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PROJECT NO. 205-5614

MARCH 2007

KAVANAUGH CONSULTING ENGINEERS

PROPOSED NEW RESIDENCE

12 BROWNELL DRIVE
WATEGOS BEACH, BYRON BAY

Gold Coast Office
Job No: 205-5614
Ref: 2-5614BR
Author: Robert Maxwell

9th March, 2007

Sumi Davies
C/o- Kavanaugh Consulting Engineers
PO Box 10826
SOUTHPORT BC 4215

ATTENTION: MR PETER KAVANAUGH

Dear Sir,

**RE: GEOTECHNICAL INVESTIGATION - PROPOSED NEW RESIDENCE
12 BROWNELL DRIVE, WATEGOS BEACH, BYRON BAY**

Enclosed is one copy of our report for the above project dated March, 2007. Three copies of the report have been issued. A report addressing the temporary shoring design for the proposed new residence at the above site is included in Appendix E.

The report has been prepared in general accordance with our proposal 2-5614P2 dated 19th December, 2006. Authorisation to proceed with the investigation was received from Sumi Davies, dated 20th December, 2006.

Should you have any queries regarding this report, please do not hesitate to contact Robert Maxwell or Noel Perkins at our Gold Coast office.

Yours faithfully,

N. T. PERKINS (RPEQ 7527)
for and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation carried out by Soil Surveys Engineering Pty. Limited for the proposed new residence located at 12 Brownell Drive, Wategos Beach, Byron Bay. A report addressing the temporary shoring design for the proposed new residence at the above site is included in Appendix E.

Authorisation to proceed with the investigation was received from Sumi Davies, dated 20th December, 2006.

The objectives of this investigation were to assess subsurface conditions at the site in accordance with the Scope of Services detailed in Section 2.0.

2.0 SCOPE OF GEOTECHNICAL SERVICES

The scope of geotechnical services provided by Soil Surveys Engineering Pty. Limited was directed towards evaluating the following items as detailed in our proposal 205-5614P2 dated 19th December, 2006.

- The nature and type of subsurface material noting depth and condition of fill, natural soils and rock, if and where encountered.
- Earthwork recommendations
- Earthquake considerations.
- Foundation recommendations
- Retention recommendations and retaining wall design parameters
- Construction recommendations, where applicable.
- Site management recommendations.

3.0 PROPOSED DEVELOPMENT

It is understood that a multi level house is proposed at No. 12 Brownell Drive, Wategos Beach, Byron Bay. The proposed building is to be partially excavated into and stepping up the hillslope between Brownell Drive and Julian Place. Excavations up to 7.5m into the hillslope are envisaged.

Construction is envisaged to comprise rendered blockwork and weatherboard walls with a colourbond roof and suspended reinforced concrete upper floors. Loads are unknown at the time of the investigation.

4.0 GEOTECHNICAL INVESTIGATION

4.1 Field Investigation

To provide the required evaluations the following field investigations were carried out.

- Drilling and rock coring of one boreholes to a depth of 6.00m, using a truck mounted Gemco HP7 drill rig.

It should be noted that the borehole location was dictated by available access at the time of field investigation.

The soil/rock classification descriptions, field and laboratory testing were carried out in general accordance with the following Australian Standards.

- AS.1726 - 1993 Geotechnical Site Investigations
- AS.1289 Methods of Testing Soils for Engineering Purposes

A description of the investigation method, borehole records and a site plan showing the location of the boreholes are included in the Appendices.

Fieldwork was carried out on the 22nd January, 2007.

4.2 Site Description

The proposed new residence is located at 12 Brownell Drive, Wategos Beach, Byron Bay. The site is bounded to the northwest by Julian Place, the south by Brownell Drive and crown land to the east.

At the time of investigation, the site was occupied by an existing cottage. Mature trees were noted on the northwest and southeast boundaries.

The site generally slopes from Brownell Drive, down towards Julian Place. Ground levels vary from approximately RL23 to RL36. Drainage was assessed as fair to good.

5.0 GEOTECHNICAL MODEL

5.1 Subsurface Profile

The subsurface profile intersected during the drilling program comprised clayey sandy gravel overlying weathered rock.

Very dense clayey sandy gravel was encountered in the borehole to a depth of 1.0m overlying weathered rock. The weathered rock generally comprising weak extremely weathered metasandstone/siltstone, becoming distinctly to slightly weathered and strong at a depth of 3.0m and continued to borehole termination depth of 6.0m.

A summary of the subsurface profile is presented below with detailed borelogs included in Appendix B.

- *0.00 - 1.00*m Clayey Sandy GRAVEL (GC) - Very dense, fine to coarse size, grey/orange brown, fine to coarse grained sand, low plastic fines, moist.
- *1.00 - 3.00*m METASILTSTONE (XW) - Weak, orange brown mottled grey, moist.
- *3.00 - 6.00*m METASILTSTONE (DW-SW) - Strong, dark grey stained orange brown on defects.

5.2 Groundwater

Groundwater was not observed in the boreholes at the time of the investigation.

However, seepage may occur along the gravel/rock interface and through jointing and fissuring in the weathered rock profile following periods of rainfall.

6.0 ENGINEERING ASSESSMENT

6.1 Civil Works

6.1.1 Earthworks - General

It is understood that excavations into the hillslope, to create the lower ground level floor of the proposed residence. The proposed lower ground level will result in cuts of up to 7.5m.

6.1.2 Excavation Characteristics

It is anticipated that excavations will consist of the following:-

- Bulk Cuts - for excavation to create the stepped levels of the proposed residence.
- Trenching - for high level footings and underground services.
- Drilling - for bored pier foundations (if adopted).

Based on the subsurface conditions encountered in the borehole, it is considered that excavations can be carried out within the soil (gravel) and XW rock material using a medium size excavator with a toothed bucket. The use of a single tyne may be required towards the base of the XW rock.

Less weathered rock, weathering grades DW or SW will be more difficult to excavate and the use of a single tyne with a medium to large sized excavator and rock breaker work in the less weathered material and for detailed excavations will be required.

Bored Piers

It should be noted that the ability to drill piers in the weathered rock material is not only dependent on material characteristics but also the type (power and size) of the bored pier drilling rig, drilling teeth, size of pier, etc. It is recommended that the drilling contractors be consulted on this matter.

6.1.3 Batters

Given the proposed development, it is envisaged that no permanent batters are proposed on the site. The adoption of a 40° angle for all temporary batters is recommended for the upper level clayey sandy gravels and extremely weathered rock, and a minimum 55° batter angle for the distinctly weathered (or better) rock, **subject to inspection at the**

time of excavation by an experienced geotechnical engineer / engineering geologist. Note the above angles are for unsurcharged slopes. Where the slope above the proposed cut is to be surcharge by footings then these angles should be reviewed. Please refer to attached the report addressing the temporary shoring design for batters of 1V:4H in Appendix E.

6.2 Earthquake Considerations

The subsurface material encountered in the boreholes consisted of clayey sandy gravels and weathered rock. Liquefaction potential due to seismic activity is therefore unlikely as excavation levels will result in the building founding on gravel/rock.

In accordance with AS1170.4-1993, "Earthquake Loads", the site may be designated as earthquake design Category 'B' (Table 2.6) based on the following:-

- Acceleration Coefficient (a) 0.06 (Table 2.3)
- Site Factor (S) 0.67 (Table 2.4) (a)
- Structure Classification Type 2 (Appendix A)

6.3 Building Foundations

6.3.1 General

It is understood that construction of the proposed new residence is to comprise excavations of up to 7.5m into the hillslope. Excavations to these depths are envisaged to encounter weathered rock. On this basis the following foundation options may be considered for the proposed development.

- High level footings/Deepened footings - founding in consistent material
- Deep Foundations - extending to higher capacity rock or where footings are located above proposed retaining walls.

6.3.2 High Level Footings

High level footings founded into extremely weathered rock or better, would be generally suitable for the proposed development. Due to the slope across the site, the use of stepped base footings may be required.

It is recommended that foundations be deepened where necessary, to found in consistent weathered rock and designed with suitable articulation. This may require the use of mass concrete filled, backhoe excavated pedestals or alternatively, short bored piers may be used. In any case, the use of the parameters as outlined in this section are recommended.

Due to the variable final subsurface profile, it is recommended that footings be designed as an equivalent Class 'M' moderately reactive site, as per AS2870.

For design, the allowable base bearing capacities as outlined in Table 1 are recommended, subject to inspection at the time of excavation.

TABLE 1 ALLOWABLE BEARING CAPACITIES FOR HIGH LEVEL FOOTINGS

Material	Allowable Bearing Capacity (kPa)
Clayey Sandy Gravel - Very Dense	NR
Weathered Metasiltstone - XW - DW - SW	500 1500 3000
NR - Not recommended	

The above allowable bearing capacities are based on a Factor of Safety of 3.

Higher allowable bearing capacities may be available but this would depend on the condition of the founding strata encountered, e.g. occurrence of weak zones etc.

It is recommended that all allowable bearing capacities be confirmed by an experienced geotechnical engineer from Soil Surveys Engineering prior to the placement of the steel and concrete.

6.3.3 Construction Considerations

Inspections

Given the nature and strength of the subsurface material encountered, it is recommended that inspections be undertaken by an experienced and qualified geotechnical engineer/engineering geologist following footing excavations to confirm the adequacy of the founding material. Inspections should be carried out prior to placement of reinforcing steel and ordering of concrete.

Underground Services

Where footings are located adjacent to underground services or adjacent other excavations etc, the footings should extend to base a minimum of 300mm below the trench base level for a distance of 1.0m out from the trench. Beyond 1.0m the footings should be taken a minimum of 300mm below an imaginary line drawn up at 45^0 from the trench base level (Figure 1).

