ANNEXURE 4

Douglas Partners
Geotechnics • Environment • Groundwater

REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED DEVELOPMENT
1112 - 1118 BARRENJOEY ROAD
PALM BEACH

Prepared for
LESIUK ARCHITECTS

Project 36684 02
October 2009

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REPORT ON GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
1112 – 1118 BARRENJOEY ROAD, PALM BEACH

1 INTRODUCTION

This report details the results of a geotechnical investigation carried out for a proposed development at 1112 - 1118 Barrenjoey Road, Palm Beach The investigation was carried out at the request of Dr Stephen Lesiuk of Lesiuk Architects

It is understood that the existing site structures (shops, residence and car park area) are to be demolished and replaced by a mixed commercial residential development The development will comprise commercial space at road level with three residential levels above and a basement car park requiring excavation into the existing slope and to about 2 5 m below the level of the existing car parking area on the southern side of the site at Barrenjoey Road

The assessment was carried out to provide information on subsurface conditions for Development Application purposes and for the preliminary assessment for site works and design The original field work and report was carried out in December 2003 and comprised a detailed geological inspection of the site and accessible adjacent areas to the south along Barrenjoey Road, general inspection of slopes in the immediate vicinity and a test bore with an adjacent dynamic penetrometer test in the existing car park area

Since 2003 the layout of the development has been slightly modified and this report updates the geotechnical report to be in accordance with the requirements of Pittwater's Geotechnical Risk Management Policy (GRMP - 2009) laminite and lithic to quartz lithic sandstone The upper slopes and crest of the Palm Beach ridge to the east are capped by the Hawkesbury Sandstone

3 FIELD WORK

The field investigation comprised,

- detailed geological inspection of the site and adjacent areas by a senior engineering geologist on 16 December 2003,
- a hand auger test bore (Bore 1) taken to 1 8 m depth, and a dynamic cone penetrometer test (DCP) taken to refusal at 2 12 m depth, at the boree location
- an inspection of the site on 21 August 2009, to confirm the current conditions

The significant geotechnical site features and location of the tests are shown on Drawing 1 and were determined by tape measurement from site boundaries and existing features The surface levels shown were determined by interpolation and estimation from the site survey plan provided

3 1 Site Observations

The significant observations made during inspection of the site and the surrounding area are that

- a the core filled, concrete block retaining wall dividing the site (refer Drawing 1) is in good condition with no cracking noted
- on the southern side of the timber residence there is a minor unsupported cut about 1 5 m high which has exposed a profile of approximately 1 m of colluvium overlying highly weathered, low strength, shaly sandstone/ sandy siltstone (refer Photo 6 and Drawing 1)
- to the south of this area there is a 100 mm diameter PVC pipe (the upslope origin of which is not known) extending down to the central retaining wall
- at the upper edge of the right of way along the eastern boundary the slope of the ground surface of the adjacent eastern properties results in generally uncontrolled surface water

runoff onto the subject site (refer Photo 5) A grated pit is evident, however the discharge pipe alignment was not apparent At least one additional PVC pipe is noted at the crest of the bamboo-covered slope

- on the adjacent southern property, there is a sewer main which entered the subject site through the southern concrete block retaining wall (Drawing 1)
- on the adjoining southern property there is a very steep grass-covered batter at about 45° (refer Photo 2) which includes a surficial slump about 5 m wide and about 0 5 m to 1 m deep
- approximately 80 m and 150 m to the south of the site there were a number of very large sandstone floaters within the lower portion of the slope (refer Photo 3)
- behind the residences at 1096 and 1098 Barrenjoey Road (refer Photo 6) low to medium strength, shaly sandstone/sandy siltstone is exposed to a height of approximately 6 m above road level Above this level, the rock comprises medium to thickly bedded sandstone as exposed beneath 1094 Barrenjoey Road and in the sites opposite the Iluka Road shops (refer Photos 4 and 7)
- to the south along Barrenjoey Road (about 1080 Barrenjoey Road), siltstone is exposed in the batter/slope at the rear of the properties to about 6 m above ground level and was in turn overlain by sandstone with some shaly beds (refer Photos 4 and 8)

3 2 Results

Details of the conditions encountered in the bore (Bore 1) and the DCP result are presented in Appendix A together with notes defining classification methods and descriptive terms

In summary, the test bore encountered gravel, asphaltic concrete and sandy clay filling to 0 25 m depth overlying fine grained silty sand to 0 6 m, then brown fine to medium grained sand to 1 8 m depth (Photo 9) Free groundwater was encountered at 1 7 m and thereafter it was not possible to drill further and recover samples

The DCP test encountered clearly defined refusal at 2 12 m depth with the hammer bouncing on the anvil suggesting the presence of rock of at least low strength

4 LABORATORY TESTING

The samples collected from the test bore were screened in the laboratory by measurement of pH after addition of distilled water (pH_F) and peroxide (pH_{FOX}) These screening tests give an approximate indication of either actual acid sulphate soils (AASS) or potential acid sulfate soils (PASS) conditions and the results are presented in Table 1

Table 1 - Summary of Acid Sulphate Soil Testing

Sample Location	Depth (m)	Material Description	Screening Tests	
			Natural pH_F	Oxidised pH_{FOX}
BH1	0 4	Dark brown/grey Silty Sand	7 80	7 36
	1 0	Orange brown Sand (slightly silty)	7 54	7 16
	1 5	Orange brown Sand (slightly silty)	7 70	7 07

The results indicate no significant change of pH which could be ascribed to acid sulphate soils within the test depth range The results are consistent with the mapping indicated on the Broken Bay Acid Sulfate Soil Risk map which indicates that there is no known occurrence of acid sulphate soils (ASS)

