Douglas Partners
Geotechnics | Environment | Groundwater

Report on Geotechnical Assessment

Proposed Mixed Use Development
Lot 202 DP1054190 Meares Pl & Lot 1 DP743509
Collins Street Kiama

Prepared for
Kiama Shores Pty Ltd

Project 38145.04
August 2018

Integrated Practical Solutions

Douglas Partners
Geotechnics | Environment | Groundwater

Document History

Document details

Project No.	38145.04	Document No.	R.001.docx
Document title	Report on Geotechnical Assessment Proposed Mixed Use Development		
Site address	Lot 202 DP1054190 Meares Pl & Lot 1 DP743509 Collins Street Kiama		
Report prepared for	Kiama Shores Pty Ltd		
File name	38145.04.R.001		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	A Castrissios	G W McIntosh	13.08.2018

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Peter V'landys, Kiama Shores Pty Ltd
Revision 0	1	0	Vanessa Alves dos Santos, Kennedy Associates

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

	Signature	Date
Author	Arthur Castrissios	13.8.2018
Reviewer	G W McIntosh	13.8.2018

bsi. ISO 9001 Quality Management
FS 604853

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
1/1 Luso Drive
Unanderra NSW 2526
PO Box 486
Unanderra NSW 2526
Phone (02) 4271 1836
Fax (02) 4271 1897

Table of Contents

Page

Report on Geotechnical Assessment
Proposed Mixed Use Development
Lot 202 DP1054190 Meares Pl & Lot 1 DP743509 Collins Street Kiama

1. Introduction

Douglas Partners Pty Ltd (DP) has reviewed existing information and preliminary architectural plans to provide comment on the geotechnical aspects that will be required to be considered for a proposed mixed use development within Lot 202 DP1054190 Meares Place and Lot 1 DP743509 Collins Street at Kiama.

It is understood that the overall design of the development has been amended to incorporate the recommendations of the Joint Regional Planning Panel (JRPP) meeting held at Kiama Council on 15 November 2017 and that geotechnical comment is required to address the amended layout.

The assessment comprised a site visit by a Principal Geotechnical Engineer and a review of the preliminary architectural plans prepared by Kennedy Associates Architects.

2. Background

DP has previously undertaken a geotechnical investigation of the site, with the relevant report summarised as follows:

- DP Project 38145.01 dated 1 June 2016 *"Report on Geotechnical Investigation, Proposed Unit Development" Lot 202 DP1054190 and Lot 1 DP43509 Meares Place, Kiama.*

The work was undertaken to supplement the subsurface information from a DP February 2003 preliminary investigation. The 2016 investigation comprised the drilling of three boreholes (Bores 101 – 103) to depths of 2.2 – 14.1 m, standpipe piezometer installation and groundwater level monitoring, followed by engineering analysis and reporting.

Details of the work undertaken and the results obtained were given in the report together with geotechnical advice on excavations, site excavation, excavation stability, retaining structures and foundations.

3. Proposed Development

3.1 Previous Layout

The previous proposal for Lot 202 comprised a medium density development including three levels of seniors living units and two levels of basement car parking. A three storey building including residential units over ground floor commercial space with partial basement excavation for the car park access ramp was proposed on Lot 1. The proposed basement of Lot 202 was proposed to be within approximately 2 m, 3 m, 4 m and 15 m of the northern, eastern, southern and western boundaries, respectively. On Lot 1, basement and ramp excavation was proposed to within 1 m of the north-eastern, north-western and south-western boundaries. Maximum excavation depths of up to approximately 16 m were expected for construction of the basement car park, with the depth of excavation increasing in the north-westerly direction.

3.2 Amended Layout

The amended layout comprises:

- a medium density development including four residential buildings up to 4 storeys over a common basement. Excavation to depths of up to 3 m is proposed to within 1 m of the northern boundary, with the main excavation for the lower basement (ie to depths of 8 – 16 m) being to within 6 m, 15 m and 6 m of the north-eastern, north-western and south-western boundaries respectively;
- a mixed use building fronting Collins Street with basement and car park excavation to depths up to 6 m to within 1 m of the southern boundary and within 3 m of the northern boundary.

From the geotechnical perspective, the principal changes from the original layout are a reduction in the lower basement footprints, and therefore generally increased offsets from the boundaries (in particular along the north eastern section of the site adjoining the existing dwellings).

The amended layout is shown on Drawing 1, with a geotechnical Section given in Drawing 2, both in Appendix B.

4. Comments

4.1 General

We have viewed the preliminary architectural plans (Job No 1521) prepared by Kennedy Associates Architects. Furthermore, an inspection of the site by the writer on 30 July 2018 confirmed that, with the exception and additional grass growth on Lot 202, surface conditions were virtually unchanged from those observed at the time of the June 2016 investigation. It is therefore considered that the comments given in the June 2016 report are applicable to the amended layout. This report should there be read in conjunction with DP Project 38145.01 dated 1 June 2016. Comments relevant to design and construction practice with specific reference to the amended layout are summarised in the following sections.

4.2 Subsurface Conditions

The previous investigations have indicated that subsurface conditions underlying the site include topsoil and filling (in parts) to depths of 0.1 – 0.3 m overlying stiff to very stiff clay with sandstone intersected at depths of 0.8 – 4.3 m. The sandstone was initially extremely low to very low strength becoming medium to high strength at depths of 1.3 – 5.1 m and continued to the termination depths of 6.5 – 14.1 m.

Groundwater levels were recorded at depths of 2.0 – 12.5 m (RL 13.0 – 16.1 m AHD following completion of installation. It is noted however that groundwater levels are transient and will vary over time.

4.3 Excavation Conditions

Excavation for the proposed basement car park will necessitate cuts of up to about 16 m to achieve the lowest basement floor level at RL 11.6 m AHD. Within this depth of excavation, a wide range of materials will be encountered grading from topsoil and residual soil near the surface to very low and medium strength, fractured to highly fractured sandstone below depths of 1.4 – 5.0 m and high strength, slightly fractured sandstone below depths of 2.0 – 6.0 m.

The methods used in the drilling of the boreholes provide an indication and guide to the excavatability of the materials to be removed in bulk. Down to the depth of auger refusal (1.0 – 5.8 m), it would be expected that the excavation could be readily carried out using hydraulic equipment fitted with excavation buckets possibly with some light ripping in the weathered sandstone. Below these depths though, medium to high strength sandstone will require medium to heavy rock breaking, rock sawing, rock grinding and/or ripping equipment. Percussion methods possibly in conjunction with rock sawing or grinding equipment would be required for detailed excavation work.

