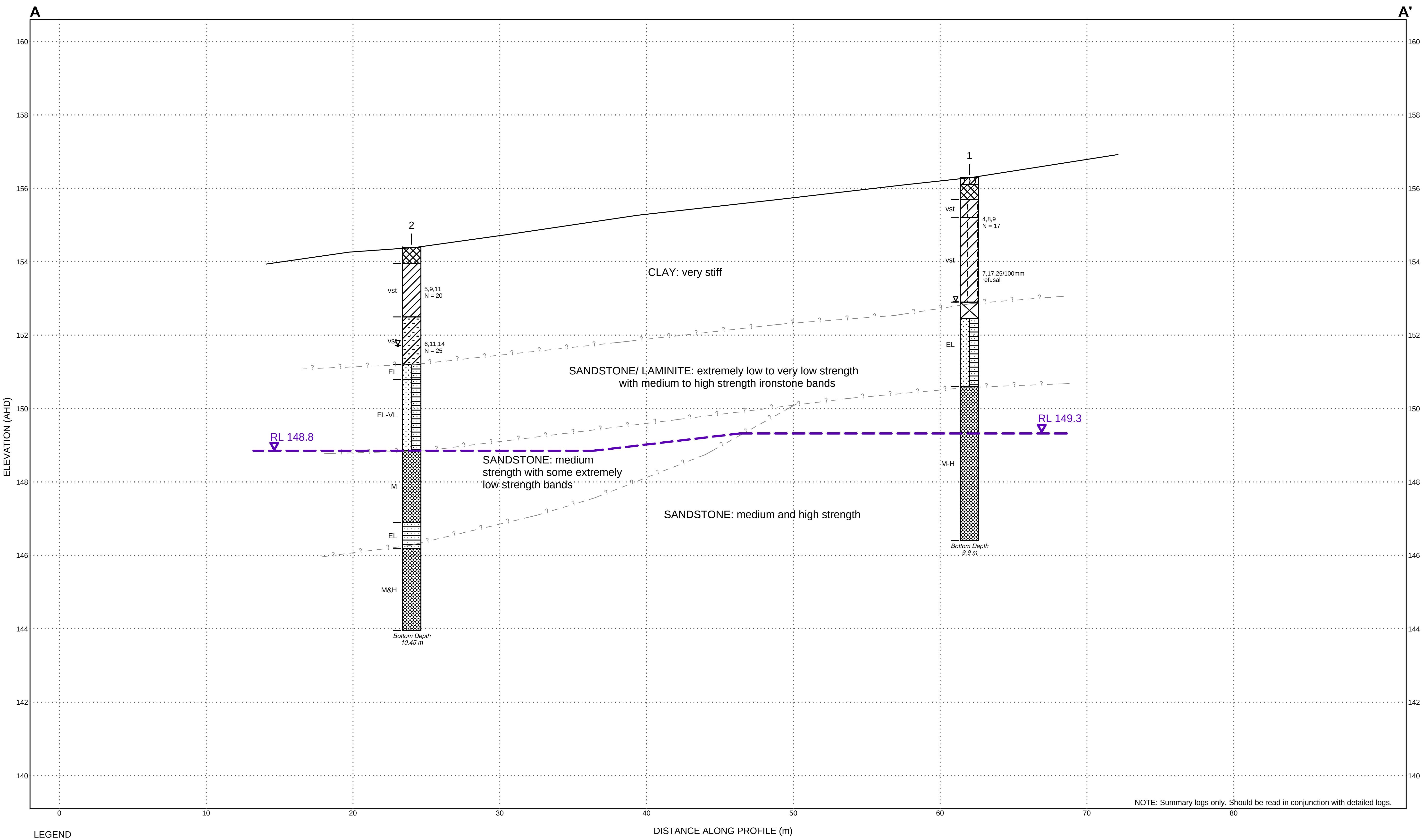

LEGEND

Core Loss	Sandstone coarse grained
Clay	Sandstone fine grained
Filling	Shaly Clay
Laminite	Silty Clay
Topsoil	Basement Level