5 COMMENTS

5 1 Proposed Development

It is understood that the existing shops and residence are to be demolished and a new mixed residential/commercial development constructed The building will be four storey with commercial use on the ground level three residential levels above and a basement carpark The development will include excavation into the existing slope and to about 2 m below the level of the existing car parking area (proposed basement at RL 0 4) The architectural sections indicate that excavation on the eastern side of the proposed structure is approximately 6 m from the boundary and is to be supported by a concrete retaining wall

The approximate footprint of the development is shown on Drawing 1 and reference should also be made to architectural Drawings DA04 and DA05

5 2 Geological Model and Inferred Type Section

The site is located at the base of a moderately sloping, westerly facing hillside It can be divided into two portions, a lower, relatively level, western portion, and a very steeply sloping eastern portion

The interpreted geological model for the site comprises

- a sandy soil cover (colluvium/slope wash material and marine sediments associated with Snapperman Beach and Pittwater foreshore) to approximately 2 m to 2 5 m depth overlying weathered sandstone and siltstone/shale bedrock within the western portion of the site,
- a variable depth of colluvium (sandy clay and clayey sand) possibly with a thin layer of residual, highly weathered sandy clay mantling a stepped bedrock profile of interbedded sandstone, siltstone shale within the eastern portion of the site

5 3 Risk Assessment

The geotechnical inspection identified no evidence of gross instability on the site or adjoining sites There is, however, evidence of ongoing creep of the upper colluvium and overburden soils, a slump within the colluvium on the adjoining southern property and the presence of large sandstone floaters in the embankment to the south of the site (Photo 3) The floaters indicate detachment of individual blocks from upslope sandstone outcrop, possibly the Hawkesbury Sandstone, at the top of the slope

The inspections of the subject site and the site survey indicates that the slope lessens significantly upslope and that no large sandstone blocks or outcrop of any significance are present at the surface It is therefore assessed that the likelihood of significant rock falls impacting on the property are 'unlikely"

The site has been assessed in accordance with the methods of the Australian Geomechanics Society-2007 (Reference 2) and Pittwater Council's GRMP-2007 Identified slope instability hazards within and above the site are summarised in Table 2, together with a qualitative assessment of likelihood of occurrence after construction (including good engineering practice and specific items detailed later in this report), together with the consequence and risk

Table 2 - Property Risk Assessment for Proposed Development

Hazard	Likelihood	Consequence	Risk
Rapid and significant erosion/slumping of slope on the eastern side of the development as a result of water flows from upslope	Unlikely for properly graded and drained surface runoff control measures and appropriately designed and constructed retaining walls	Minor	Low
Rapid collapse of proposed retaining walls	Rare – for engineer designed inspected and constructed wall	Major	Low
Minor creep effects on landscaping walls	Possible	Insignificant	Very Low
Major rapid soil mass movement	Rare for a site with engineer designed retaining structures and no features (cliff lines undercuts or floaters) observed suggesting incipient instability	Medium	Low

For loss of life, the individual risk can be calculated from

$R_{(LoL)} = P_{(H)} \times P_{(S H)} \times P_{(T S)} \times V_{(D T)}$

where

$R_{(LoL)}$ is the risk (annual probability of loss of life (death) of an individual)

$P_{(H)}$ is the annual probability of the hazardous event

$P_{(S H)}$ is the probability of spatial impact by the hazard

$P_{(T S)}$ is the temporal probability given the spatial impact

$V_{(D T)}$ is the vulnerability of the individual (probability of loss of life of the individual given the impact)

A quantitative assessment of risk of loss of life (person most at risk) related to the identified slope instability hazards is summarised in Table 3

Table 3 - Life Risk Assessment for Proposed Development

Hazard	$P_{(H)}$	$P_{(S H)}$	$P_{(T S)}$	$V_{(D T)}$	$R_{(LoL)}$
Significant erosion/slumping of slope on eastern side of the development as a result of water flows from upslope	1×10^{-4}	0 1	0 1	0 01	1×10^{8}
Rapid collapse of proposed retaining walls and striking of building	1×10^{5}	1	1	0 1	1×10^{6}
Minor creep affects on landscaping walls	1×10^{3}	1	0 01	0 01	1×10^{7}
Major rapid soil mass movement and impact to building	1×10^{5}	1	0 1	0 5	5×10^{7}

When compared to the requirements of the Pittwater GRMP, it is considered that the proposed development will achieve the "Acceptable Risk Management" criteria for both property and life under current and foreseeable conditions and that the site is suitable for the proposed development

It is also considered that, provided the construction is undertaken in accordance with the recommendations contained in this report, the proposed works would not be expected to affect the overall stability of the site or negatively influence any of the geotechnical hazards identified in Tables 2 and 3 Should on site or adjacent site conditions unexpectedly change (e g breakage of either above or below ground drainage services), the risk assessment should be reviewed

5 4 Excavation Conditions

Inspection and investigation has indicated that the site is likely to be underlain by

- pavement materials and sand to about 2 m then rock of at least low strength across the western portion of the site,
- variable depth though relatively shallow soil/ colluvium to about 2 m, then possibly a thin layer of highly weathered rock overlying interbedded sandstone siltstone and shale across the eastern portion of the site

The layout and architectural plans indicate that the development will require excavation to depths of about 2 5 m across the front of the site and to about 10 m on the eastern side of the development (offset 6 m from the eastern boundary)

The upper overburden materials, down to the level of low strength sandstone, should be readily excavatable using conventional earth moving equipment (a hydraulic excavator) However large, high strength sandstone floaters may be present and their removal will require the use of hydraulic rock breaking equipment to break the boulders down to a manageable size for removal Medium and high strength sandstone bedrock is anticipated below depths of about 2 m to 3 m and will require the use of rock sawing, rotary milling head or hydraulic rock breaking equipment for bulk and detailed excavation