The excavation of rock is dependent both upon rock mass characteristics, primarily the spacing and orientation of jointing and rock strength, as well as the equipment used and skill of the operator. All tenderers should be advised to make their own judgement of the excavatibility of the strata based upon their experience and on inspection of the rock core.

4.4 Vibrations Induced by Excavation Plant

Vibration and noise associated with rock excavation, particularly the use of rock hammers, may be unacceptable to neighbouring properties and operating restrictions are likely to be required. It is recommended that dilapidation surveys be carried out on adjoining structures (or other nearby structures that potentially may be susceptible to vibration damage), thereby providing a baseline for comparison in the event of possible claims for damage.

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibration within acceptable limits. There are three aspects of vibration which need to be assessed:

- Effects on structures;
- Effects on architectural finishes; and
- Effects on humans.

From current knowledge of site conditions and proposed works, the guideline (DIN4150-3-1999) and standard (AS2670.2-1990) detailed in Appendix C are considered appropriate documents on which to base the management of ground vibration.

From current information, it is considered that the structures adjacent to the site can withstand vibration levels which are higher than those required to maintain the comfort of their occupants. Human comfort is therefore of more concern and a human comfort criterion is indicated for managing vibration levels. The control parameters proposed are the instantaneous vector sum peak particle velocities (VSPPV) of the directional components of any installed particle velocity sensors. These components of peak particle velocity are designated PPVr (radial), PPVt (transverse) and PPVz (vertical) or PPVi (any direction i).

Based on the guidelines in Appendix C (extended from component-based criteria to vector sum criteria), it is recommended that a Provisional Allowed Vibration Limit of 10 mm/sec (VSPPV) be set during normal working hours, at foundation level of the potentially affected building/s. This limit, based on Australian Standard AS2670.2-1990, may need to be modified as the result of future building condition surveys.

It must be noted that comparing the vector sum peak particle velocity against the Provisional Allowed Limit is a conservative measure, as many guidelines are based only on vertical components. The measurement of vector sums is considered a logical and justifiable extension, as these vibration levels are real (albeit in variable directions) and can exceed the Provisional Allowed Limit even when the orthogonal components do not.

DP maintains a database of vibration trial results which can provide guidance for the selection of plant. Trial data is dependent on site conditions and equipment, hence actual vibration levels may differ from predictions and a specific trial is recommended at the commencement of rock excavation. The database suggests buffer distance ranges, such as those shown for selected plant in Table 1 (following page), which should be maintained between excavation plant and adjacent buildings, in order to reduce the likelihood of vibration exceeding the Provisional Allowed Vibration Limit. These estimates should be examined in relation to the distances between adjacent buildings and the proposed excavation footprint, in order to select suitable plant.

It is noted that people may find vibration levels above about 3 – 5 mm/s as being strongly perceptible to disturbing, even though they may not cause structural damage to structures. Hence, complaints from neighbours are possible and some reassurance, possibly by vibration monitoring, is likely to be necessary. Consequently, the excavation process should include:

- Notification of neighbouring occupiers of the proposed timing of the excavation so that any vibration sensitive items can be secured;
- Excavation of loose or rippable sandstone blocks by bucket or single tyne attachments prior to commencement of rock hammering;
- Progressive breakage from open excavated faces;
- Use of rock hammers in short bursts to prevent generation of resonant frequencies;
- The movement of large blocks away from the structures prior to breaking up for transport from site;

- Commencement of excavation as far away from other structures as possible and monitor vibration whilst working towards these buildings;
- The use of rock sawing or grinding equipment adjacent to the site boundaries, noting that this equipment also reduces impact damage, overbreak and loosening of the rock mass, typically resulting in reduced requirement for rock support measures.

Table 1: Approximate Buffer Distances for Excavation Plant

Provisional Allowed Vibration Limit:	10 mm/s PPVi
Likely equivalent maximum vector sum:	14 mm/s VSPPV
Excavation Plant	**Buffer Distance**
Rock Saw on Excavator (1)	1.2 m
Ripper on 20t Excavator	1.0 m
Rock Hammer < 500 kg operating weight	4 m
Rock Hammer 501 – 2000 kg operating weight (2)	10 m

Notes:

1. Buffer distances for rock hammers may be reduced by prior saw cutting along, or parallel to, excavation boundaries, to reduce vibration transmitted in surface waves. These cuts should be progressively deepened and extended laterally, to maintain a barrier to direct surface waves between the hammer and the structures to be protected;
2. Buffer distances for rock hammers increase with increasing hammer operating weight up to approximately 2000 kg, however a trend reversal is apparent in currently available data for heavier hammers (on heavier carriers), which is attributed to more efficient application of force and less carrier vibration;
3. Loading effects from adjacent buildings may reduce vibration levels, to enable boundary saw cuts with few exceedances; and
4. The use of diamond saw, may produce negligible foundation vibration, even when cutting immediately adjacent to those foundations.

4.5 Disposal of Excavated Materials

Under the Protection of the Environment Operation Act (1997), the burden of proof that materials received by a landfill or fill site meet the environmental criteria for proposed land use rests on the waste/fill receiving site. Inspection and testing will need to be carried out to classify the spoil in accordance with the Waste Classification Guidelines (2014) prior to removal from site. The type and extent of testing undertaken would depend on final use or destination of the spoil and requirements of the receiving site. As a minimum, allowance should be made during bulk excavation to stockpile fill materials separately from the underlying residual soils and rock to enable the best possible waste classification of the natural soils/rock to be achieved.

As a geo-environmental consultant, Douglas Partners Pty Ltd has the capabilities to assist excavation contractors in classifying materials and negotiating disposal, if required.

4.6 Excavation Support

The proposed basement excavation will be irregular in layout and depth. The deepest section is mostly square and up to about 16 m depth. The perimeter varies in distance from the lot boundaries between approximately 2 m and 8 m. Therefore different strategies could be used to support the excavations. Extreme care will obviously be required when working adjacent to existing structures or nearby properties.

4.6.1 Soil Profile

The soils (including extremely weathered rock) exposed in cut to maximum estimated depths of between 1.0 m and 5.1 m will not be able to stand vertically without support. Where space permits it will be practicable to batter the sides of the excavation and in this regard, it is suggested that maximum temporary batters of 1:1 (H:V) be adopted for preliminary design purposes.