ROCK STRENGTH
- EL - Extremely Low
- VL - Very Low
- L - Low
- M - Medium
- H - High
- VH - Very High

SOIL CONSISTENCY
- vs - very soft
- s - soft
- f - firm
- st - stiff
- vst - very stiff
- h - hard
- vl - very loose
- l - loose
- md - medium dense
- d - dense
- vd - very dense

TESTS / OTHER
- N - Standard penetration test value
- Water level

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Geotechnics / Environment / Groundwater

CLIENT: Ausprospect Pty Ltd		TITLE: **Interpreted Geotechnical Cross Section A-A'**	PROJECT No: 84348
OFFICE: Sydney	DRAWN BY: STE/LD	**Proposed Residential Development**	DRAWING No: 2
SCALE: 1:250 (H) 1:100 (V) @ A3	DATE: 16.10.2014	**124-128 Killeaton Street, St Ives**	REVISION: 0

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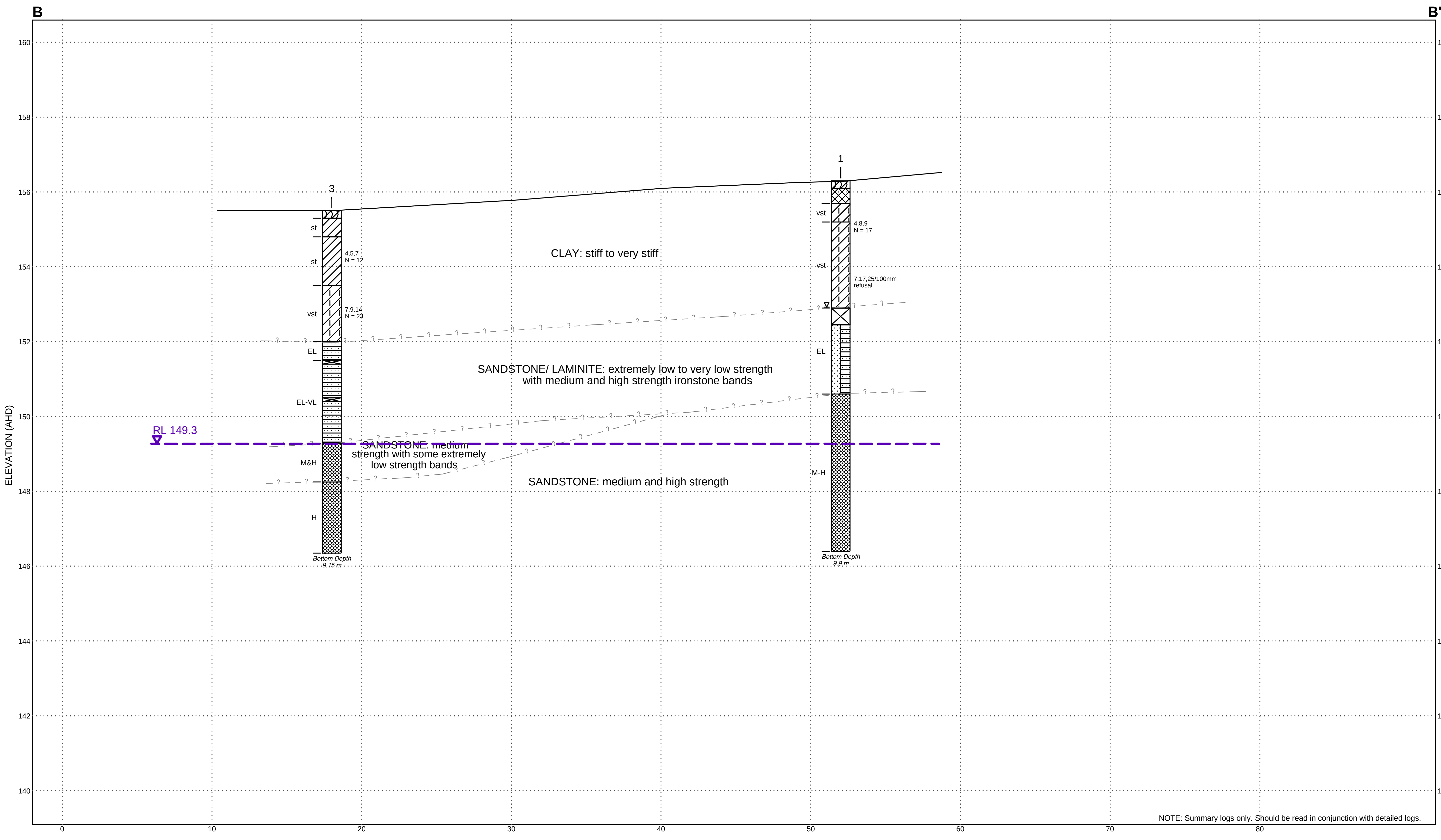