Confirmation of rock levels strength and the extent of material requiring the use of rockbreaking equipment will need to be undertaken as part of a detailed geotechnical investigation required to provide information necessary for the structural design of the project

Progressive inspection of the excavation within rock will be required to advise on the requirement for localised support measures Such measures are likely to comprise rock bolts, to support individual blocks of rock created by the intersection of unfavourable oriented jointing and face protection which may include the application shotcrete or sandstone facing for shaly and highly weathered seams

Across the lower, western section of the site groundwater was encountered at 1 7 m depth A proposed excavation depth of about 2 5 m (to RL 0 4) indicates that the basement will need to be constructed as a partially tanked structure Construction will necessitate the use of either a cut off wall to reduce groundwater inflow during construction or temporary dewatering Additional investigation, comprising test bores and groundwater monitoring will be required to confirm appropriate construction methods and design information particularly the range of groundwater levels during and following major rainfall events and the possible tidal affects on the water level

5 5 Excavation Vibration

All excavation methods give rise to some vibration From work carried out by Douglas Partners in the Sydney area, it has been found that residential buildings founded on bedrock are generally not adversely structurally affected by vibration levels below 10 mm/s (peak particle velocity) However, complaints from residents and building occupants have been recorded for values greater than 1 0 mm/s While vibration is only slightly perceptible to humans at levels of the order of 1 0 mm/s, it becomes strongly perceptible at levels above 3 0 mm/s and disturbing at levels above 5 0 mm/s

The damage threshold due to vibration is dependent on the foundation material and the quality of both a building's foundations and construction, as well as the peak particle velocity and the frequency of the vibration Lower frequencies are more likely to produce resonance in a building and to cause damage

It should be noted however that the governing factor for setting a vibration limit is likely to be the human comfort of occupants in the adjoining northern and eastern residence and it is suggested that, based on the Australian Standard AS2670 2 (1990) for daytime human comfort, a Provisional Allowed Vibration Limit of 8 0 mm/sec (PPVi) should be set

It may be prudent to undertake vibration monitoring during the site works to measure vibration levels and help determine appropriate excavation methods and equipment to avoid excessive vibration

Additional recommendations regarding the size of appropriate excavation equipment, suggested distance for use from adjoining structures, will be presented in the report of the detailed geotechnical investigation when subsurface conditions across the eastern part of the site have been confirmed

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5 6 Retaining Structures

Engineer designed retaining structures will be required both as part of the temporary excavation support and for the final structure

Retaining walls will be required specifically around the excavation boundary, down to at least the level of low to medium strength rock The nature of the support required will be determined by the depth to and the nature of the underlying bedrock across the sloping eastern portion of the site as well as the nature of the adjoining structures and their sensitivity to movement and possible settlement

Where the adjoining structures are sensitive to movement, possibly the adjoining southern sewer main, it may be necessary to provide positive lateral support designed for at rest (Ko) conditions

Excavation support along the eastern, upslope side could be provided by the use of bored soldier piles and infill shotcrete panels, probably with temporary anchors, whilst across the lower western portion it may be necessary to use contiguous piles for control of the shallow groundwater Depending upon the underlying soil profile and bedrock conditions and the structural design of the building it may be necessary to provide permanent anchor support for the eastern retaining wall and parts of the northern and southern walls, or alternatively the structure may brace/buttress the excavation

It is recommended that all proposed retaining walls be engineer designed in accordance with the parameters provided in Table 4

Table 4 – Summary of Retaining Wall Design Parameters

Material	Coefficient of Active Earth Pressure (Ka) *	"At rest" Coefficient of Earth Pressure (Ko) *	Unit Weight (kN/m^3)
Filling - uncompacted - compacted	0 4 0 3	0 6 0 45	20
Colluvium/sandy clay - uncompacted	0 35	0 5	20
Extremely to Highly Weathered Bedrock - very low strength	0 2	0 3	22

* Allowance will need to be incorporated to accommodate the slope of the site and any additional surcharge loads

All retaining structures will need to be designed taking into consideration additional loads due to any adjoining structures, the slope above the wall and any surcharges due to external loads They should be founded on in situ bedrock and should be designed to incorporate free draining backfill material behind the structure and appropriate subsoil drainage to discharge all seepage and groundwater collected within the backfill material and to prevent water pressure building up behind the wall Alternatively, the additional load due to full hydrostatic pressure should be allowed for in the design

5 7 Foundations

The inferred subsurface profiles across the proposed development area comprise either

- pavement and filling materials overlying sand to about 2 m depth (marine sediments associated with Snapperman Beach and Pittwater foreshore) underlain by medium to high strength bedrock , or
- 0 5 m to 2 m of sandy clay colluvium over low to high strength siltstone and sandstone bedrock

It is recommended that all foundations be taken down to the level of at least low strength bedrock It is considered that an allowable bearing pressure of 1000 kPa is appropriate for preliminary design purposes

However it is anticipated that following a detailed geotechnical investigation (including the drilling of cored test bores), a substantially greater value should be possible

Confirmation of allowable bearing pressure for pad or piered foundations will be required during construction

5 8 Drainage and Stormwater Control

It is recommended that the proposed development include stormwater and subsoil drainage control measures Such measures are very important to the maintenance and improvement of the stability of the site, particularly of the upper colluvium and overburden soils on the upslope side of the site

Appropriately sized grate-covered surface drainage should be installed along the boundary with the adjoining right-of-way with lined catch drains at the crest of slopes and batters and subsoil drains behind all new retaining walls All collected water should be directed by pipe-work into the Council stormwater system All piped drainage lines should include inspection ports to permit periodic maintenance by the owners