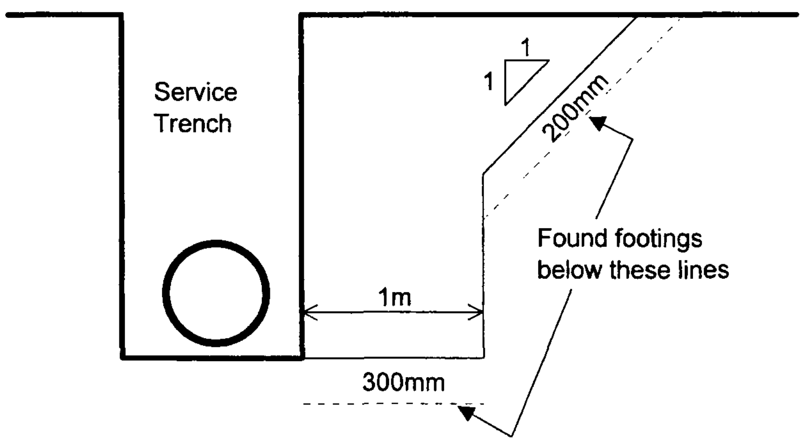

FIGURE 1

These requirements do not override minimum footing levels.

Bored Piers

Some difficulty with fall-in may occur with bored piers. It should be ensured that all loose material is removed from the base of piers prior to pouring of concrete. The use of a 'clean-out' bucket should be explicit in instructions to the drilling contractor. The practice of 'using water and spinning the augers' to remove loose material from the pier base is generally unacceptable.

6.4 Basement Slabs

Based on the subsurface profile encountered, it is expected the basement subgrade will comprised mostly of weathered rock.

The use of a sub-base "levelling layer" could be considered.

6.5 Excavation & Retention

6.5.1 General

As part of the construction of the proposed new residence, a maximum retained height of up to 7.5m is envisaged.

Given the geotechnical model based on the boreholes drilled over the site, it is expected that the retaining wall will retain up to 1.50m of natural clayey sandy gravels over weathered rock. The base of the excavation is expected to comprise DW-SW rock.

TABLE 2 GEOTECHNICAL MODEL PARAMETERS

Material	γ (kN/m^3)	c' (kPa)	ϕ' (°)
Clayey Sandy Gravel	18	0	32
Weathered Rock	25	5	35

The lateral earth pressure distribution that affects the retaining walls on the site will depend upon the following parameters:-

- Insitu material properties
- Design water regime at the rear of the wall
- Wall and cut geometry
- Boundary conditions ie location and type (ie. movement sensitivity) of nearby buildings, need for underpinning, surcharges, etc.
- Wall type
- The structural bracing/anchoring of the wall

Based on the expected cuts and subsurface profile encountered, two methods of retention of the proposed cuts are recommended:-

- Where space allows, cut the temporary batters to a safe batter angle. Refer to section 6.1.3 for recommended batter angles.

- Where sufficient space can not be found, the use of artificial support (passive dowels and shotcrete facing) could be adopted. Refer to attached design for a temporary soil nail/rock dowel support solution in Appendix E.

6.5.2 Pressure Distribution

The following situation should be considered :

- For structurally braced walls the wall design should be checked for both a trapezoidal (clay soils - short term conditions) and triangular distribution with Ko values (long term conditions).

The pressure distributions as referred to above are shown in Figures 2 and 3. The parameters selected for use in the figures are dependent on the preconstruction geometry of the face being retained.

The lateral pressure distributions shown in Figures 2 and 3 include hydrostatic pressure and show typical pressure distributions due to surcharge loadings. It is recommended that where the retaining walls are expected to be surcharged (eg. by footings, traffic loads, sloping ground surface, etc.) Soil Surveys Engineering should be contacted to provide a recommended lateral earth pressure distribution. Detailed assessment of earth pressures include assessment of anchor forces can be undertaken when the proposed retention systems are finalised.

As a general guide (using an elastic model) line loads located 2.5H away from the wall (where H is the wall height) result in lateral forces of less than 10% of the line load. Parameters for assessment of lateral earth pressures are outlined in Table 3.

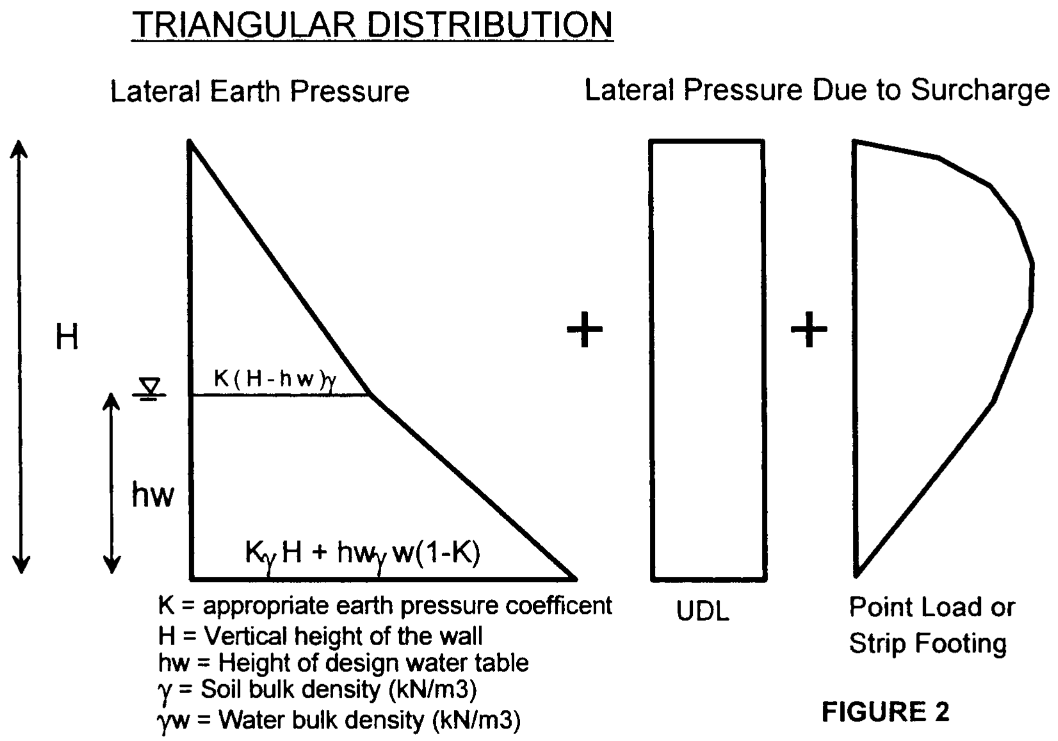

FIGURE 2

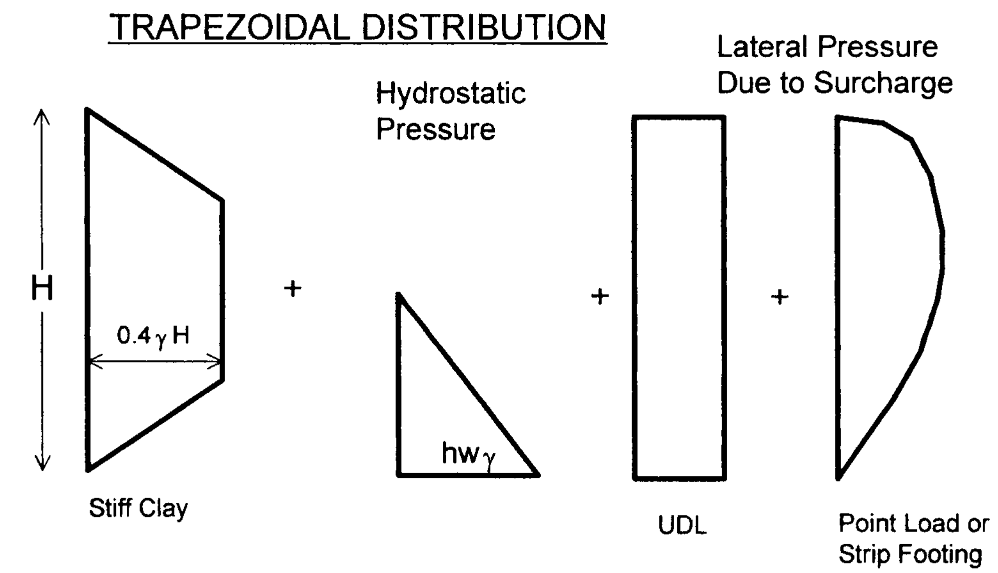

FIGURE 3

TABLE 3 RETAINING WALL DESIGN PARAMETERS (UNFACTORED)

Material	Density (kN/m^3)	Earth Pressure Coefficient		Long Term Drained Ø (degrees)
		Ka	Ko	
Clayey Sandy Gravel	19	0.32	0.50	30
Weathered Rock	25	0.27	0.43	35

Water Regime

Due to possible long term problems with blocking of gravel filters and drains and short term storm conditions that could flood the fill behind retaining walls, it is recommended that all retaining walls be designed for some water pressure distribution.

Design Criteria

It is recommended that any retaining structures be designed in accordance with AS4678-2002 Earthworks Retaining Structures.

7.0 CONSTRUCTION INSPECTION

It is recommended that regular inspections be carried out during the excavations by Soil Surveys Engineering Pty. Ltd.

As noted in the report the most critical section of the work is the drilling on the pier wall and excavation and stressing of anchors in the first row of panels, inspections of the pier hole cuttings will need to be undertaken with reassessment of the design if required. The need for inspections will reduce with depth.

Activities to be carried out will include:-

- Inspection and confirmation of footing system
- Assessment of pier hole cuttings
- Mapping of rock defects.
- Assessment of allowable bearing capacity of rock face for determination of stressing plates.
- Identification of zones requiring additional support eg. short rock bolt and shotcrete in weak or highly fractured zones.