LEGEND

Core Loss	Sandstone coarse grained	Basement Level
Clay	Sandstone fine grained	
Filling	Silty Clay	
Laminite	Topsoil	

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

SOIL CONSISTENCY
vs - very soft
s - soft
f - firm
st - stiff
vst - very stiff
h - hard
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
▽ - Water level

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Ausprospect Pty Ltd	TITLE: **Interpreted Geotechnical Cross Section B-B'**	PROJECT No: 84348
	OFFICE: Sydney — DRAWN BY: STE/LD	**Proposed Residential Development**	DRAWING No: 3
	SCALE: 1:250 (H) 1:100 (V) @ A3 — DATE: 16.10.2014	**124-128 Killeaton Street, St Ives**	REVISION: 0

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Appendix C

Results of Field Work

Sampling

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This 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.

Description and Classification Methods

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

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and loner sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock uality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100\text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

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

Symbols & Abbreviations

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Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core Drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

Asphalt

Road base

Concrete

Filling

Soils

Topsoil

Peat

Clay

Silty clay

Sandy clay

Gravelly clay

Shaly clay

Silt

Clayey silt

Sandy silt

Sand

Clayey sand

Silty sand

Gravel

Sandy gravel

Cobbles, boulders

Talus

Sedimentary Rocks

Boulder conglomerate

Conglomerate

Conglomeratic sandstone

Sandstone

Siltstone

Laminite

Mudstone, claystone, shale

Coal

Limestone

Metamorphic Rocks

Slate, phyllite, schist

Gneiss

Quartzite

Igneous Rocks

Granite

Dolerite, basalt, andesite

Dacite, epidote

Tuff, breccia

Porphyry

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BORE 1 PROJECT 84348 JUN 2014

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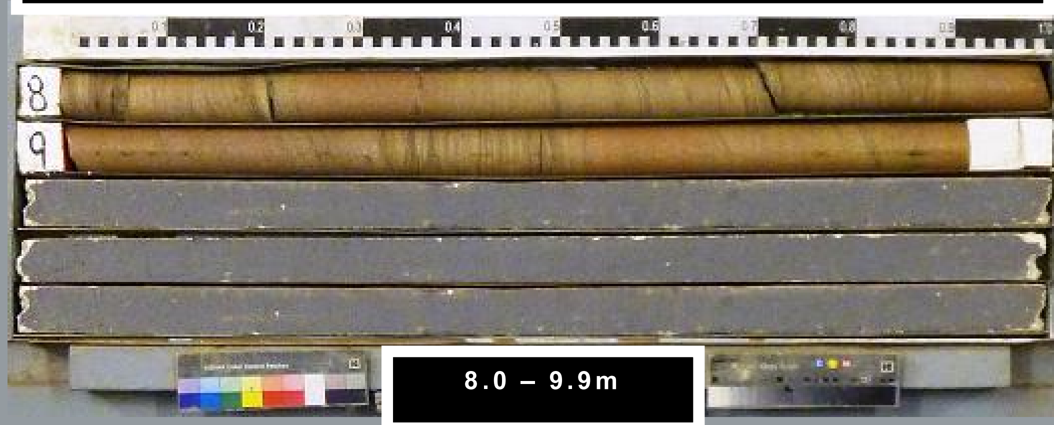
BORE 1 PROJECT 84348 JUN 2014