6 DESIGN LIFE

Douglas Partners Pty Ltd interprets the reference to design life requirements specified within the GRMP to refer to structural elements designed to retain the subject slope and maintain the risk of instability within acceptable limits

Specific structures that may affect the maintenance of site stability are considered to

- proposed retaining structures, and
- stormwater and subsoil drainage systems

These features should be designed and maintained for the design life of the proposed structures, which in our experience is normally taken to be in the order of 60 years In order to attain a life of 100 years, as required by the GRMP, it will be necessary for the structural engineer to incorporate appropriate design and structural inspection considerations and for the property owner to adopt and implement a maintenance and inspection program, details of which are included in Section 7 4

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7 CONSTRUCTION AND MAINTENANCE REQUIREMENTS

7 1 General

It is considered that, from a geotechnical perspective, the scope of site development depicted within the architectural plans is consistent with the comments and recommendations provided in this report

It is also considered that the site is suitable for the proposed development and that the development can be carried out within the "Acceptable Risk Management' criteria as defined by the GRMP, subject to the conditions detailed in the following sections

7 2 Construction Certificate Requirements

To provide suitable input to the structural design of the project, additional detailed geotechnical investigation (comprising four cored test bores – two on the upper half and two on the lower half of the site) will be required

There will be a requirement for Douglas Partners to examine all structural drawings prepared for the project (and any subsequent amendments) to verify that the recommendations given herein and as part of the detailed investigation, have been adopted or taken into account by the structural engineer to enable completion of a Pittwater Council GRMP Form 2 Part B for Construction Consent

All engineering support structures should have their design life nominated by the structural engineer together with any inspection/maintenance program required to attain the notional design life

7 3 Construction Inspection Requirements

Inspections by a geotechnical consultant will be required during construction to enable completion of a Pittwater Council GRMP Form 3 (Final Geotechnical Certificate – Post Construction Geotechnical Certificate)

Inspections and documentation of the following items/construction points will be required for

- all footing excavations for structural retaining walls (exceeding 1 m height)
- progressive inspection (each 2 m to 3 m lift) of the excavation in bedrock, to advise on the need and type of local support measures
- all footings for structural support of the structure
- any new subsurface drainage measures and drainage measures behind retaining walls (exceeding 1 m height)

7 4 Maintenance and On-going Inspection Requirements

To attain a life of 100 years it will be necessary to adopt and implement a detailed inspection regime as specified by the structural engineer and as recommended in Table 5 It will also be necessary to ensure that subsequent owners and occupants of the property are aware of the ongoing nature and frequency of the inspections

Table 5 – Recommended Maintenance and Inspection Program

Structure	Maintenance/Inspection Task	Frequency
Drainage lines	Inspect to ensure line is flowing and not blocked	Every 5 years or following each significant rainfall event
Drainage pits	Inspect to ensure that pits are free of debris and sediment buildup Clear surface grates of vegetation/litter build-up	During normal grounds maintenance and following each significant rainfall event
Retaining walls	Inspect walls for the presence of cracking or rotation from vertical or as-constructed condition	Every 5 years or following each significant rainfall event

If the maintenance inspections reveal noticeable changes, prompt reference should be made to an appropriate professional (eg structural or geotechnical engineer)

8 LIMITATIONS

Douglas Partners (DP) has prepared this report for this project at 1112 to 1118 Barrenjoey Road Palm Beach as requested and the work was carried out under DP Conditions of Engagement This report is provided for the exclusive use of the Lesiuk Architects Pty Ltd for

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the specific project and purpose as described in the report It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party

The results provided in the report are considered to be indicative of the sub-surface conditions on the site only to the depths investigated at the specific sampling and/or testing locations, and only at the time the work was carried out DP's advice may be based on observations measurements, tests or derived interpretations The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections DP cannot be held responsible for interpretations or conclusions from review by others of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report In preparing this report DP has necessarily relied upon information provided by the client and/or their agents

DOUGLAS PARTNERS PTY LTD

Reviewed by

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APPENDIX A
Notes Relating to this Report
Results of Field Work
Photographic Plates
Drawing

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and Classification Methods

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

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

Soil Classification	Particle Size
Clay	less than 0 002 mm
Silt	0 002 to 0 06 mm
Sand	0 06 to 2 00 mm
Gravel	2 00 to 60 00 mm

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

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

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

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

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

Sampling

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

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

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

Details of the type and method of sampling are given in the report

Drilling Methods

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

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

Large Diameter Auger (eg Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter The cuttings are returned to the surface at intervals (generally of not more than 0 5 m) and are disturbed but usually unchanged in moisture content Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling

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

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

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

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

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

The test results are reported in the following form.

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

  as 4, 6, 7
  N = 13

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

  as 15, 30/40 mm

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

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

Cone Penetrometer Testing and Interpretation

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

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises —

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

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

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

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

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

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

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

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

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

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Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration Normally, there is a depth limitation of 1 2 m but this may be extended in certain conditions by the use of extension rods

Two relatively similar tests are used

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289. Test 6 3 3) This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6 3 2) The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities

Laboratory Testing

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

Bore Logs

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

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

Ground Water

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

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

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

Engineering Reports

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

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

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

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

Site Inspection

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

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DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

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

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding The test procedure is described by Australian Standard 4133 4 1 1993