Where excavation is close to the boundaries, precluding batters (for example along parts of the northern boundary) either temporary shoring or the construction of permanent walls in advance of excavation designed to shore up the boundaries will be necessary. The design of shoring will need to take account of lateral loads induced on the basement walls due to structures on adjoining properties.

Where adjacent structures are within the zone of influence of the excavation (such as can be expected along parts of the northern boundary of the site and along the southern extent of the proposed access ramp) it will be necessary to adopt a construction methodology which accounts for these loads. In the absence of details of existing footings this could comprise installation of anchored soldier piles (drilled at maximum 2.4 m spacings) with close shuttering or shotcrete infill panels, contiguous piling or the construction of blockwork walls in conjunction with panelled excavation. Where founding details of adjacent structures cannot be determined, contiguous piling must be allowed for where the excavation is within a 1:1 (H:V) zone of influence of the site boundary.

Where piles have been included within a retaining wall design and are to provide support to structures or pavements, anchoring may be required as a cantilever system may not be sufficiently rigid to avoid excessive lateral surface deflections. The piles should be installed to at least 1 m below the base of any proposed excavation, which will require a high torque boring rig. Where soldier piles are found above the excavation level, anchors may need to be installed in the toe of each pier to provide support and restraint.

4.6.2 Rock

Due to the presence of moderately to steeply-inclined jointing within the medium to high strength rock, and potential for "wedge-type" failures within the batters, allowance will need to be made for the support of the sandstone over the full depth of the excavation. As the support requirements will depend on a number of factors including extent of disturbance during excavation, orientation (bearing), persistence (lateral continuity) and spacing (horizontal separation) of jointing, clay infilling of open jointing and groundwater, detailed design can only be undertaken following inspection of the excavated batters.

As a guide, preliminary design should allow for the application of a steel mesh-reinforced shotcrete layer with a minimum nominal thickness of 150 mm where permanent support is required or 75 mm for temporary support. The installation of a grid of rock bolts of nominal 1.5 m spacing (vertically and horizontally) may also be required. Whilst the required bolt lengths can only be determined following assessment of fracture characteristics, it is suggested that preliminary design be based on a minimum length of 3 m with working bond strength of 1000 kPa to be adopted for medium strength rock. Drainage behind all permanent linings will be required to ensure long-term dissipation of groundwater pressures and could include vertical drainage strips at maximum 2 m horizontal spacings.

It is suggested that inspection of excavations be made by an experienced engineering geologist or geotechnical engineer during the course of the bulk excavation at maximum 2 m depth intervals. The inspections would enable assessment of potentially unstable blocks which may require local bolting or anchoring in addition to the nominal bolting. Furthermore, shotcrete should be placed over the full height of the excavation unless the sides of the excavation are supported by backfilled retaining walls.

4.7 Retaining Structures

It is suggested that earth pressures on cantilever retaining walls due to the retained soils be based on a triangular pressure distribution calculated as follows:

$$\exists_z = \uparrow.Ka.z$$

where	$\exists_z$	=	horizontal pressure at depth z
	$\uparrow$	=	unit weight of retained soil
		=	20 kN/m^3
	Ka	=	active lateral earth pressure coefficient
		=	0.3 for stiff clays and horizontal compacted filling
		=	0.15 for very low to medium strength rock

Design must also make allowance for the ground slope behind any retaining structure (as the earth pressure coefficients given above are for horizontal backfill only). "At rest" earth pressure coefficients (K_o) of 0.6 and 0.25 in clay and very low to low strength rock respectively, will be appropriate where support must be provided to structures and where movement-intolerant services are present within the zone of influence of the retaining walls.

Drainage must be provided behind the walls, but allowance must also be made for a partial hydrostatic head over the top 1 m of wall (to accommodate short-term inundation during storm events) and for all superimposed or surcharge loads that will occur.

4.8 Groundwater

The results of the investigation indicated that the standing groundwater table (approx. RL13 to RL16) is within the rock profile and above the proposed excavation level (RL11.9). There is also a potential for groundwater inflow into the excavation through open bedding planes or joints and at the clay/rock interface particularly during and following inclement weather.

Based on previous experience in the Kiama area, it is considered that inflows during bulk excavation will be controllable by pumping from suitably located collector sumps,\ but in the long term, the requirement for drainage behind perimeter walling (including any shotcrete walls) and underfloor drainage (with discharge via a permanent pump system) will be required as part of the final design.

If a permanent pump-out system (apart from that used on a periodic basis to drain overland flows which enter the basement from the driveway) is not preferred by the client or not allowed by the consent authority, then allowance will need to be made for watertight retaining walls in conjunction with a 'tanked' basement. Based on information available from the previous investigation, preliminary design could be based a static water level at RL 16. Notwithstanding this, as groundwater levels are transient and can fluctuate over time, monitoring of standpipe piezometers is recommended during the design phase.

As the basement is below surrounding ground levels, allowance will need to be made for draining any overland flows which enter the basement from the driveway via a pump-out system.

4.9 Foundations

It is expected that at least medium to high or high strength sandstone will be encountered at the base of the excavation across the majority of the site (with the exception of south-eastern end, which will be within 1.5 m of medium strength sandstone). As a result, it is recommended that all footings be founded on a uniform stratum of at least medium strength sandstone. The main advantage with founding on rock is that settlements (total and differential) would be minimal.

Footings could comprise pad and strip footings for the basement excavation, where rock is exposed at bulk excavation level. For parts of the commercial space on the south-eastern side of the site, bored piers may be required. Alternatively, deep excavated pad footings could be used. In this case, Workcover require that the excavation is to be supported while the steel reinforcement is tied in place, unless the reinforcement mats are pre-fabricated at the surface and lowered into place immediately before the concrete is placed.

Footings could be proportioned using an allowable base bearing pressure on at least medium strength rock of 2,500 kPa. Using this bearing pressure, a 1.2 m square pad footing will be required to support a working load of 3,500 kN. Footings should be inspected by a geotechnical engineer and footing depths adjusted where necessary, to ensure founding on a uniform bearing stratum of appropriate strength.

The weathered sandstone underlying the site would be prone to soften rapidly on contact with water in any open excavation conditions. For this reason, it would be appropriate to provide for the blinding of all pad and strip footing excavations immediately following inspection and approval, prior to the placement of concrete.