BOREHOLE LOG

CLIENT: Ausprospect Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 124-128 Killeaton Street, St Ives

SURFACE LEVEL: 156.3 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 84348
DATE: 18/6/2014
SHEET 1 OF 1

Depth (m)	Description of Strata	Water	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.2	TOPSOIL - brown, silty clay topsoil with some rootlets			A			
0.6	FILLING - brown mottled red-brown, silty clay filling with a trace of rootlets			A			
1.1	SILTY CLAY - very stiff, brown, silty clay with some ironstone gravel, Mc>PL			A			
	SILTY CL AY - very stiff, grey mottled red-brown, silty clay with some ironstone bands, Mc>PL			S			4,8,9 N = 17
			Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			7,17,25/100mm refusal
3.4		▼ 19-06-14	3.4m: CORE LOSS: 450mm				
3.85	SANDSTONE/LAMINITE - extremely low strength, extremely and highly weathered, fractured and slightly fractured, light grey and red-brown, fine grained sandstone/laminite. Some medium and high strength ironstone bands		3.96-4.02m: Ds 4.09-4.45m: B (x5) 0°-5°, fe, cly, 5-20mm	C	58	0	PL(A) = 2.2 PL(A) = 0.9
			4.5-4.88m: Ds 4.88m: J50°, un, ro, fe, cly 5.07m: B5°, fe, cly, 5mm 5.22-5.7m: Ds	C	100	0	
5.7	SANDSTONE - medium then high strength, slightly weathered and fresh, slightly fractured and unbroken, light grey and light grey-brown, medium to coarse grained sandstone		5.7m: J30°, un, ro, cly 6.7m: B5°, cly, 10mm 6.9m: B10°, cly	C	100	96	PL(A) = 1.2 PL(A) = 0.9
			8.07m: B5°, cly, 5mm 8.2m: B5°, cly co, 1mm 8.7m: B20°, cly vn, ti	C	100	99	PL(A) = 1.5 PL(A) = 1.2 PL(A) = 1.6
9.9	Bore discontinued at 9.9m						

RIG: Bobcat **DRILLER:** LC **LOGGED:** SI/JMS **CASING:** HW to 2.8m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.4m; NMLC-Coring to 9.9m

WATER OBSERVATIONS: No free groundwater observed whilst augering. Water measured in standpipe at 3.4m on 19/6/14

REMARKS: Standpipe installed to 9.9m (Screen 3.9-9.9m; Gravel 3.0-9.9m; Bentonite 2.6-3.0m; Backfill to GL with Gatic cover)

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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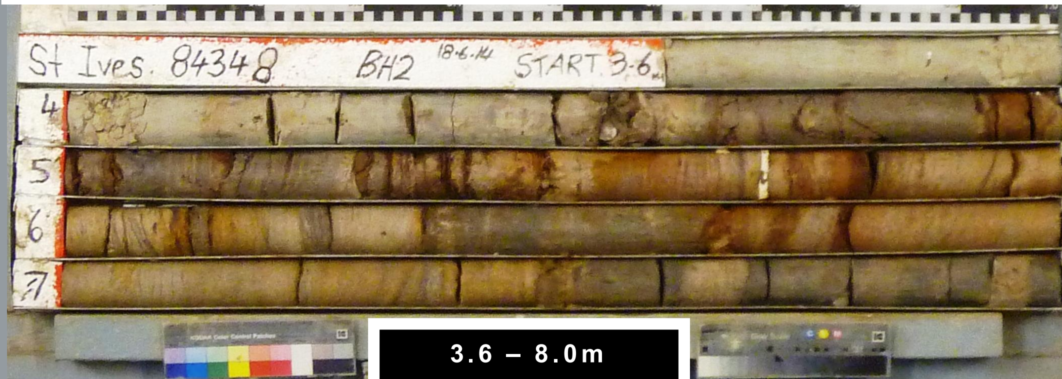
PROPOSED RESIDENTIAL DEVELOPMENT – ST IVES