Term	Symbol	Field Guide	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0 03	< 0 6
Very low	VL	Material crumbles under firm blows with sharp end of pick can be peeled with a knife too hard to cut a triaxial sample by hand SPT will refuse Pieces up to 3 cm thick can be broken by finger pressure	0 03 0 1	0 6 2
Low	L	Easily scored with a knife indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point has dull sound under hammer A piece of core 150 mm long 40 mm diameter may be broken by hand Sharp edges of core may be friable and break during handling	0 1 0 3	2 6
Medium	M	Readily scored with a knife a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty	0 3 1 0	6 20
High	H	Can be slightly scratched with a knife A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow rock rings under hammer	1 3	20 60
Very high	VH	Cannot be scratched with a knife Hand specimen breaks with pick after more than one blow rock rings under hammer	3 10	60 200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material rock rings under hammer	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass which may be considerably weaker due to rock defects

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done

The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20 1 This ratio may vary widely

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STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0 2 m
Medium bedded	0 2 m to 0 6 m
Thickly bedded	0 6 m to 2 m
Very thickly bedded	>2 m

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous These include bedding plane partings joints and other rock defects but exclude known artificial fractures such as drilling breaks The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis Note that where possible recordings of the actual defect spacing or range of spacings is preferred to the general terms given below

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm
Highly Fractured	Core lengths are generally less than 20 mm 40 mm with occasional fragments
Fractured	Core lengths are mainly 40 mm 200 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths are generally 200 mm 1000 mm with occasional shorter and longer sections
Unbroken	The core does not contain any fracture

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i e low strength or better) core in lengths of greater than 100 mm to the total length of the core expressed in percent If the core is broken by handling or by the drilling process (i e the fracture surfaces are fresh irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece

SEDIMENTARY ROCK TYPES

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

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel sized (greater than 2 mm) fragments
Sandstone	More than 50% of the rock consists of sand sized (0 06 to 2 mm) grains
Siltstone	More than 50% of the rock consists of silt sized (less than 0 06 mm) granular particles and the rock is not laminated
Claystone	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated
Shale	More than 50% of the rock consists of silt or clay sized particles and the rock is laminated

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

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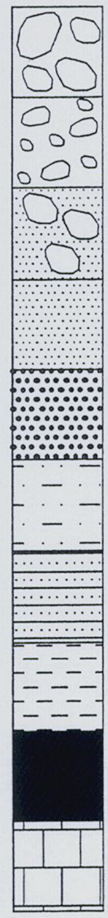
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL

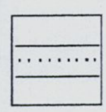
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SILT
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
COBBLES/BOULDER
TALUS

SEDIMENTARY ROCK

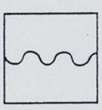


BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

SEAMS

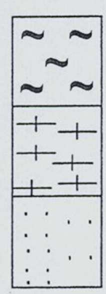


SEAM >10mm



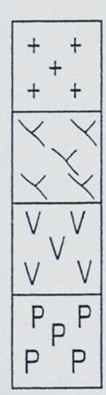
SEAM <10mm

METAMORPHIC ROCK



SLATE, PHYLLITE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

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TEST BORE REPORT

CLIENT: LESUK ARCHITECTS
PROJECT: PROPOSED DEVELOPMENT
LOCATION: 1112-1118 BARRENJOEY RD, PALM BEACH

PROJECT No: 36684
SURFACE LEVEL: 2.3 AHD
DIP OF HOLE: 90°

BORE No: 1
DATE: 16 Dec 03
SHEET 1 OF 1
AZIMUTH: --

Depth (m)	Description of Strata	Sampling & In Situ Testing: Type	Depth (m)	Test Results & Comments	Core Rec. %
0.02	GRAVEL				
0.07	ASPHALTIC CONCRETE				
	AGGREGATE/ROADBASE				
0.15	FILLING - brown dark grey silty sandy clay filling				
0.25	SILTY SAND - loose to medium dense, dark brown to black fine grained silty sand	A	0.4		
0.6	SAND - loose to medium dense, brown fine to medium grained sand (damp)	A	1.0		
	- moist from 1.5m, with rounded pebbles to 30mm	A	1.5		
	- wet from 1.7m, unable to recover cuttings				
1.8	TEST BORE DISCONTINUED AT 1.8m				

RIG: HAND TOOLS DRILLER: LLOYD LOGGED: LLOYD CASING: UNCASED
TYPE OF BORING: 100mm DIAMETER HAND AUGER
WATER OBSERVATIONS: FREE GROUNDWATER OBSERVED AT 1.7m
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: 22/12
Date: RMC

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RESULTS OF DYNAMIC PENETROMETER TESTS

CLIENT: LESIUK ARCHITECTS
DATE: 16/12/2003

PROJECT: PROPOSED DEVELOPMENT
PROJECT NO: 36684

LOCATION: 1112 – 1118 BARRENJOEY ROAD, PALM BEACH
PAGE NO: 1 of 1

TEST LOCATIONS	1									
RL of Test	2.3									
DEPTH m	PENETRATION RESISTANCE BLOWS/150mm									
0.00 - 0.15	-									
0.15 - 0.30	-									
0.30 - 0.45	-									
0.45 - 0.60	6									
0.60 - 0.75	6									
0.75 - 0.90	4									
0.90 - 1.05	4									
1.05 - 1.20	4									
1.20 - 1.35	3									
1.35 - 1.50	3									
1.50 - 1.65	4									
1.65 - 1.80	3									
1.80 - 1.95	8									
1.95 - 2.10	13									
2.10 - 2.25	15/20									
2.25 - 2.40	B									
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										

TEST METHOD AS 1289.6.3.2, CONE PENETROMETER ✓ TESTED: RKL

AS 1289.6.3.3, FLAT END PENETROMETER ☐ CHECKED: RKL

Note : 15/20 = 15 blows for 20 mm penetration : B = bouncing ; - = predrilled

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Photo 1 The site viewed towards the east

Photo 2 The site viewed towards the north from the adjoining southern property Grassed benching located on adjoining property Upslope residence visible in upper right hand section of photo.