4.10 Ground Slabs

The basement floor slabs can be designed as slabs-on-ground supported by the underlying bedrock that will be exposed at the prepared surface. Floor slab design could be based on a subgrade CBR of 7% for at least medium strength sandstone with sub-floor drainage provided.

On grade slabs for the driveway and near existing ground surface walkways can also be designed as slabs-on-ground supported by the underlying clays and weathered bedrock that are likely to be exposed at the prepared surface. Floor slab design could be based on a subgrade CBR of 3% for clays and weathered bedrock with sub-floor drainage provided. Articulation and control jointing should be provided to allow for differential movements in the variable subgrade, particularly at the clay/rock transition (if exposed).

5. Summary

A review of existing geotechnical information and preliminary architectural plans has been undertaken to provide comment on the applicability of the DP June 2016 report to the amended layout. In summary, it is considered that the comments given in the June 2016 report are still applicable, with the main recommendations summarised in Section 4.2 – 4.10 of this report. The preliminary design drawings are to be subject to geotechnical review.

6. Limitations

Douglas Partners (DP) has prepared this report for this project at Lot 202 DP1054190 Meares Pl & Lot 1 DP743509 Collins Street Kiama in accordance with DP's proposal WOL180342 dated 23 July 2018 and acceptance received from Mr Peter V'Landys of Kiama Shores Pty Ltd dated 24 July 2018. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Kiama Shores Pty Ltd for this project only and for the purposes 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. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

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

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

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report
Douglas Partners

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

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

Copyright

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

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

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

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

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

Reports
The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

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

Transported soils may be further subdivided into:

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

Rock Descriptions

Douglas Partners

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 2007. The terms used to describe rock strength are as follows:

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

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Degree of Fracturing

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

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

Rock Descriptions

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

- Asphalt
- Road base
- Concrete
- Filling

Soils

- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks

- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks

- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks

- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

Appendix B

Borehole Logs (Bores 1, 2, 101 – 103)
Drawings (2 sheets)

TEST BORE REPORT

BORE No: 1

CLIENT: SEBASTIAN BUILDERS
PROJECT No: 38145
DATE: 23-1-03

PROJECT: PROPOSED UNIT DEVELOPMENT
SURFACE LEVEL: 25.5
SHEET 1 **OF** 1

LOCATION: LOT 202 MEARES PLACE, KIAMA
DIP OF HOLE: 90°
AZIMUTH:

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0 – 0.75	SILTY CLAY - stiff dark brown silty clay					
0.75 – 1.33	SANDSTONE - extremely low strength, purple brown sandstone		S			9,14/150mm
1.33 – 6.53	SANDSTONE - medium then high strength, slightly weathered to fresh, highly fractured then slightly fractured, brown fine grained sandstone	1.66m: J40° P,S	C	87	41	
2.62 – 2.81		CORE LOSS 190mm				
3.36 – 3.5		CORE LOSS 140mm	C	89	81	
		3.6-3.65m: J45° P,S				
		4.1-4.2m: J45° P,S	C	100	96	
		4.9-4.94m: J45° P,S	C	100	83	
		5.04-5.1m: J30° P,S	C	100	100	
		6.44-6.53m: J50° U,S				
6.53	TEST BORE DISCONTINUED AT 6.53m					

Column headings also shown: Degree of Weathering (EW, HW, MW, SW, FS, FR); Graphic Log; Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High); Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00).

RIG: GEMCO 210B
DRILLER: BOERS
LOGGED: CASTRISSIOS
CASING: 1.3m

TYPE OF BORING: SPIRAL FLIGHT AUGER THEN NQ CORING

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	PL point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials: MJT
Date: 2/03

Douglas Partners
Geotechnics · Environment · Groundwater

TEST BORE REPORT

CLIENT: SEBASTIAN BUILDERS
PROJECT: PROPOSED UNIT DEVELOPMENT
LOCATION: LOT 202 MEARES PLACE, KIAMA

PROJECT No: 38145
SURFACE LEVEL: 20.4
DIP OF HOLE: 90°

BORE No: 2
DATE: 23-1-03
SHEET 1 OF 1
AZIMUTH:

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0 – 0.3	TOPSOIL - dark brown					
0.3	SILTY CLAY - stiff light brown high plasticity silty clay					
1			S			2,5,7 N=12
2			S			4,6,9 N=15
3	Becoming gravelly		S			5,8,12 N=20
4.3	SANDSTONE - extremely low strength brown to orange brown sandstone		S			13,11,10/90mm
4.8	'V-BIT' refusal	CORE LOSS 280mm				
5.08	SANDSTONE - medium then high strength, slightly to moderately weathered highly fractured to fractured, fine grained sandstone	5.42-5.47m: J40° P,S 5.94-6.06m: J50° P,S	C	80	59	
		6.8-6.94m: J80°, J50° P,S,healed	C	100	0	
6.92		CORE LOSS 80mm	C	40	0	
7.0						
7.04		CORE LOSS 50mm				
7.09	7.09-7.43m Highly weathered, light brown	7.22-7.21m: J45° P,S,FeS	C	100	77	
		7.73-7.84m: J60° U,R,FeS	C	100	53	
8	8.0-8.19m: Highly weathered, light brown		C	100	31	
8.19	TEST BORE DISCONTINUED AT 8.19m					

RIG: GEMCO 210B
DRILLER: BOERS
LOGGED: CASTRISSIOS
CASING: 4.8m
TYPE OF BORING: SPIRAL FLIGHT AUGER THEN NQ CORING
WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A auger sample	Pl point load strength I_s (50)MPa
B bulk sample	S standard penetration test
C core drilling	Ux x mm dia. tube
pp pocket penetrometer (kPa)	V Shear Vane (kPa)

CHECKED:
Initials: MJT
Date: 2/03

Douglas Partners
Geotechnics · Environment · Groundwater

BOREHOLE LOG

CLIENT: Kiama Shores Pty Ltd
PROJECT: Proposed Unit Development
LOCATION: Lot 202, Meares Place, Kiama

SURFACE LEVEL: 18.1 AHD
EASTING: 303333
NORTHING: 6161700
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 38145.01
DATE: 28/4/2016
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water	Well Construction Details
0.13	CONCRETE - 130mm thick concrete slab (no reinforcing steel noted).	D/E	0.2				Gattic Cover with Concrete Seal
0.3	FILLING - fine to medium grained gravel (crushed latite) with some fine grained sand and a trace of silty clay (Basecourse)	D/E	0.5				Bentonite Seal; Blank Pipe
0.4	FILLING - orange mottled brown and grey, silty clay with trace fine to medium grained gravel and sand	D/E	1.0				
		D	1.0–1.45		3,6,8 N = 14		2mm Filter Sand; Slotted Pipe
1.8	SILTY CLAY - stiff to very stiff, orange brown mottled grey silty clay, damp					05-05-16	
2.2	SANDSTONE - very low strength, highly weathered, medium grained, brown sandstone						Base Cap

Bore discontinued at 2.2m
Auger Refusal on Low to Medium Strength Sandstone

RIG: DT100 **DRILLER:** Ground Test (W.G) **LOGGED:** DJM / KH **CASING:** NIL
TYPE OF BORING: 110mm Spiral Flight Auger (SFA)
WATER OBSERVATIONS: No Free Ground Water Observed
REMARKS: Location coordinates are in MGA94 Zone 56.