- Re-analysis of restraint requirements (ie. anchors) depending on assessment of rock quality.

8.0 LIMITATIONS

We have prepared this report for the use of **SUMI DAVIES** and **KAVANAUGH CONSULTING ENGINEERS** for design purposes in accordance with generally accepted soils and foundation engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has not been prepared for use by parties other than **SUMI DAVIES** and **KAVANAUGH CONSULTING ENGINEERS** or their associated consultants. It may not contain sufficient information for purposes of other parties or for other uses.

Soil Surveys Engineering offer a documentation review service to verify that the intent of geotechnical recommendations is properly reflected in the design. It is recommended that the client avail themself of this service; our standard rates will apply.

N. T. PERKINS (RPEQ 7527)

for and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

APPENDICES

APPENDIX A

NOTES RELATING TO THIS REPORT

INTRODUCTION

These notes are provided by Soil Surveys Engineering Pty Limited (the Company) to complement the geotechnical report in regard to classification methods and field procedures. Not all notes are necessarily relevant to all reports.

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

DESCRIPTION AND CLASSIFICATION METHODS

Soils - The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726-1993 (Geotechnical Site Investigations), where appropriate. In general, descriptions cover the following properties - soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the dominant particle size and behaviour as set out in AS 1726-1993.

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, shear vane, laboratory testing or engineering examination. The strength terms are defined in AS1726-1993 Table A4.

Non-cohesive soils are classified on the basis of relative density usually based on insitu testing or engineering examination (see AS1726-1993 Table A5).

Rocks - Rock types are classified by their geological names (AS1726-1993 Table A6), together with

Table 1 Estimated strength descriptions given to rock based on engineering examination

Strength Term	Approximate Qu (MPa)
Extremely Weak	< 1.0
Very Weak	1.0 - 5.0
Weak	5.0 - 25
Medium Strong	25 - 50
Strong	50 - 100
Very Strong	100 - 250
Extremely Strong	> 250

Ref ISRM "Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses"

descriptive terms regarding weathering (AS1726-1993 Table A9), strength (refer Table 1 below), defects (AS1726-1993 Table A10), etc. Where strength testing (ie Point Loads) is carried out, AS1726-1993 Table A8 is used. Where relevant, further information regarding rock classification is attached.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon sample disturbance, (information on strength and structure).

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

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

INVESTIGATION METHODS

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

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

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

Continuous Spiral Flight Augers - The borehole is advanced using 75 to 300 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the augers. Information from the drilling (as distinct from specific sampling) is of relatively lower reliability due to remoulding, inclusion of cuttings from above or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table has a lower reliability than augering above the water table. Various drill bits are attached to the base of the augers during the drilling. The depth of refusal of the different bit types can provide information as to the strength of the material encountered. Generally two different bit types are used. The 'V' bit is a V shaped steel bit and the 'TC' bit is a tungsten carbide tipped screw type bit.

Wash Boring - The borehole is usually advanced by a rotary bit with water or fluid pumped down the hollow drill rods and returned up in the space between the rods and the soil or casing, 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. More accurate information on soil strata is gained by regular testing and sampling using the Standard Penetration Test (SPT) and undisturbed thin walled tube samples (U50).

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

Continuous Core Drilling - A continuous core sample is obtained using a diamond or tungsten carbide tipped core barrel. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable method of investigation. In rocks, NMLC coring (nominal 52 mm diameter) is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses is determined on site by the supervisor. If the location of the loss is uncertain, it is placed at the top end of the run, when the core is placed in a storage tray and recorded on the log.

Standard Penetration Tests - Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength. 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 50mm diameter split sample tube with a tapered shoe, 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, the upper 150 mm being neglected due to possible disturbance from the drilling method. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued at a reduced penetration.

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

4, 6, 7 N = 13

In a 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, the record shows:

15, 30/40mm

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

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

A modification to the SPT test is where the same driving system is used with a solid 60^0 tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid SPT are shown as "N_c" on the borehole logs, together with the number of blows per 150 mm penetration.

<u>**Cone Penetration Tests**</u> - Test Method - Cone Penetration Tests (CPT) are carried out in accordance with AS 1289 Test 6.5.1-1977, using an electrical friction-cone penetrometer.

The test essentially comprises the measurement of resistance to penetration of a cone of 35.7 mm diameter pushed into the soil at a rate of 10-20 mm per second by hydraulic force. The resistance to penetration is recorded in terms of pressure on the end area of the cone (cone resistance, q_c, in MPa) and friction on the side of the 135 mm long sleeve immediately above the top of the cone (friction resistance, f_s, in kPa). These forces are measured by electrical transducers (strain gauges) within the cone device. The ratio between friction resistance and cone resistance is also calculated as a percentage, ie.-

$$Friction\ Ratio\ (FR) = \frac{Friction\ Resistance, f_s\ (kPa) \times 100}{cone\ resistance,\ q_c\ (kPa)}$$

The friction ratio, FR, is generally low in sands (less than 1% or 2%) and generally higher in clays (say 3% or more). The interpretation of sandy clays, clayey sands and material with a high silt content is more difficult, but intermediate values (between 1% and 3%) would be expected. Highly organic clays and peats generally have a friction ratio in excess of 5%.

Static cone data is recorded in the field on disc for later presentation using computer aided drafting.

The equipment can be operated from any conventional drill rig. A total applied load in the range of 4 to 10 tonnes is required for practical purposes, although lighter loads may be used. The cone penetrometers are available with various capacities of cone resistance ranging up to 100 MPa for general purpose investigations, while a range of 0 to 10 MPa can be used where more sensitive investigations of soft clay are required.

The cone resistance value provides a continuous measure of soil strength or density, and together with the friction ratio, provide very useful indications of the presence of narrow bands of geotechnically significant layers such as thin, soft clay layers or lenses of sand which might otherwise be missed using conventional drilling methods.

The lithology of the encountered soils is interpreted from static cone data and is generally presented on the static cone log sheets.

It is important to note that the lithology is interpreted information and is based on research by Schmertmann (1970), Sanglerat (1972), Robinson and Campinalli (1986), modified to suit local conditions as indicated by borehole information and laboratory testing.

As soils generally change gradually it is sometimes difficult to accurately describe depths of strata changes, although greater accuracy is obtained with the static cone compared with conventional drilling. In addition, friction ratios decrease in accuracy with low cone resistance values, and in desiccated soils. As a result, some overlap and minor discrepancies may exist between static cone and nearby borehole information.

<u>**Portable Dynamic Cone Penetrometers**</u> - Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 100mm increments of penetration.

The DCP comprises a Cone of 20 mm diameter with 30 degree taper attached to steel rods of smaller section.

The cone end is driven with a 9 kg hammer falling 510 mm (AS. 1289 Test 6.3.2). The test was developed initially for pavement subgrade investigations, and empirical correlations of the test results with California Bearing Ratio have been published by various Road Authorities. The Company has developed their own correlations with Standard Penetration tests and Density Index tests in sands.

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms and the attached explanatory notes summarize important aspects of the Laboratory Test Procedures adopted.

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 the information and interpretation may not be relevant if the design proposal is changed. If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. Since the test sites in any exploration represent a very small proportion of the total site and since the exploration only identifies actual ground conditions at the test sites, even under the best circumstances actual conditions may vary from those inferred to exist. No responsibility is taken for:-

- Unexpected variations in ground and/or groundwater conditions.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of other persons.
- Any work where the company is not given the opportunity to supervise the construction using the Companies designs/recommendations.

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

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are 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.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer. We would be happy to assist in this regard as an extension of our investigation commission.

SITE INSPECTION

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

i) Site visits during construction to confirm reported ground conditions

ii) Site visits to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, the stability of a filled or excavated slope; or

iii) Full-time engineering presence on site.

In the vast majority of cases it is advantageous to the principal for the geotechnical engineer who wrote the investigation report to be involved in the construction stage of the project.

Soil Surveys Engineering Pty. Limited

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PO Box 2
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LEGEND SHEET

Information

DRILLING TYPES
V - open hole drilling using augers and a steel "V" bit
TC - open hole drilling using augers and a Tungsten Carbide bit
WB - wash boring using a drag or blade bit
RR - wash boring using a rock roller bit
NMLC - coring using a NMLC core barrel
Casing - steel casing in hole

DEPTH
Expressed in metres below the surface unless otherwise noted

ROCK DESCRIPTION
Rock name, grain size, colour, texture, fabric and any other relevent comments

WEATHERING GRADES
XW - Extremely Weathered
DW - Distinctly Weathered
SW - Slightly Weathered
FR - Fresh

STRENGTH
(estimate of UCS)
EW- extremely weak <1.0 MPa
VW - very weak 1.0 to 5.0 MPa
W - weak 5.0 to 25.0 MPa
MS - medium strong 25 to 50 MPa
S - strong 50 to 100 MPa
VS - very strong 100 to 250 MPa
ES - extremely strong >250 MPa

CORING DETAILS
Rec - core recovery in each run expressed as a %
RQD - Rock Quality Description expressed as a %

Defect Description

3.00m;J45;p,s,o,z

Depth in metres; Defect Type and angle with the core axis;planarity,roughness,aperture, infill

Defect Type	Planarity	Roughness	Aperture	Infill
J - Joint	p - Planar	s - Smooth	o - Open	z - Clean
F - Foliation	s - Sub-planar	r - Rough	c - Closed	c - Clay
B - Bedding	c - Curvi-linear	v - Very Rough		q - Quartz
V - Vein	u - Undulating	l - Slickensides		k - Calcite
S - Shear Zone				w - Weathered Rock
T - Fault				l - Limonite
C - Clay Seam				
Z - Contorted Zone				
RJ - Relict Joint				