PROPOSED DEVELOPMENT 1112 - 1118 BARRENJOEY ROAD PALM BEACH	Project 36684	December 2003	Plate 1

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Photo 3 Large sandstone floaters in slope behind residences approximately 150m to the south of the site

Photo 4 Shaly sandstone /sandy siltstone outcrop and overlying sandstone with minor shaly beds opposite Iluka Road shops

PROPOSED DEVELOPMENT 1112 - 1118 BARRENJOEY ROAD PALM BEACH	Project 36684	December 2003	Plate 2

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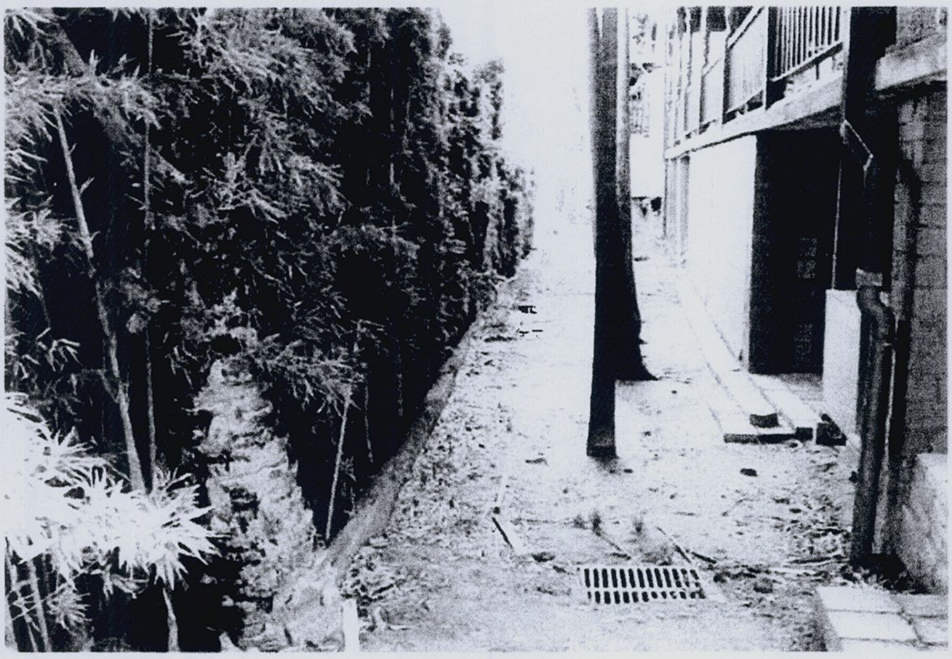

Photo 5. Upslope property above right-of-way Note grated drain, discharge location alignment not evident.

Photo 6. Colluvium overlying highly weathered outcrop of low strength shaly sandstone on southern side of timber residence.

Proposed Development 1112 - 1118 Barrenjoey Road Palm Beach	Project 36684	December 2003	Plate 3

Photo 7. Outcrop behind 1094 Barrenjoey Road showing sandstone above shaly sandstone/sandy siltstone.

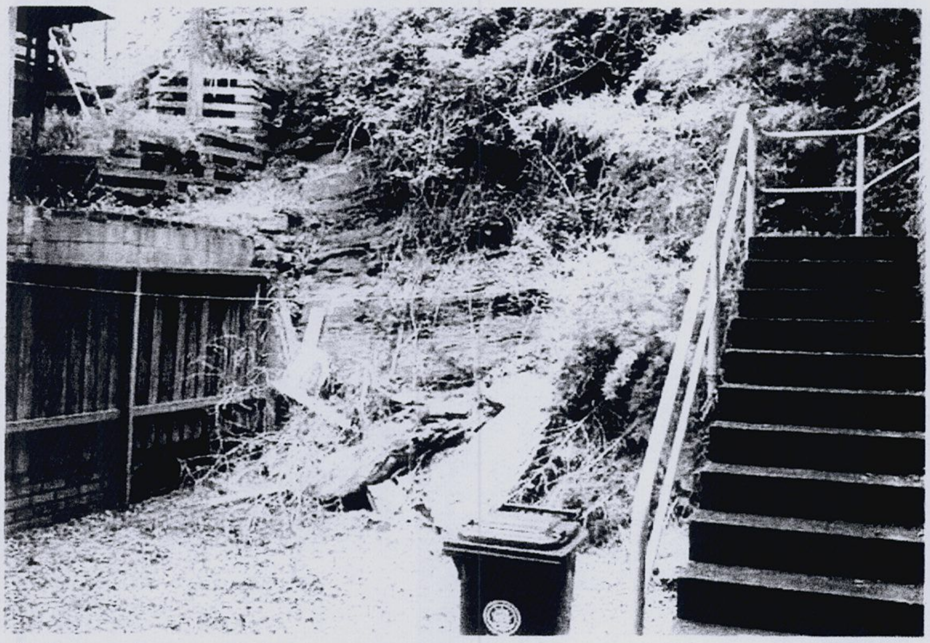

Photo 8 Shaly sandstone /sandy siltstone outcrop between 1096 and 1098 Barrenjoey Road.