SAMPLING & IN SITU TESTING LEGEND

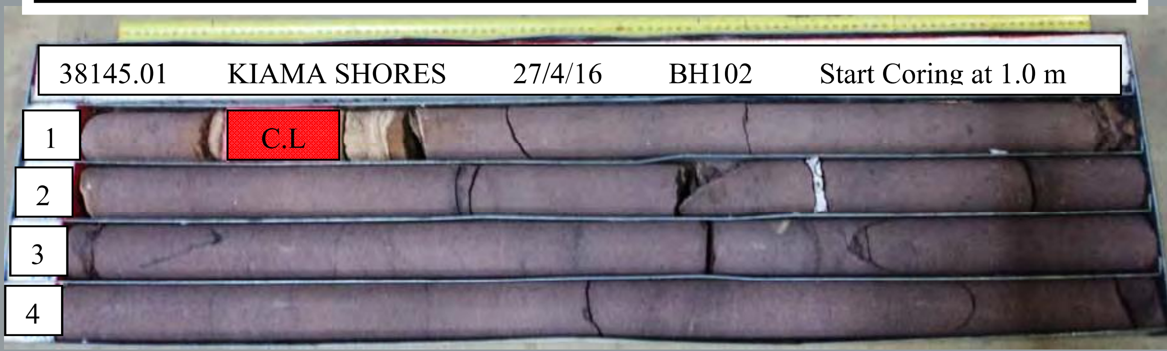
A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED UNIT DEVELOPMENT - KIAMA

BORE 102 PROJECT 38145.01 MAY 2016



1.0 – 5.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED UNIT DEVELOPMENT - KIAMA

BORE 102 PROJECT 38145.01 MAY 2016

5.0 – 10.0m

D O U G L A S   P A R T N E R S   P T Y   L T D

PROPOSED UNIT DEVELOPMENT - KIAMA

BORE 102   PROJECT 38145.01   MAY 2016

10.0 – 14.1m

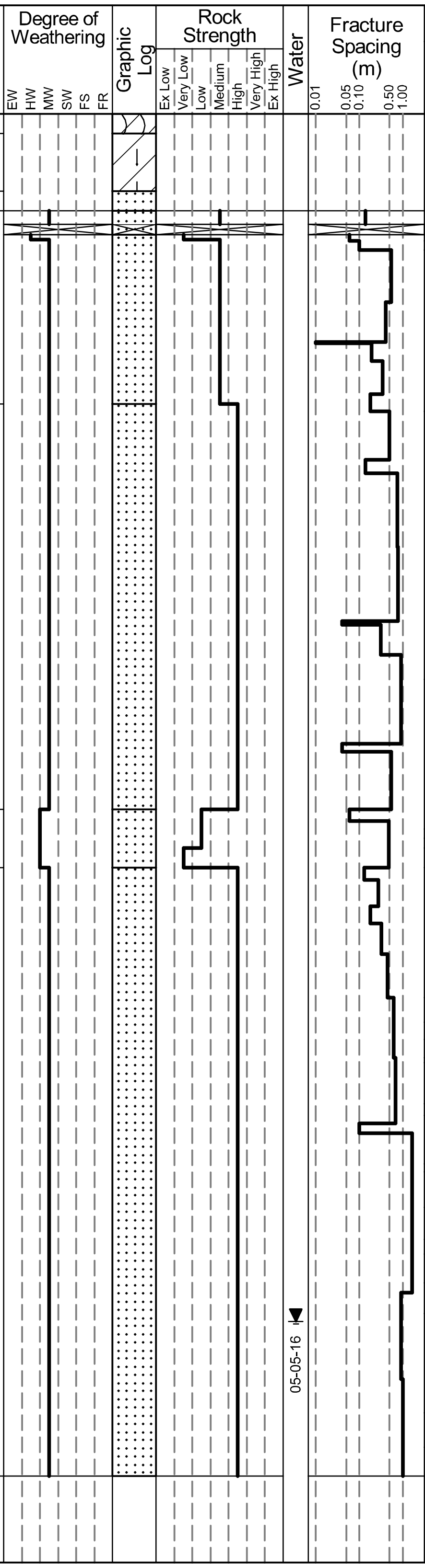
BOREHOLE LOG

CLIENT: Kiama Shores Pty Ltd
PROJECT: Proposed Unit Development
LOCATION: Lot 202, Meares Place, Kiama

SURFACE LEVEL: 25.5 AHD
EASTING: 303277
NORTHING: 6161735
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 38145.01
DATE: 27/4/2016
SHEET 1 OF 1

Depth (m)	Description of Strata
0.2	TOPSOIL, brown, clayey silt with some rootlets, moist
0.8	SILTY CLAY, stiff to very stiff, orange brown, silty clay, damp
1.0 – 1.25	SANDSTONE, very low to low strength, highly weathered, red brown, fine grained sandstone.
	SANDSTONE - medium strength, moderately weathered, slightly fractured, red brown, fine to medium grained sandstone
3.0	SANDSTONE - high strength, moderately weathered, slightly fractured, red brown, fine to medium grained sandstone
7.2	- coarse grained sandstone from 7.0m to 7.3m
7.8	SANDSTONE - very low and low strength, highly to moderately weathered, fractured, light brown, fine to medium grained sandstone
	SANDSTONE - high strength, moderately weathered, slightly fractured to unbroken, fine to medium grained, tuffaceous sandstone
14.1	Bore discontinued at 14.1m Limit of Investigation



Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)