Testing

SPT
10,15,15
SPT testing
N value
Blows/150mm or as noted

U50 (50)
PP = 500 kPa
U50 Tube Samples (% Recovery)
Pocket penetrometer reading

VS
1.30,5 : 25
Vane Shear Testing
Sealing, peak and residual : Corrected value

DIST
Disturbed Samples

Water Intersections

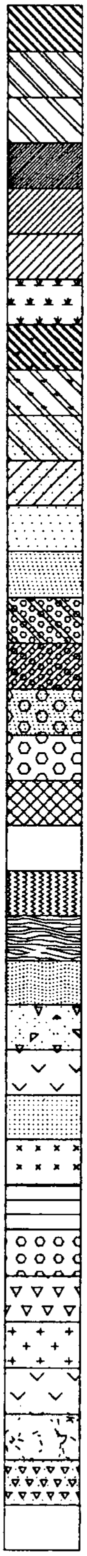
Water steady level

Water first noted

Testing Results

Is50=4 (A)
Corrected Point Load Test Result
A - Axial, D - Diametral

Graphic Legend



CH - CLAY of high plasticity
CI - CLAY of intermediate plasticity
CL - CLAY of low plasticity
MH - SILT of high plasticity
MI - SILT of intermediate plasticity
ML - SILT of low plasticity
Pt - PEAT
OH - Organic SILTS and CLAYS of high plasticity
OL - Organic SILTS and CLAYS of low plasticity
SC - Clayey SAND
SM - Silty SAND
SP - poorly graded SAND
SW - Well graded SAND
GC - Clayey GRAVEL
GM - Silty GRAVEL
GP - poorly graded GRAVEL
GW -well graded GRAVEL
Fill
Core loss
Phyllite
Meta-Siltstone
Meta-Sandstone
Greywacke
Tuff
Sandstone
Siltstone
Mudstone
Conglomerate
Breccia
Granite
Basalt
VOLCANIC ASH
VOLCANIC AGGOLMERATE
MARINE DEPOSITS

APPENDIX B

BOREHOLE RECORDS

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11 Production Ave
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Ph 61 7 5493 1733
Fax 61 7 5493 2837

BOREHOLE RECORD SHEET

Borehole Number : 1
Project Number : 205-5614
Project Name : Proposed Multi level Dwelling
Location : 12 Brownell Dr., Byron Bay
Client : Kavanaugh Consulting Engineers
Date : 22/01/2007
Page : 1

Easting : Northing : RL :
Logger : BM Driller : md Drilling Rig : Gemco HP7

Drilling Method	Depth	Description	Weathering	Rec (%)	RQD	Samples and Remarks
TC	0.15	Clayey Sandy GRAVEL (GC) Very dense, fine to coarse size, grey brown, fine to coarse grained sand, low plasticity fines, moist.				
TC	1.00	Clayey Sandy GRAVEL (GC) Very dense, fine to coarse size, orange brown, fine to coarse grained sand, low plasticity fines, moist.				
TC	1.20	SANDSTONE (XW) Weak, orange brown mottled light grey, fine to coarse grained sand.				
TC / WB	3.0	SILTSTONE (XW) Weak, orange brown mottled grey, moist.				SPT 23,30/110mm SPT 30/30mm
NMLC	3.00 – 6.00	METASILTSTONE (DW-SW) Strong, dark grey stained orange brown on defects, trace of fine sand, with very close to closely spaced 20-40deg. inclined rough weathered defects.	DW SW DW-SW SW DW SW	100 99	33 36	3.06m;J20;u,v,o,l 3.1m;J40;s,r,o,l 3.15m;J75;s,r,o,l 3.2m;J70;c,r,o,l 3.33m;J40;s,r,o,l 3.43m;J40;s,v,o,w 3.53m;J20;s,v,o,w 3.72m;J45;p,r,o,l 3.8m;J25;s,r,o,l 3.91m;J60;s,r,o,l 4.03m;J50;c,r,o,l 4.14m;J30;s,v,o,w 4.25m;J40;p,r,o,l 4.32m;J60;s,r,o,l 4.4m;J40;s,r,o,z 4.43m;J45;p,r,o,l 4.49m;J25;u,v,o,z 4.55m;J70;c,r,o,l 4.62m;J80;s,r,o,l 4.67m;J70;s,r,o,l 4.73m;J40;s,r,o,z 4.84m;J35;u,v,o,z 5.0m;RJ75;u,r,c,w 5.06m;J70;u,v,o,z 5.09m;J45;p,r,o,w 5.2m;J40;p,r,o,w 5.25m;J20;u,v,o,z 5.39m;J25;u,v,o,z 5.62m;J5;u,v,o,z 5.85m;J30;u,v,o,z 5.95m;J50;s,v,o,z 5.99m;J30;u,v,o,z
	6.00	Borehole Terminated 6.00m				

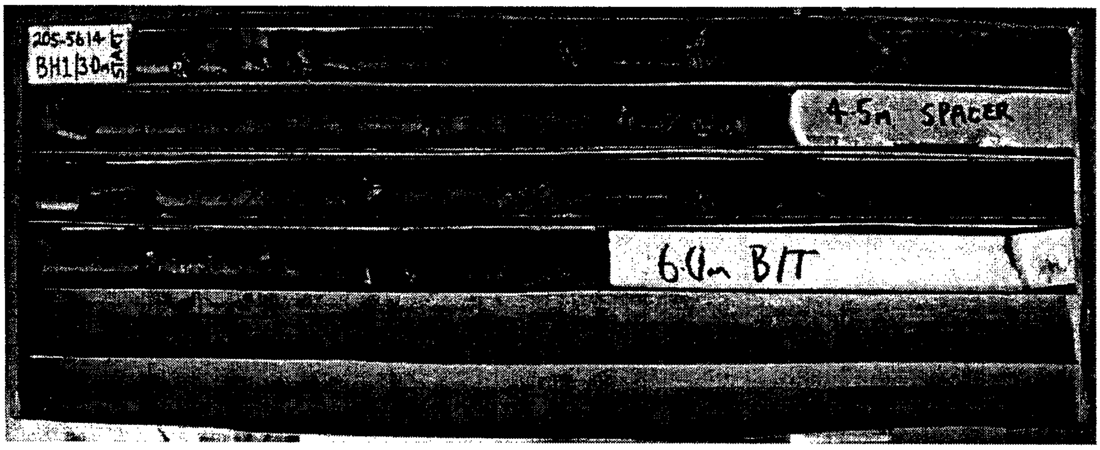
COMMENTS
1) Groundwater not observed.

Approved:
Date : 12/3/07

ISSUE No. 1.1 08/10/97 RS008A

APPENDIX C

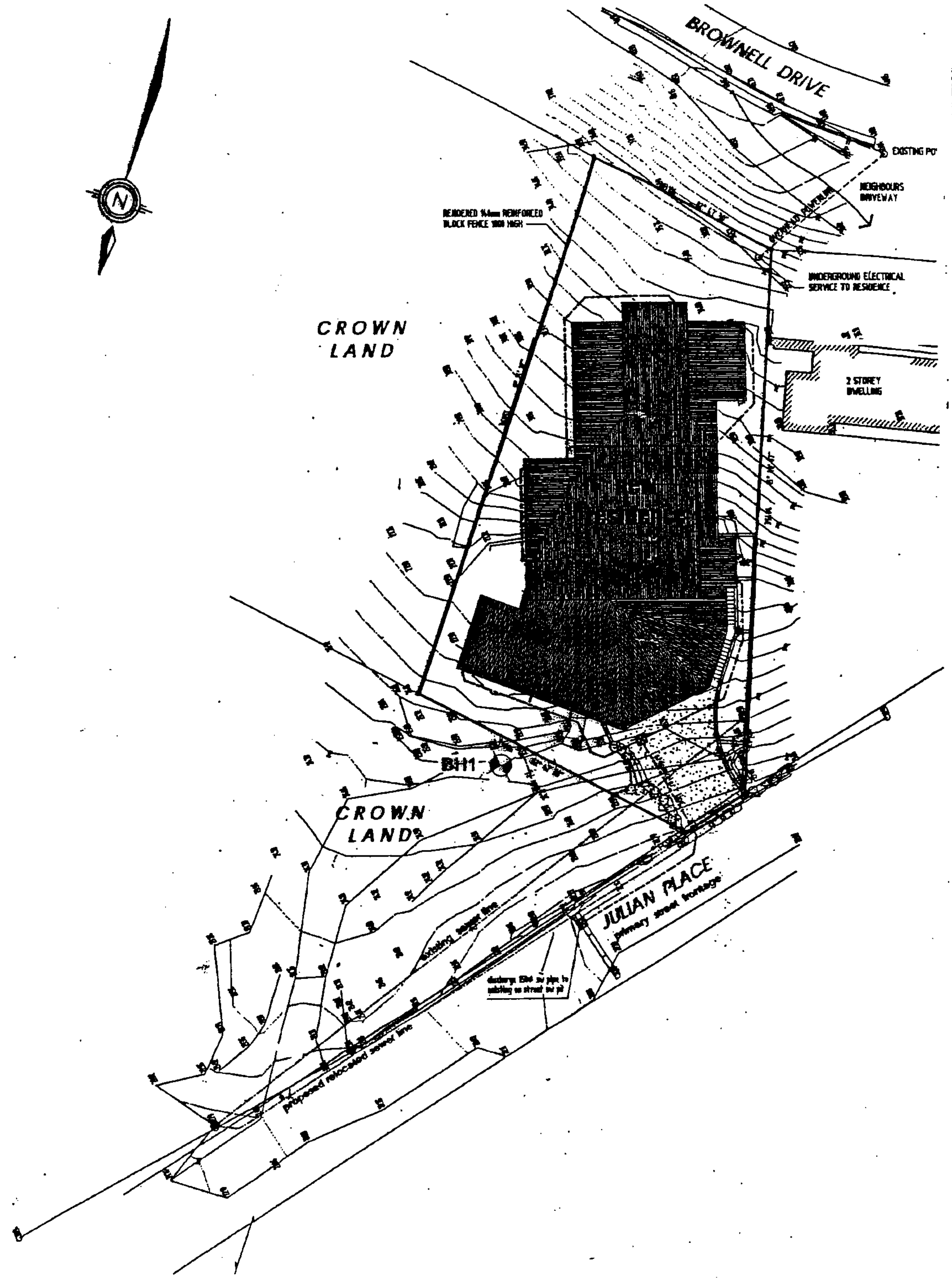
CORE PHOTOS



Borehole 1, 3.00m to 6.00m

APPENDIX D

SITE PLAN



SOIL SURVEYS
Soil Surveys Engineering Pty Limited
A.C.N. 054 043 631
Consulting Geotechnical Engineers