Proposed Development 1112 - 1118 Barrenjoey Road Palm Beach	Project 36684	December 2003	Plate 4

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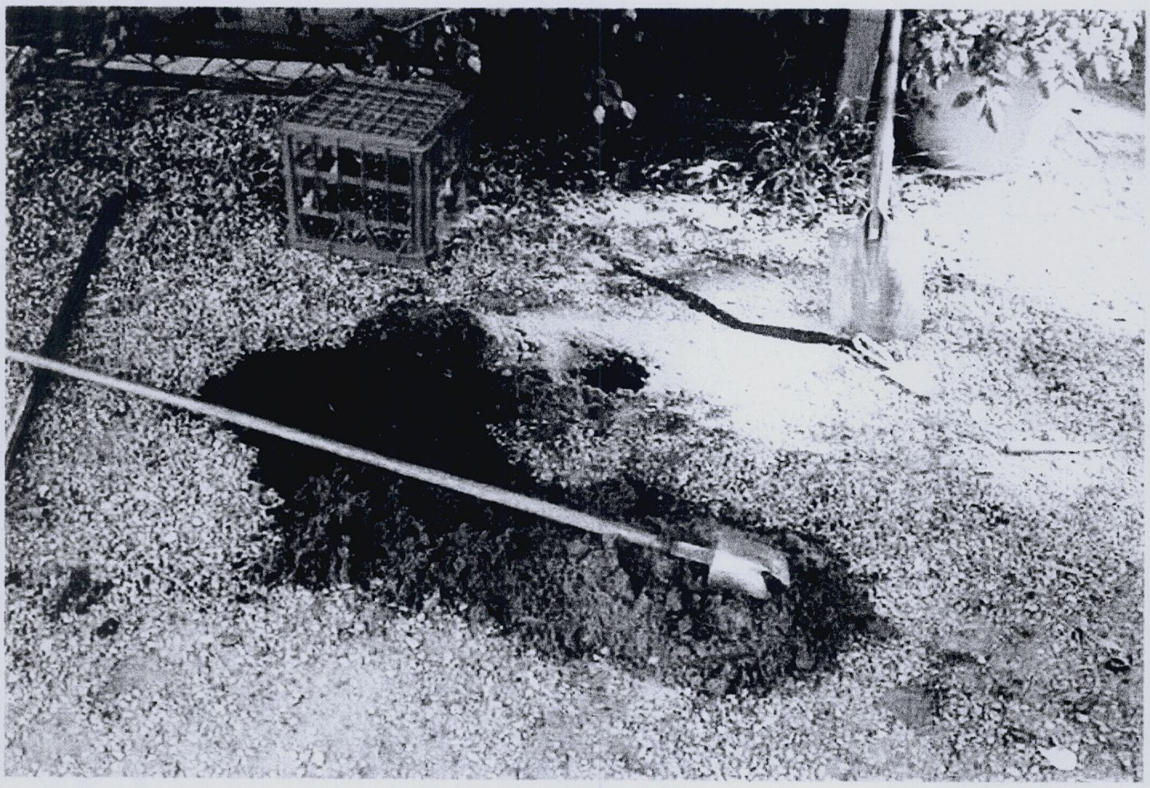

Photo 9. Material encountered in Bore 1.

Proposed Development 1112 - 1118 Barrenjoey Road Palm Beach	Project 36684	December 2003	Plate 5

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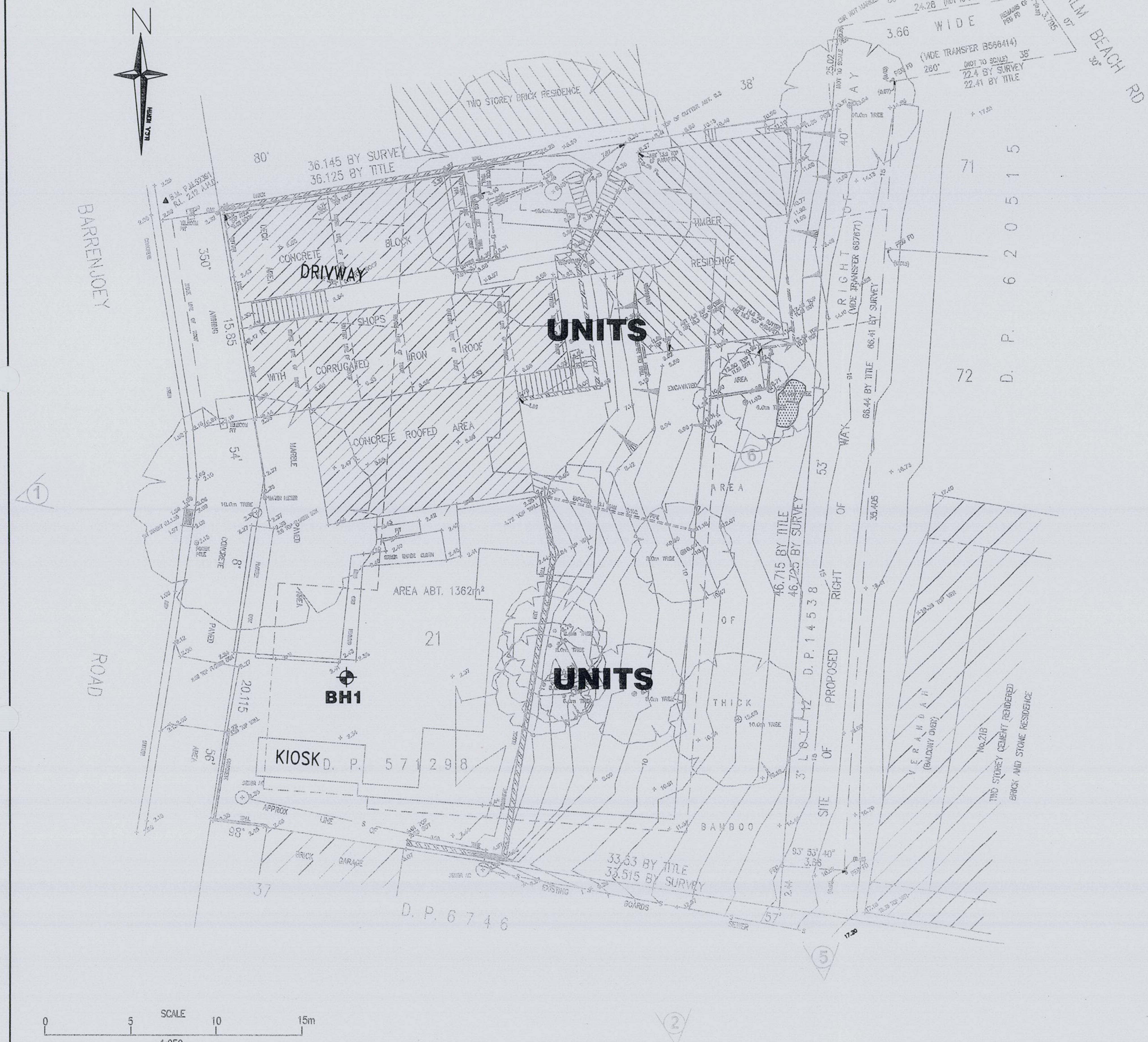