BORE 2 PROJECT 84348 JUN 2014



3.6 – 8.0m

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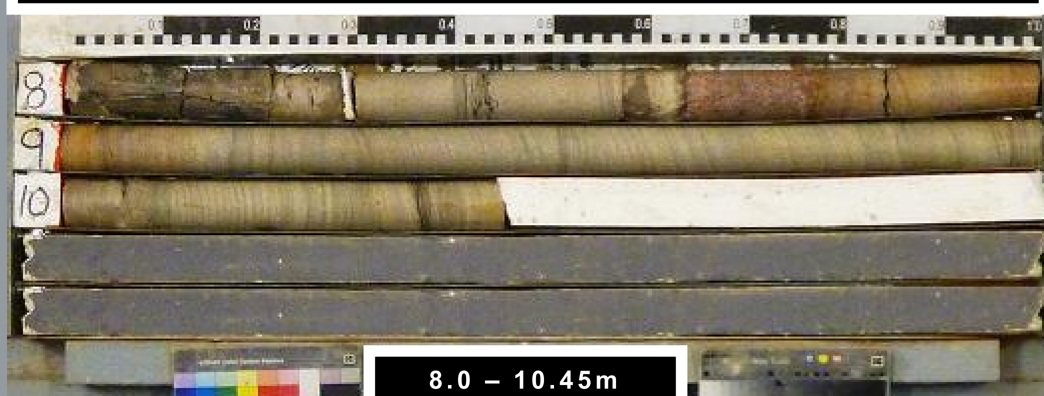
PROPOSED RESIDENTIAL DEVELOPMENT – ST IVES

BORE 2 PROJECT 84348 JUN 2014



8.0 – 10.45m

BOREHOLE LOG

CLIENT: Ausprospect Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 124-128 Killeaton Street, St Ives

SURFACE LEVEL: 154.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 84348
DATE: 18/6/2014
SHEET 1 OF 2

Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0–0.45	FILLING - grey to grey brown, fine grained silty sand filling with a trace of grass roots			A			
0.45–1.9	CLAY - very stiff, mottled brown-light grey, clay with a trace of ironstone gravel, moist			A A S			5,9,11 N = 20
1.9–3.2	SHALY CLAY - very stiff, light grey, shaly clay, damp	19-06-14	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			6,11,14 N = 25
3.2–3.6	SANDSTONE/LAMINITE - extremely low strength, extremely weathered, light grey, fine grained sandstone						
3.6–5.53	SANDSTONE/LAMINITE - extremely low then very low strength, extremely to highly weathered, slightly fractured and fractured, light grey and grey-brown, fine grained sandstone/laminite		4.5m: J85°, pl, ro, cly 4.65m: B0°, fe, cly, 10mm 4.78m: J70°, pl, sm, cly 4.94-4.98m: B0°, fe 5.1m: J70°, un, ro, fe, cly 5.3-5.4m: fg, fe, cly 5.5m: B5°, fe, cly, 30mm	C C	100 100	0 15	pp = 490 pp = 520 PL(A) = 0.9
5.53–7.5	SANDSTONE - medium strength, moderately weathered, slightly fractured, grey and light grey-brown, medium grained sandstone. Some extremely low strength bands		5.8m: B5°, fe 6.13m: B0°, fe, cly, 10mm 6.25m: B0°, cly, 15mm 6.37m: B0°, fe, Ds, 250mm 6.67m: B10°, fe, cly 6.78m: J35°, pl, ro, fe	C	100	82	PL(A) = 0.5 PL(A) = 0.4 PL(A) = 0.4
7.5–8.22	LAMINITE - extremely low strength, extremely weathered, slightly fractured, grey laminite with medium strength sandstone bands		7.5-7.62m: Ds 7.7-7.93m: Ds 7.93m: J80°, cu, ro, cln 7.98-8.22m: Ds	C	100	30	
8.22–	SANDSTONE - medium and high strength, slightly weathered and fresh, slightly fractured and unbroken, light grey and light grey-brown, medium grained sandstone		8.58-8.63m: Ds	C	100	97	PL(A) = 0.8 PL(A) = 1.2 PL(A) = 0.8