Unless otherwise stated, rock is fractured along rough (ro), planar (pl), bedding dippings (B) at 0 - 10° or joints (J) at 30 - 45°
1.14m: CORE LOSS: 110mm
1.31m: B, 20°, clay
1.41m: & 1.95m: J 35° clay
2.56m: J 65° clay
3.02m: J 45° clean
3.08m: J 70° healed, tight
3.72m: J 70° iron oxide
4.48m: J 30° iron oxide
4.8m: J 45° clean
5.04m: J 30° clean
5.6m: J 20° and 70° stepped, ro, clean
5.85m: J 60° clean
7.2m: & 7.32m: B 5-10° clay
7.4m: J 30° clean
7.5m: J 30-35° curved, ro, clean
7.6m: J 60° clay
7.8m: J 45° clay
8.82m: J 85° undulating, ro, clean
9.15m: B 10° clay
9.4m: J, 25° and 45° stepped, ro, clean
9.58m: J 30° and 70° stepped, ro, clean
9.77m: J 35° iron oxide
9.82m: J 80°-90° undulating, ro, clean
10.25m: J 80° undulating, ro, clean
10.37m: to 10.55m: J (x3) 30°-40° clean
10.87m: J 45° clean
10.95m: J 45° clean
11.32m: J 45°to 70° curved, ro, clean
11.65m: J 35° clean
11.8m: J 45° undulating, ro, clean
12.1m: J 70° undulating, ro, clean
12.42m: to 13.33m: J(x3), 80°, un, ro, clean
13.45m: J 45° clean
13.8m: J 20° clean

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
D/E			
D/E			
D/E			
C	94	90	PL(A) = 0.8 PL(A) = 0.8
C	100	95	PL(A) = 2.5 PL(A) = 1.5 PL(A) = 1.9
C	100	86	PL(A) = 3.0 PL(A) = 1.8
C	100	89	PL(A) = 2.5 PL(A) = 1.6 PL(A) = 1.6
C	100	98	PL(A) = 3.0
C	100	100	PL(A) = 1.4 PL(A) = 2.9

RIG: DT100 **DRILLER:** Ground Test (W.G) **LOGGED:** SI **CASING:** 1.0 m HQ
TYPE OF BORING: 110mm SFA to 1.0m, NMLC Rotary Core to 14.5m
WATER OBSERVATIONS: No Free Ground Water Observed Whilst Augering
REMARKS: Location coordinates are in MGA94 Zone 56.

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED UNIT DEVELOPMENT - KIAMA

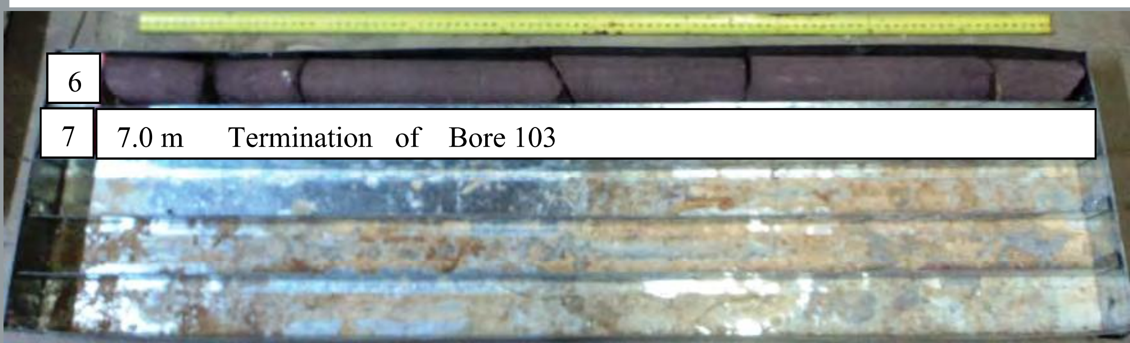
BORE 103 PROJECT 38145.01 MAY 2016

1.5 – 6.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED UNIT DEVELOPMENT - KIAMA

BORE 103 PROJECT 38145.01 MAY 2016



6.0 – 7.0m

BOREHOLE LOG

CLIENT: Kiama Shores Pty Ltd
PROJECT: Proposed Unit Development
LOCATION: Lot 202, Meares Place, Kiama

SURFACE LEVEL: 18.8 AHD
EASTING: 303273
NORTHING: 6161697
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 38145.01
DATE: 28/4/2016
SHEET 1 OF 1