Drawn ELH	Project	Geotechnical Investigation	Drawing No.	
Date March 2007	Location	12 Brownell Dr, Byron Bay	205-5614-01	A4
Checked	Client	Kavanaugh Consulting Engineers		

APPENDIX E

TEMPORARY SHORING DESIGN

SOIL SURVEYS

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Email Transmission

To:	**Kavanaugh Consulting Engineers**	
Attention:	**Peter Kavanaugh**	
Address:	**peter@kaveng.com.au**	
From:	**Noel Perkins**	**Reviewed By:**
Date:	**27th February, 2007**	
Project No.	**205-5614**	
No. of pages including this page: 6		

Subject: ***No. 12 Brownell Drive, Wategos Beach, Byron Bay***

1.0 INTRODUCTION

As requested, a design has been undertaken to temporarily retain a proposed cut face at the above site. The face is to be excavated as part of the construction for a new residence.

2.0 SITE COMMENTS

Soil Surveys Engineering carried out a site investigation on the site in late January. As part of the investigation, one borehole was drilled at the base of the site where access permitted.

As part of the development, rock cuts of up to 7.5m have been proposed (assuming cut slopes of 4V:1H). Given the results of the site investigation, observation on site and on nearby sites, the use of a temporary support system would be required for the proposed batter to allow the safe construction of the structure.

3.0 SUPPORT SYSTEM

A passive dowel system for the site has been selected. The surface treatment adopted will be highly dependant on the properties of the excavated material. Where a high fractured face is found the use of mesh and shotcrete may be required. If the face is relatively slightly fractured a chain mesh type facing may be suitable. The selection of facing will need to be done during excavation and both options may need to be adopted.

4.0 SUPPORT DESIGN

4.1 General

The preferred method of support was the use of steel tendons (dowels) placed and grouted centrally in pre drilled holes across the face of the rock cut.

4.2 Parameters

Design parameters for the dowel wall are set out below :-

• Face Angle	60^0 above horizontal
• Top Angle	up to 15^0
• Rock Density	26kN/m^3
• Rock Plane Cohesion	10kPa (based on back analysis)
• Rock Plane Friction Angle	40^0 (based on back analysis)
• Maximum Height	7.5m
• Ultimate Grout/Rock Bond Strength	400kPa

The values given above are assumed values based on the site inspection, experience in similar types of material.

4.3 Dowel Design

A typical section for the wall is shown in Figure 1. The following general comments should be made with respect to the design.

Design Assumptions

- Structure Classification is type B as per Table 1.1 AS4678-2002 Earth-retaining structures
- Dowels are to be designed as temporary i.e. Anchor category 2 as per Table B1 Appendix B AS4678-2002 Earth-retaining structures.

Dowel Holes

- Dowels are centrally located in a pre drilled hole.
- Hole diameter 125mm (minimum)
- Drill hole at 15 degrees below horizontal
- Dowel holes located on a staggered pattern across face (i.e. holes displaced 1/2 hole horizontal spacing from row to row)
- Base row of dowels to be 1.0m above base of cut
- Vertical spacing is 2.5m (3 Rows H=7m, 2 Rows H=5m)
- Horizontal spacing is 2.0 m

Dowel Bars

- All dowels to be threaded at the ends to allow the dowels to be nominally stressed against the face.
- All dowels to have centralisers attached such that a minimum grout cover of 25mm is obtained. As a minimum centralisers should be located no further than 2m apart with the distal centraliser located no more than 0.3m from the end of the dowel.

- Plate size is 125mm (minimum) square.
- Minimum Dowel Length = 2.75m.
- Dowel size to be RB25.
- Base row of dowels nominally tensioned onto face.

Drain Holes and behind shotcrete drainage

- Drain hole on 4m spacing offset from dowels located 1m above base of cut.
- Drain hole 75 mm diameter (min)
- Slotted length of PVC placed in the drain hole
- Drain hole length = 3.0m
- Drain hole drilled at 15 degrees above horizontal (or as determined on site)
- Strip drain placed against exposed face prior to spraying shotcrete.

Face Protection - Shotcrete Option

- Face mesh SL72
- Spray concrete thickness = 100mm (min)
- Short lengths of dowel driven into top of bank on 1.5m spacing (Figure 1)

Face Protection Chain Mesh option

- 4mm Chain mesh fencing wire suitably lapped (or similar)
- Short lengths of dowel driven into top of bank on 1.5m spacing (Figure 1)

It should be noted that the controlling value for dowel length is the bond stress between the grout and soil. It is highly recommended that pull out tests be carried out on the site to confirm the design value. Some reduction of dowel length may be possible if greater bond values are identified.

4.4 Construction Procedure

The wall must be excavated in stages so as to maintain as safe a work place as possible.

i) Excavate to 0.7m below top row of dowels.

ii) Geotech to locate hole position (Hold Point).

iii) Drill holes for Row 1 across face.

iv) Centrally place dowels and grout.

v) Drive top pins.

vi) Place mesh (Hold Point to confirm reinforcement details).

vii) Shotcrete, leaving a minimum of 0.3m unshotcreted at the base, to lap to the next spray.

viii) Nominally stress dowels onto plate on face.

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ix) Excavate to 0.7m below next row of dowels.

x) Repeat ii) to x) to base.

4.5 Inspection & Certification of Anchors

During excavation and installation of the retention works, regular inspections must be undertaken to provide suitable certification of the support, refer to Section 4.4 for details of inspection points.

Hold points have been nominated above. Construction is to cease until inspected by Soil Surveys Engineering to confirm design assumptions and construction procedures are correct. Details of activities at each hold point are set out below.

Should you have any queries in relation to the above, please contact Noel Perkins at our Milton office.