RIG: Bobcat **DRILLER:** LC **LOGGED:** SI **CASING:** HW to 2.7m

TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.6m; NMLC-Coring to 10.45m

WATER OBSERVATIONS: No free groundwater observed whilst augering. Water measured in standpipe at 2.7m on 19/6/14

REMARKS: Standpipe installed to 10.45m (Screen 4.45-10.45m; Gravel 4.0-10.45m; Bentonite 3.5-4.0m; Backfill to GL with Gatic cover)

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Ausprospect Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 124-128 Killeaton Street, St Ives

SURFACE LEVEL: 154.4 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 84348
DATE: 18/6/2014
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
144	10.45	SANDSTONE - medium and high strength, slightly weathered and fresh, slightly fractured and unbroken, light grey and light grey-brown, medium grained sandstone *(continued)*						10.06-10.09m: Ds	C	100	97	PL(A) = 0.7
		Bore discontinued at 10.45m										

RL scale: 144, 143, 142, 141, 140, 139, 138, 137, 136, 135
Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19

RIG: Bobcat **DRILLER:** LC **LOGGED:** SI **CASING:** HW to 2.7m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 3.6m; NMLC-Coring to 10.45m
WATER OBSERVATIONS: No free groundwater observed whilst augering. Water measured in standpipe at 2.7m on 19/6/14
REMARKS: Standpipe installed to 10.45m (Screen 4.45-10.45m; Gravel 4.0-10.45m; Bentonite 3.5-4.0m; Backfill to GL with Gatic cover)

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BORE 3 **PROJECT 84348** **JUN 2014**

4.0 – 8.0m

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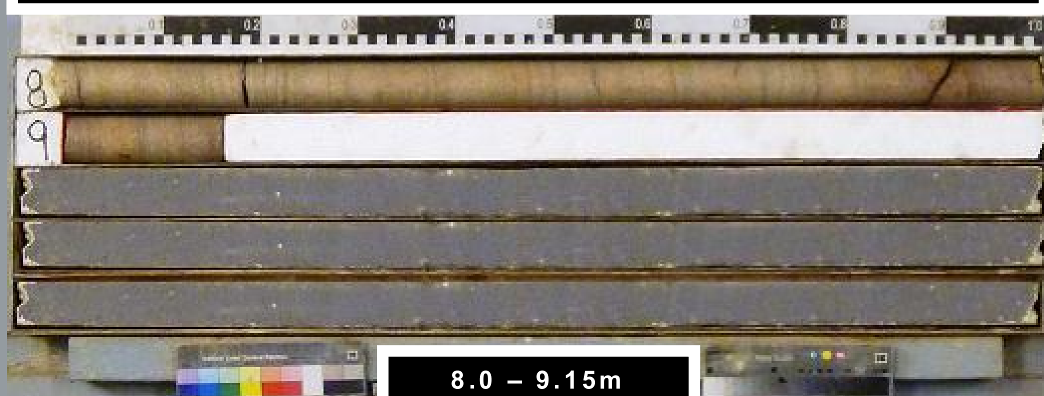
PROPOSED RESIDENTIAL DEVELOPMENT – ST IVES

BORE 3 **PROJECT 84348** **JUN 2014**



8.0 – 9.15m

BOREHOLE LOG

CLIENT: Ausprospect Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 124-128 Killeaton Street, St Ives