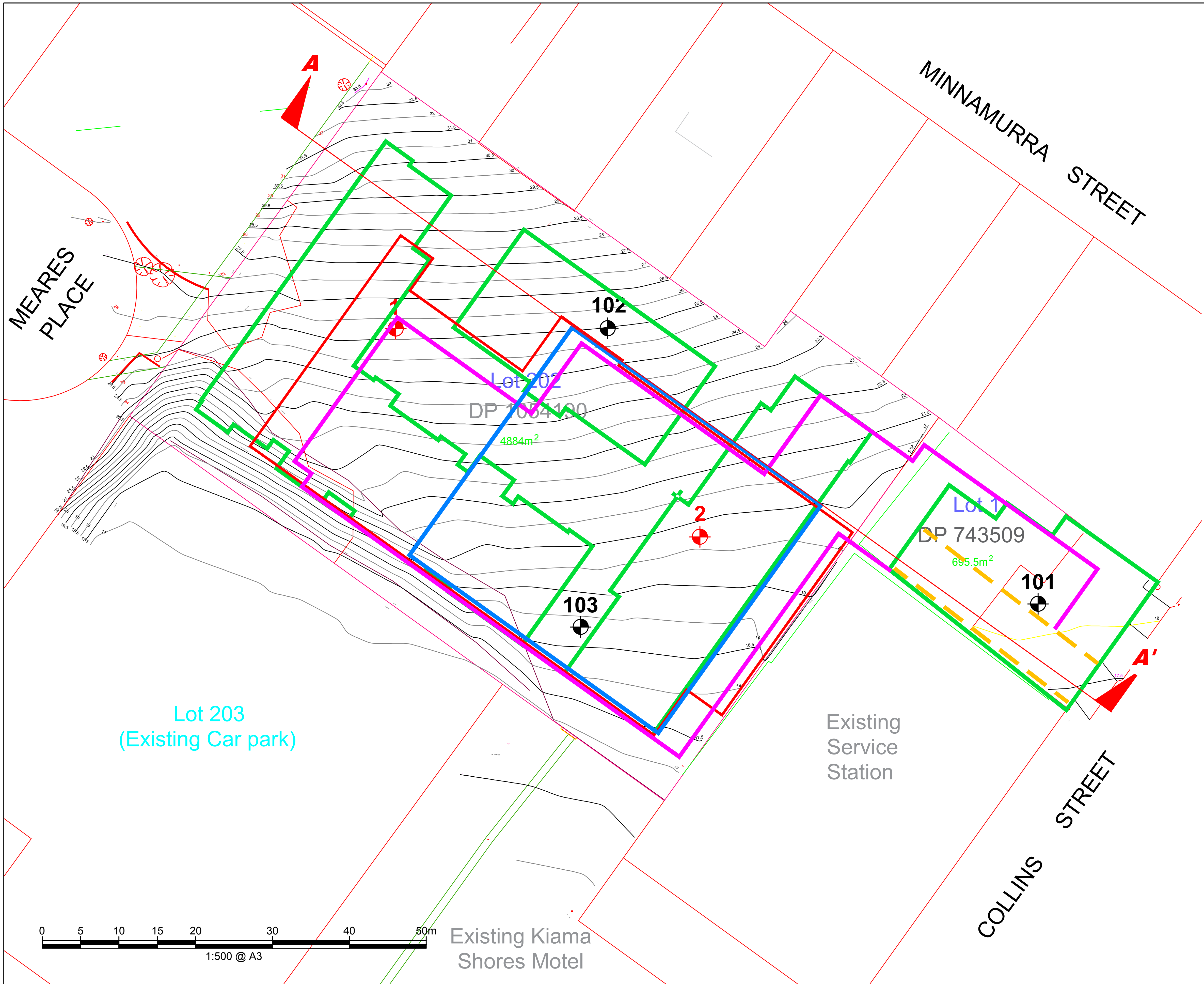
Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL - brown clayey silt with some rootlets, moist		Unless otherwise stated, rock is fractured along rough (ro) planar (pl) bedding dippings at 0-10 or ro pl joints at 35 - 45 degrees	A			
0.9	CLAY - brown grey, slightly silty, slightly sandy, clay, damp			A			
1.58	SANDSTONE - extremely low strength, extremely weathered, red brown, sandstone			A, S			6,18,25/50 refusal
2.1	SANDSTONE - very low to low strength, highly weathered, fractured, light grey, fine grained sandstone with some medium strength bands		1.5m: CORE LOSS: 80mm 1.58m: to 1.8, fragmented, clay 1.95m: to 2.10, fragmented, clay 2.3m: B 0° clay coating 3mm 2.9m: to 3.01m J 30°-35° clean 3.12m: J 60° undulating, rough, clean 3.36m: B 5° fe 3.75m: B 5° fe	C	98	80	PL(A) = 2.2 PL(A) = 2.3 PL(A) = 1.7
7.0	SANDSTONE - high strength moderately weathered, fractured to slightly fractured, red brown, fine to medium, grained tuffaceous sandstone	05-05-16	3.9m: J 35° cln 4.08m: J 35° to 40° curved, ro, iron oxide 4.2m: J 40° iron oxide 4.52m: J 25°-45° curved, ro, iron oxide 4.65m: to 4.85m, B 10° iron oxide 5.2m: to 5.6m, J 45°-60° curved, ro, clean 5.75m: J 70° cln 6m: J 45° iron oxide 6.45m: J 45° iron oxide 6.9m: J 45° iron oxide	C	100	100	PL(A) = 2.0 PL(A) = 1.4
	Bore discontinued at 7.0m Limit of Investigation						

RIG: DT100 **DRILLER:** Ground Test (W.G) **LOGGED:** SI **CASING:** HW to 1.5m
TYPE OF BORING: 110mm SFA to 1.5m, NMLC Rotary Core to 7.0m
WATER OBSERVATIONS: No Free Ground Water Observed Whilst Augering
REMARKS: Location coordinates are in MGA94 Zone 56.

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics | Environment | Groundwater



Locality Plan

LEGEND

- Borehole Location (Project 38145.01)
- Previous Borehole Location (Project 34145.00)
- Approx. Site Boundary
- Cross Section A - A' (refer Drawing 3)
- Building footprint
- Level 00 (RL17.86)
- Level -01 Basement (RL14.56)
- Level -02 Basement (RL11.6)
- Access Ramp Beneath

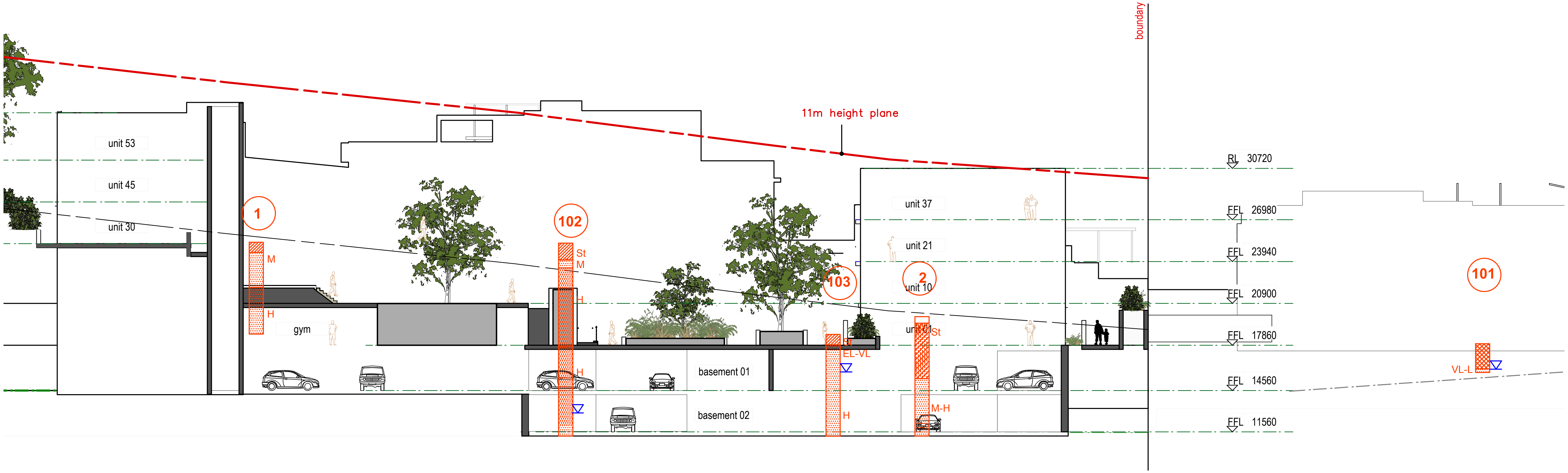
NOTE: Base drawing from DTB Architects Pty Ltd
(Drawing 1512-003-P8 Site Survey Plan)