Yours faithfully,

N. T. PERKINS (RPEQ 7527)

for and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

2-5614, 2007-02-08, Email

SOIL SURVEYS

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Rock Dowel Wall Detail

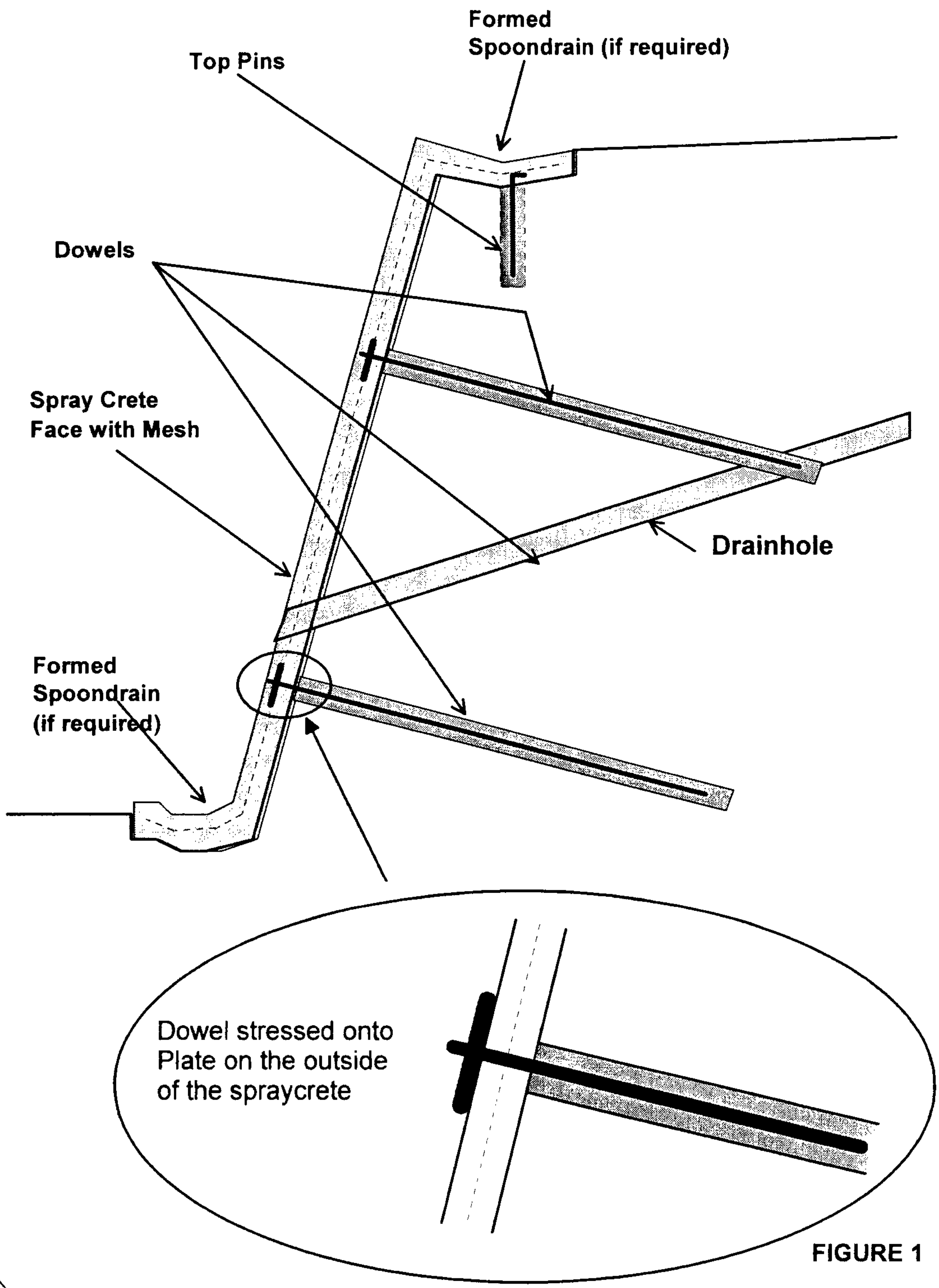
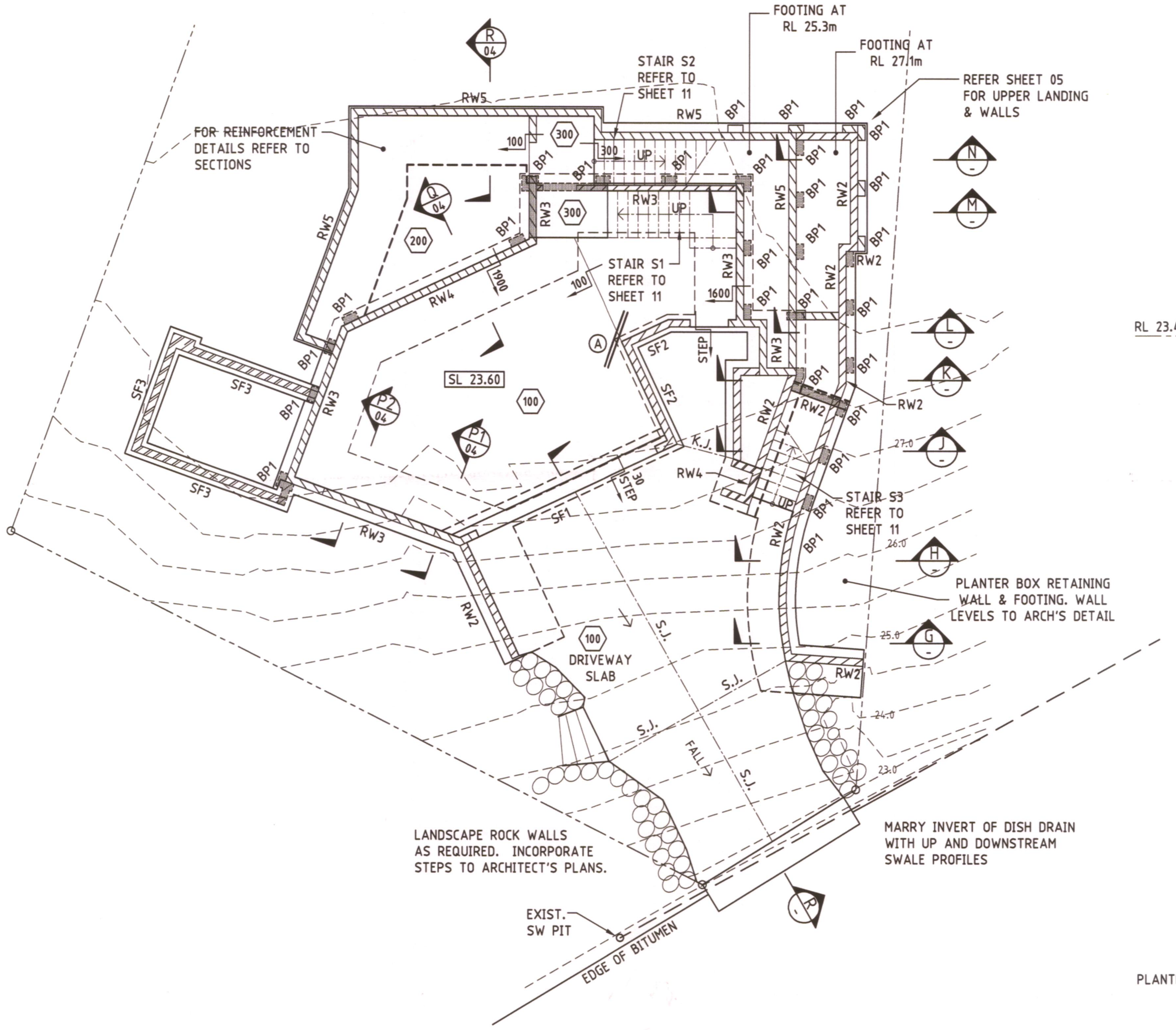
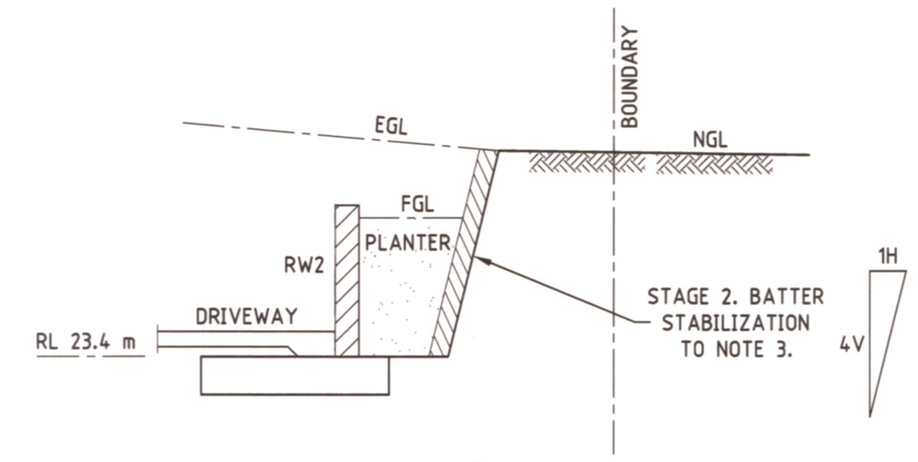


FIGURE 1

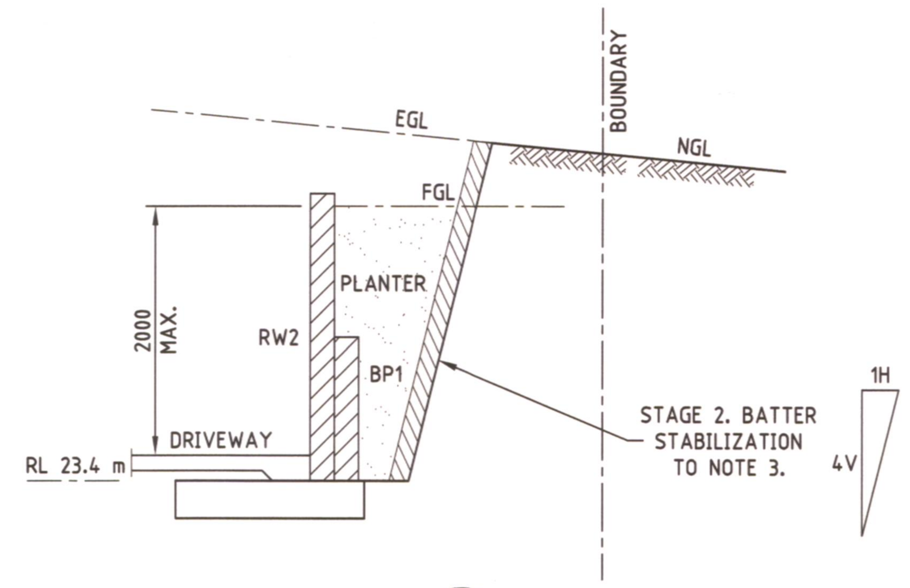
DO NOT ASSUME, IF IN DOUBT ASK.



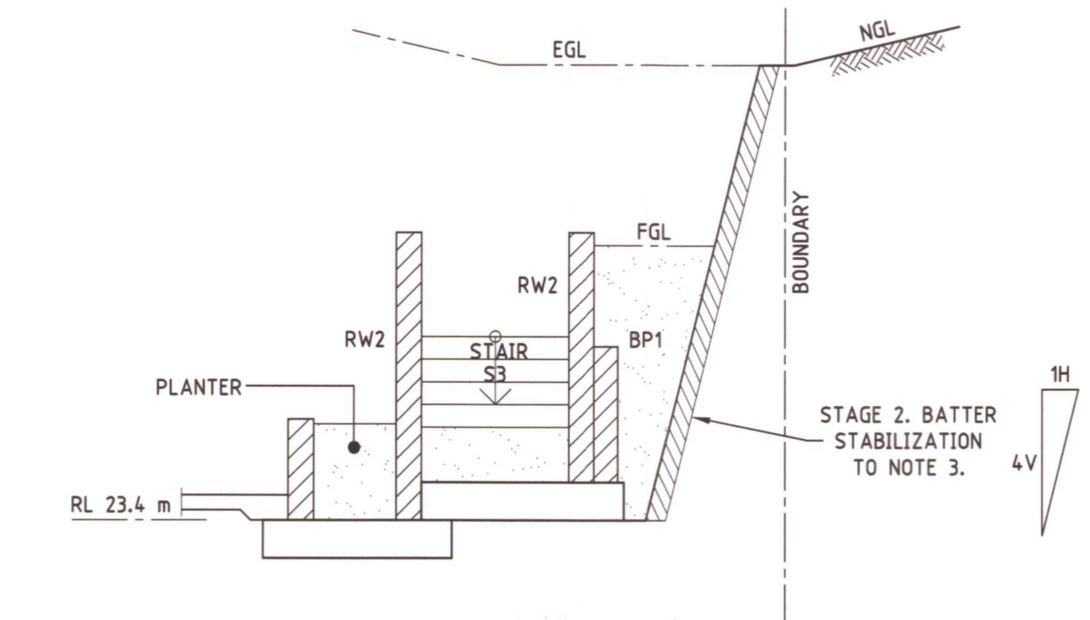
GARAGE FOOTINGS & SLAB ON GROUND LAYOUT PLAN 1:100