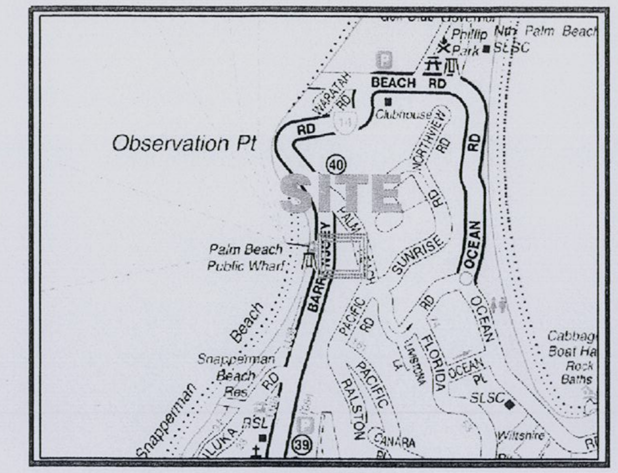

LOCALITY PLAN

LEGEND

- TEST BORE LOCATION AND DYNAMIC CONE PENETROMETER TEST LOCATION
- PHOTO NUMBER & DIRECTION OF VIEW
- SANDSTONE OUTCROP
- BASEMENT FOOTPRINT
- GROUND FLOOR BUILDING FOOTPRINT

Douglas Partners *Geotechnics · Environment · Groundwater*	CLIENT: Lesiuk Architects		TITLE: **Site Features & Location of Tests**	PROJECT No: 36684.02
	DRAWN BY: PSCH	SCALE: As shown	OFFICE: Sydney	**Proposed Development**	DRAWING No: 1
	APPROVED BY:		DATE: 29.10.2009	**1112to 1118 Barrenjoey Road, PALM BEACH**	REVISION: A

PITTWATER COUNCIL

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for ______________________________
Name of Applicant

Address of site 1112 – 1118 ~~Pittwa~~ Barrenjoey Road Palm Beach.

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Report on Geotechnical Investigation Proposed Development 1112–1118 Barrenjoey Road, Palm Beach
Report Date: 29 October 2009
Author: Project 36684.02 Richard Lloyd
Author's Company/Organisation: (Douglas Partners).

Please mark appropriate box

- [x] Comprehensive site mapping conducted 16/12/2003 & 19/8/2009.
(date)
- [x] Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- [x] Subsurface investigation required
  - [ ] No Justification ....
  - [x] Yes Date conducted 16/12/2003 & Additional recommended for CC.
- [x] Geotechnical model developed and reported as an inferred subsurface type-section
- [x] Geotechnical hazards identified
  - [x] Above the site
  - [x] On the site
  - [ ] Below the site
  - [x] Beside the site
- [x] Geotechnical hazards described and reported
- [x] Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - [x] Consequence analysis
  - [x] Frequency analysis
- [x] Risk calculation
- [x] Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- [x] Design Life Adopted:
  - [x] 100 years
  - [ ] Other ....
specify
- [x] Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- [x] Additional action to remove risk where reasonable and practical have been identified and included in the report.
- [ ] ~~Risk assessment within Bushfire Asset Protection Zone.~~

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature .... [signature]
Name .... GR WILSON
Chartered Professional Status .... RPGEO
Membership No. 10007
Company .... Douglas Partners P/L.

PITTWATER COUNCIL

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for ______________________________

Name of Applicant

Address of site 1112 – 1118 Barrenjoey Road Palm Beach

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, ______________________ *on behalf of* ______________________

(Insert Name) (Trading or Company Name)

on this the ______________________ certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million.

I have:

Please mark appropriate box

- ☐ Prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☑ I am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ Have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ☐ Have examined the site and the proposed development/alteration in detail and am of the opinion that the Development Application only involves Minor Development/Alterations that do not require a Detailed Geotechnical Risk Assessment and hence my report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements for Minor Development/Alterations.
- ☐ Provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title:	Report on Geotechnical Investigation Proposed Development 1112 – 1118 Barrenjoey Road, Palm Beach
Report Date:	Project 36684.02 29 October 2009.
Author:	
Author's Company/Organisation:	Richard Lloyd (Douglas Partners)

Documentation which relate to or are relied upon in report preparation:

Architectural Drawings – Lesiuk Architects Drawings DA00 to DA21.

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature .... [signature]

Name .... G R WILSON

Chartered Professional Status............ RPGEO

Membership No. 10007

Company............ DOUGLAS PARTNERS P/L