SURFACE LEVEL: 155.5 AHD
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 84348
DATE: 19/6/2014
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.2	TOPSOIL - grey to dark grey, silty clay topsoil with some fine sand and grass rootlets, moist		A			
0.7	CLAY - stiff, light brown clay, moist		A			
	CLAY - stiff, mottled brown-light grey, clay with a trace of ironstone gravel, moist		A			
			S			4,5,7 N = 12
2.0	SILTY CLAY - very stiff, light grey silty clay, moist		S			7,9,14 N = 23
3.5	LAMINITE - extremely low strength, light grey to grey laminite	Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°				
4.0 / 4.1	LAMINITE - extremely low then very low strength, highly weathered, slightly fractured, light grey to grey laminite	4m: CORE LOSS: 100mm	C	82	0	pp >600
5.1		5m: CORE LOSS: 100mm 5.7m: B0°, cly 5.9m: B0°, fe 6.06m: B0°, Ds, 100mm	C	98	0	pp >600 PL(A) = 1.7
6.22	SANDSTONE - medium and high strength, slightly weathered, slightly fractured, light grey and brown, medium grained sandstone. Some extremely low strength bands	6.2m: CORE LOSS: 20mm 6.27m: B5°, fe, cly 6.97m: B10°, fe, cly 7.04m: B0°, fe, Ds, 50mm	C	100	94	PL(A) = 0.4
7.25	SANDSTONE - high strength, slightly weathered, slightly fractured and unbroken, light grey-brown, medium to coarse grained sandstone	7.25m: J35°, un, ro, cly 7.45m: B0°, fe 8.85m: J45°, pl, ro, fe				PL(A) = 1.1 PL(A) = 1.8 PL(A) = 1.1
9.15	Bore discontinued at 9.15m					

RIG: Bobcat **DRILLER:** LC **LOGGED:** SI **CASING:** HW to 2.7m
TYPE OF BORING: Solid flight auger to 2.5m; Rotary to 4.0m; NMLC-Coring to 9.2m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
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Appendix D

Results of Laboratory Testing

Douglas Partners
Geotechnics / Environment / Groundwater

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Results of Moisture Content, Plasticity and Linear Shrinkage Tests

Client:	**Ausprospect Pty Ltd**	**Project No:**	84348
		Report No:	1
Project:	ST IVES Proposed Residential Development	**Report Date:**	31/07/2014
		Date Sampled:	18/06/2014
		Date of Test:	29/07/2014
Location:	124-128 Killeaton Steet, St Ives	**Page:**	1 of 1

Test Location	Depth (m)	Description	Code	W_F %	W_L %	W_P %	PI %	*LS %
BH1	1.0-1.45	Orange brown silty clay	2,5	18.3	47	18	29	14.5 CU
BH2	2.0-2.5	Light grey silty clay	2,5	17.8	63	19	44	20.5 CU

Legend:

W_F	Field Moisture Content
W_L	Liquid limit
W_P	Plastic limit
PI	Plasticity index
LS	Linear shrinkage from liquid limit condition (Mould length125mm)

Code:

Sample history for plasticity tests

1. Air dried
2. Low temperature (<50ºC) oven dried
3. Oven (105ºC) dried
4. Unknown

Method of preparation for plasticity tests

5. Dry sieved
6. Wet sieved
7. Natural

*Specify if sample crumbled CR or curled CU

Test Methods:

Moisture Content:	AS 1289 2.1.1
Liquid Limit:	AS 1289 3.1.2
Plastic Limit:	AS 1289 3.2.1
Plasticity Index:	AS 1289 3.3.1
Linear Shrinkage:	AS 1289 3.4.1

Sampling Methods: Sampled by Engineering Department

Remarks:

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

NATA Accredited Laboratory Number: 828

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards.
Accredited for compliance with ISO/IEC 17025

Tested: LW
Checked: MM

Mark Matthews
Laboratory Manager