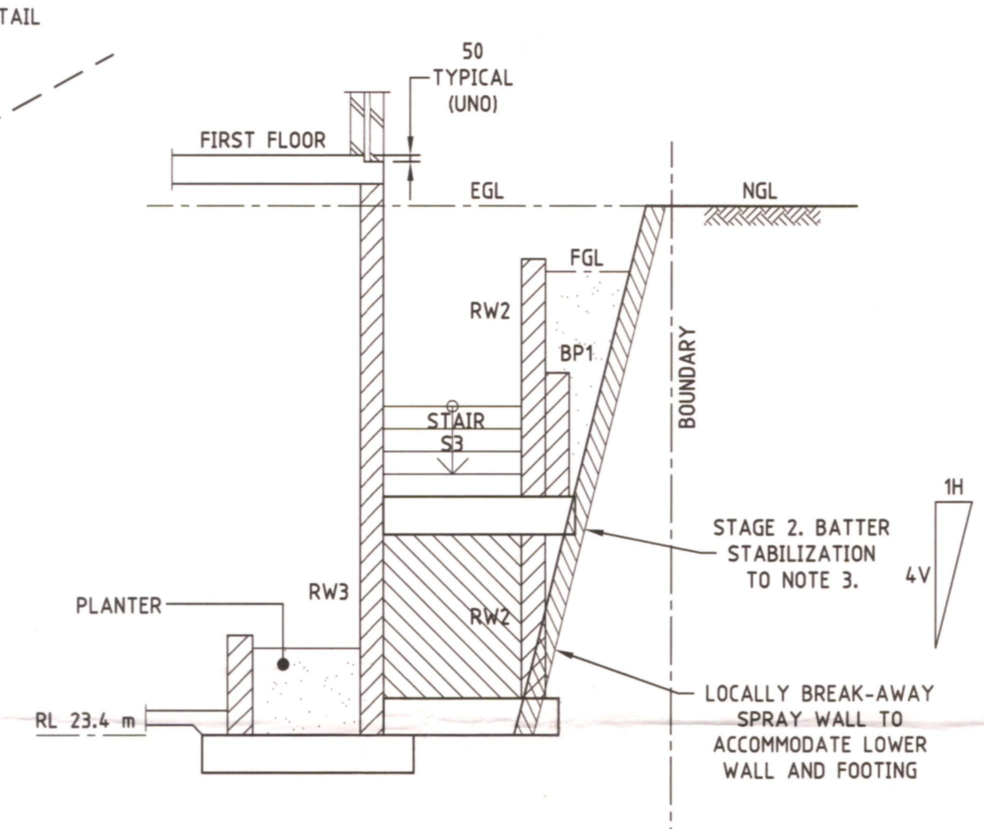
SECTION G 1:50



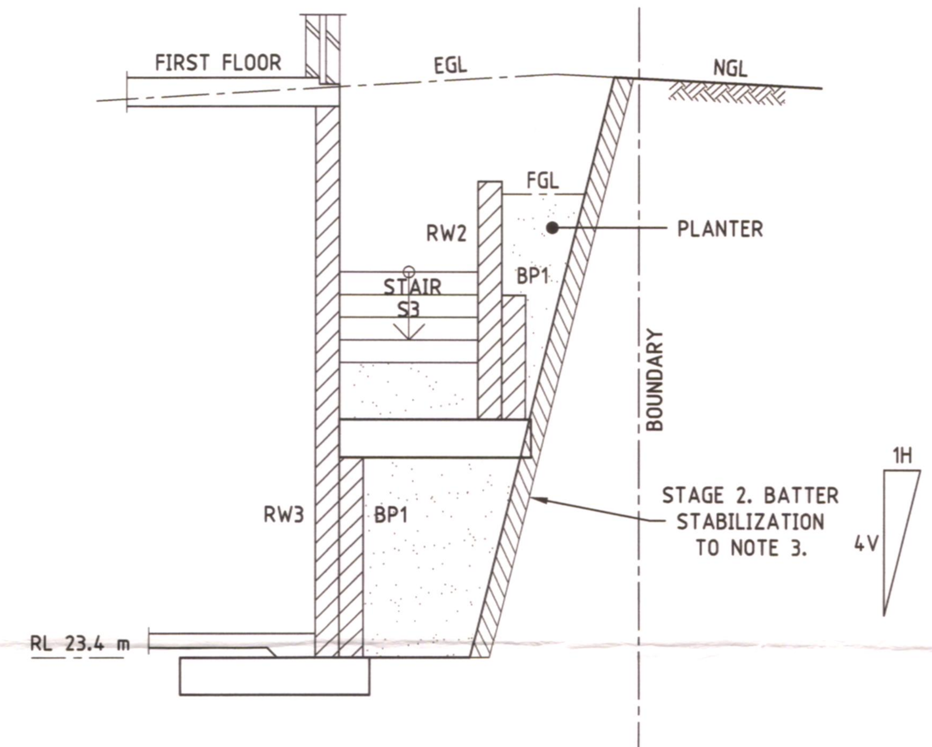
SECTION H 1:50



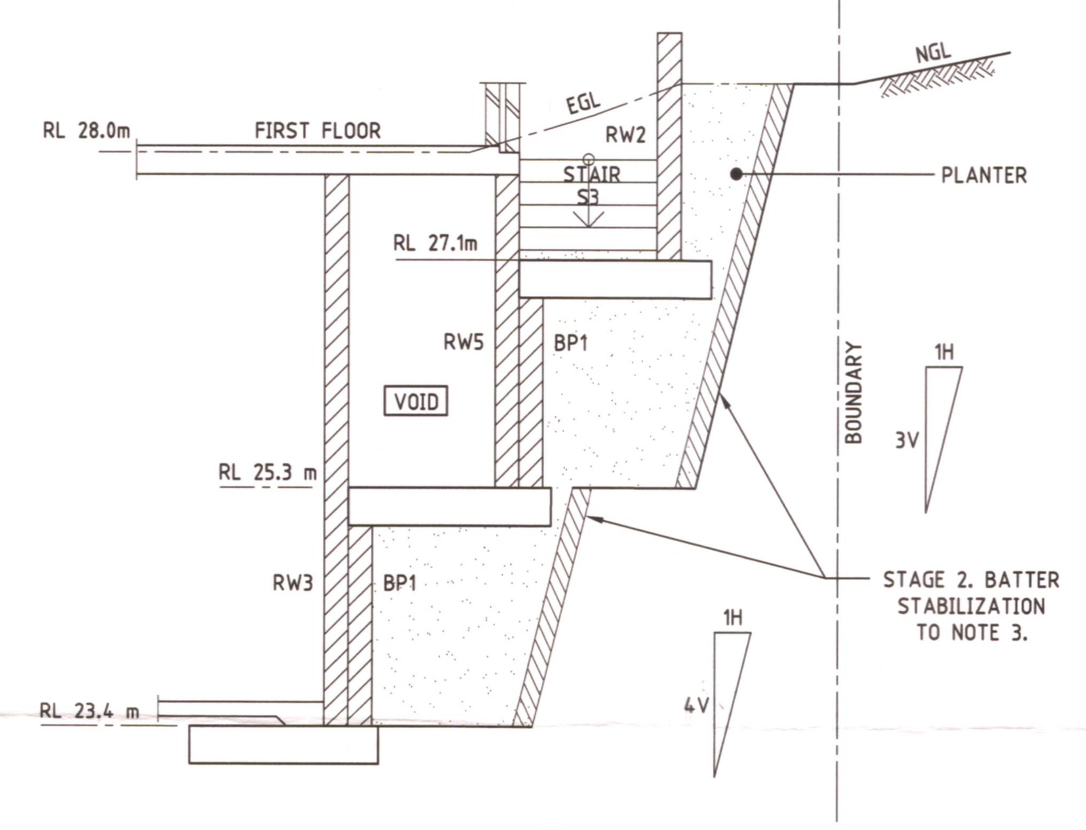
SECTION J 1:50



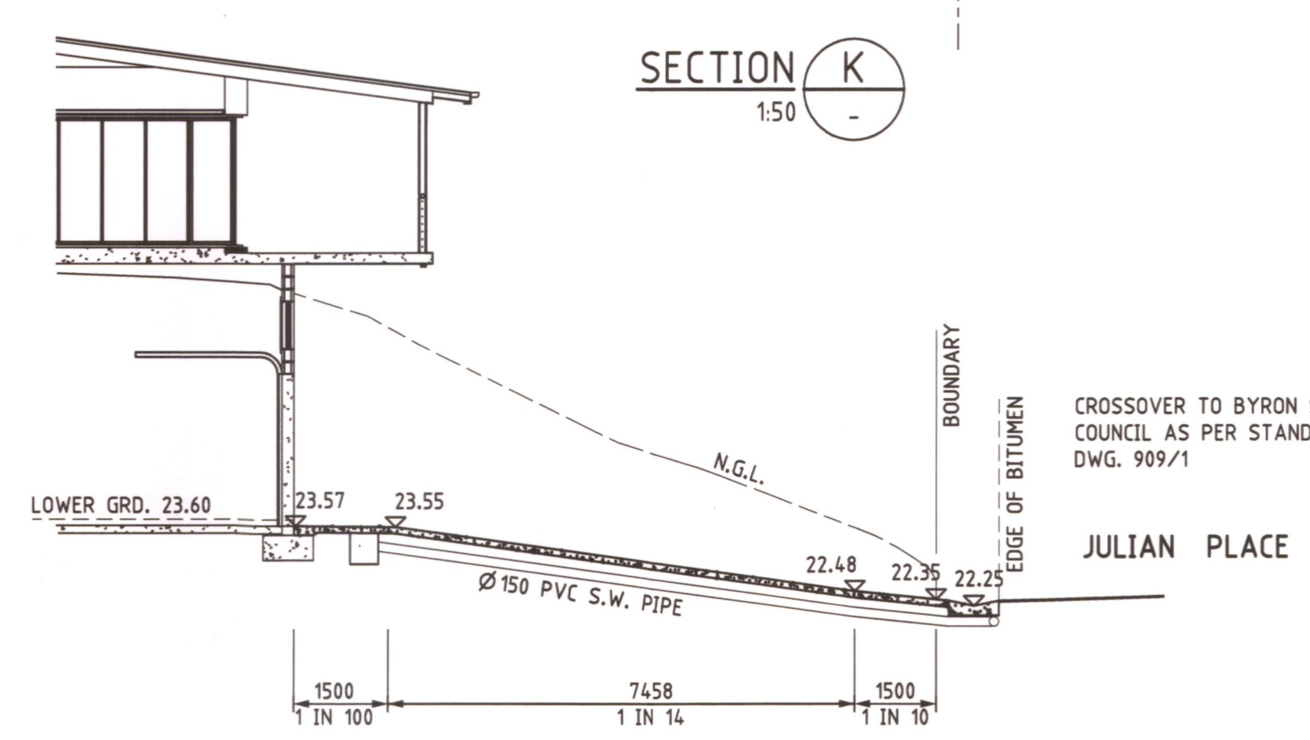
SECTION K 1:50



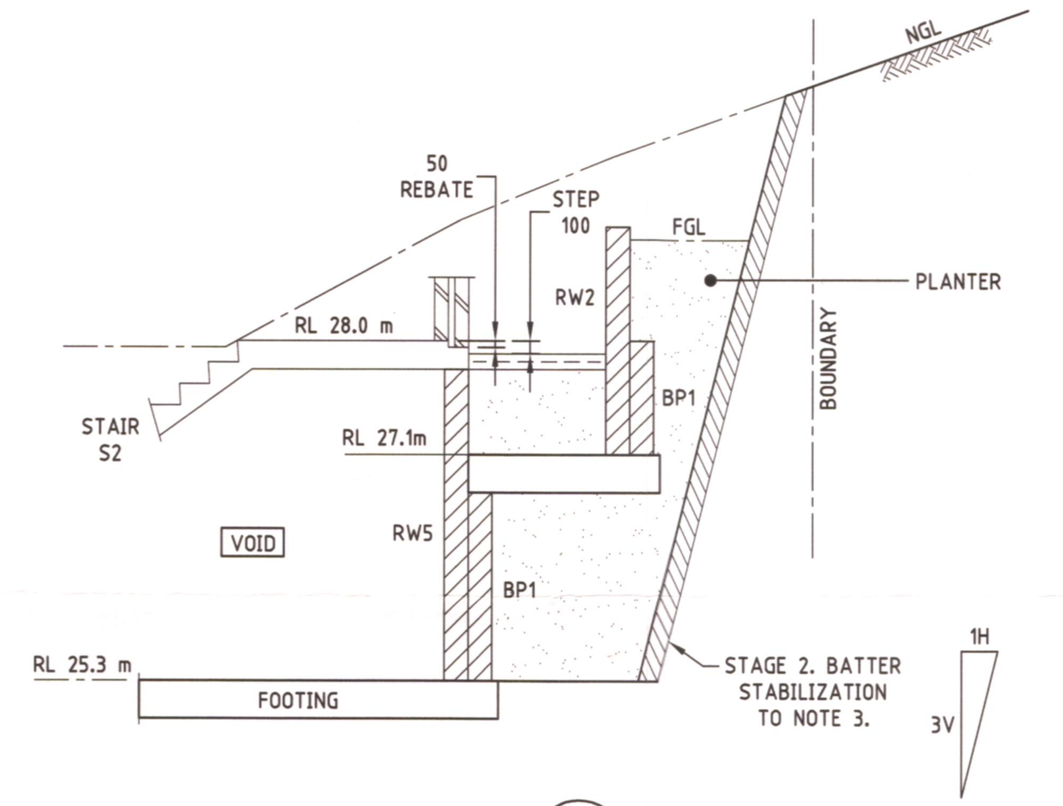
SECTION L 1:50



SECTION M 1:50



SECTION R 1:100



SECTION N 1:50

SLAB ON GROUND AND FOOTING NOTES

SF01. SITE SOIL CLASSIFICATION = 'CLASS P/M'
REFER TO SITE SOILS REPORT BY SOIL SURVEYS ENGINEERING JOB No. 205-5614

SF02. CONSTRUCT THE SLAB(s) ON GROUND ON A PREPARED BUILDING PAD WITH FINISHED LEVELS AS SHOWN ON PLAN AND/OR BY OTHERS. VERIFY ALL LEVELS ON SITE PRIOR TO ADOPTION. DIRECT ALL SITE DRAINAGE WELL AWAY FROM THE NEW BUILDING AND SURROUNDS, REFER STANDARD NOTES.

SF03. CONSTRUCT SLAB(s) ON GROUND WITH 120mm THICK N20 CONCRETE, REINFORCE WITH SL82 FABRIC AT 30 TOP COVER. TIE ALL TRIMMERS TO UNDERSIDE OF SLAB FABRIC.

SF04. POUR SLAB(s) ON GROUND ON AN APPROVED MOISTURE BARRIER, LAPPED 200mm, OVER 50mm COMPACTED SAND BEDDING LAYER. TAPE SEAL ALL LAPS AND PENETRATIONS. TERMITE CONTROL TO AS 3660.1 BY OTHERS.

SF05. REFER TO THIS SHEET AND SHEET 04. FOR CONSTRUCTION DETAILS.

SF06. BUILDER IS TO CHECK AND VERIFY THE EXISTENCE OF ANY UNDERGROUND SERVICES WITHIN 3.0m OF THE BUILDING FOOTINGS. IF EVIDENCE IS FOUND CONTACT THE ENGINEERS, AS ADDITIONAL FOUNDATION WORKS MAY BE REQUIRED.

SF07. CONSTRUCT SHOWER RECESSES AS INDICATED AND/OR AS OTHERWISE DIRECTED BY THE ARCHITECT. BATHROOMS, ENSUITES & WET AREAS TO HAVE 35mm RECESS WITH A FURTHER 40mm STEP DOWN AT THE SHOWERS OR AS OTHERWISE DIRECTED BY THE ARCHITECT.

LEGEND

(A) - 2-N12 TRIMMERS x 1500 LONG

BP1 - 190w x 390 long BLOCK PIER WITH 2-N16 VERTICAL BARS LAPPED 500 TO MATCHING STARTERS.

RW_ - RETAINING WALL TO DETAILS ON SHEET 04.

SF_ - STRIP FOOTING TO DETAILS ON SHEET 04.

K.J. - FULL DEPTH SLAB KEYED JOINT.

S.J. - SOFF-CUT SAWN JOINT WITHIN 24 HOURS OF CASTING SLAB. CUT ALTERNATE FABRIC WIRES PRIOR.

BUILDING SERVICES
P.O. BOX 1049
BALLINA NSW 2478
Ph: (02) 6681 5258
Fax: (02) 6681 5258

ISSUE	DATE (D.M.Y)	REMARKS	INIT.
C	07-06-07	AMEND. TO SUIT ARCHITECTS REQ'S.	RTK
B	02-05-07	MISCELLANEOUS AMENDMENTS	AS
A	13-04-07	APPROVAL & CONSTRUCTION ISSUE	TP

ISSUE	DATE	REMARKS	INIT.

SHEET TITLE
GARAGE FLOOR SLAB PLAN AND SECTIONS

KAVANAUGH CONSULTING ENGINEERS PTY LTD
STRUCTURAL AND CIVIL
ABN 86 170 891 669
SUITE 5, STRADBROKE PLAZA, 66 MARINE PDE, SOUTHPORT P.O. BOX 10826, SOUTHPORT BC, QUEENSLAND 4215
TELEPHONE: (07) 5532 4050 FACSIMILE: (07) 5531 1034 EMAIL: admin@kaveng.com.au

DRAWING SCALES USED 1:100, 1:50
DRAWN TP
DESIGNED BY T. D. PUGH NPER No – RPEQ No –
CHECKED BY P. J. KAVANAUGH NPER No 247492 RPEQ No 1118

ORIGINAL SHEET SIZE 841x597mm UNREDUCED A1
CAD FILE NAME K0016S1-01.dwg
APPROVAL DATE (D.M.Y) 07-06-07
APPROVED BY NPER No 247492 RPEQ No 1118

ARCHITECT PROJECT PLANNING GROUP (07) 5538 2883
PROJECT PROPOSED RESIDENCE LOT 15 BROWNELL DRIVE WATEGOS BEACH
FOR MR & MRS DAVIES

PROJECT No. K0016-S1
DRAWING No. 03 OF 11
ISSUE D

Filename: K0016S1-01.dwg
Location: I:\STR_DWG\K0001-K0099\K0016-s1 Lot 15 Brownell Drv Wategos Beach for Davies
Plot Date: 19/06/2007 8:08:18 AM