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Kiama Shores Pty Ltd		TITLE:	**Borehole Location Plan (Amended Layout)**
OFFICE: Wollongong	DRAWN BY: DJM		**Proposed Mixed Use Development**
SCALE: As shown	DATE: 1.8.2018		**Lot 1 DP743509 and Lot 202 DP1054190 Meares Place, Kiama NSW**

N
MGA

PROJECT No:	38145.04
DRAWING No:	1
REVISION:	0



SECTION A-A'

LEGEND

- Filling
- Clay
- Silty clay
- Sandstone

St Stiff
VL Very low strength
L Low strength
M Medium strength
H High strength

Water level

NOTE:
1: Base drawing from DTB Architects Pty Ltd
(Dwg 1821-DADA25C, dated 18.7.2018)
2: Test locations are approximate only and
are shown with reference to existing features.

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Kiama Shores Pty Ltd
OFFICE: Wollongong | DRAWN BY: PSCH
SCALE: 1:300 @ A3 | DATE: 1.8.2018

TITLE: **Cross Section A-A' (Amended Layout)**
Proposed Mixed Use Development
Lot 1 DP743509 and Lot 202 DP1054190 Meares Place, Kiama NSW

N

PROJECT No: 38145.04
DRAWING No: 2
REVISION: 0

Appendix C

Ground Vibrations
Design of Braced Excavations

Ground Vibrations

Ground vibrations can be described by measurement of the acceleration, velocity or displacement of the ground particles at one or more locations. Triaxial geophone sensors for example can measure the peak velocities of radial, transverse or vertical particle motion (designated PPVr, PPVt and PPVz respectively and PPVi for any directional component) within selected sample periods and peak velocities can also be determined in the resultant direction of particle motion, from calculations of instantaneous vector sums throughout the sample period. Vector sum velocities are designated VSPPV, or in many cases simply PPV.

There are three aspects of vibrations which need to be assessed:

1. Effects on structures
2. Effects on architectural finishes
3. Effects on humans

Numerous standards and guidelines exist worldwide which provide a basis for these assessments. Their focus varies from structural damage to human comfort and from transient to intermittent to continuous vibrations. Most provide guideline vibration limits for protection against damage or human discomfort, however these limits are not always consistent and application of a particular standard or guideline should be based on the expected types of vibrations, the types and conditions of the potentially affected buildings and the potential for discomfort of their occupants.

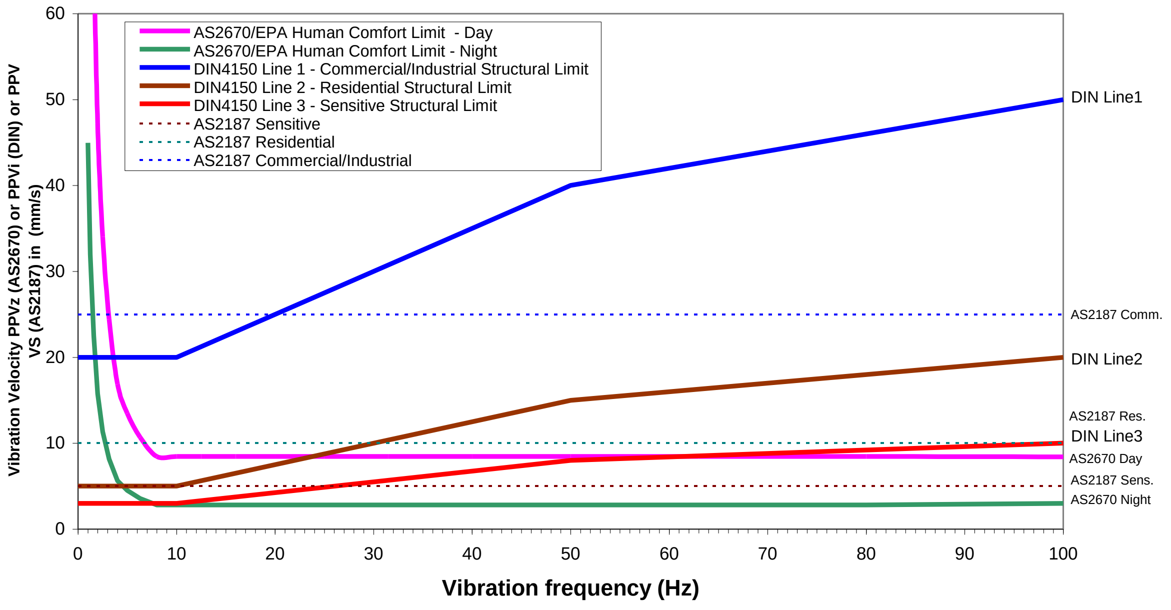
Both the guideline and the vibration limits should be determined on a case by case basis and the adopted limits (damage and human comfort or the lower of the two) may vary from guideline values, according to the experience of the vibration consultant, due to the sensitivity of the building or the activities of its occupants. Some applicable guidelines are summarised in the graph below.

Depending on site conditions, proposed works, results of building condition surveys and on-site vibration trials (indicating vibration attenuation rates and dominant vibration frequencies of excavation plant), the standards, guidelines and limits discussed below are considered appropriate for management of ground vibrations generated during rock excavation.

Effects on Structures

The German Standard DIN4150-3 (1999) “Structural vibration – effects of vibrations on structures”, recommends that ground vibrations at foundation level of residential buildings, in good condition bearing on sound rock foundations, be limited to 5 - 15 - 20 mm/s PPVi (at vibration frequencies of 10 - 50 - 100 Hz typical of excavation plant), in order to reduce the potential for structural damage. Higher limits (20 - 40   50 mm/s PPVi) and lower limits (3 - 8 -10 mm/s PPVi) are recommended for commercial/industrial and sensitive buildings respectively. From DP experience where buildings are bearing on loose sand, maximum vibration levels should be significantly reduced to the order of 5 to 7 mm/s VSPPV to reduce the risk of vibration-induced sand densification and settlement.

Guidelines for Evaluating the Effects of Intermittent or Impulsive and Short Term Vibrations on Human Comfort and Structures (Based on AS2670.2/EPA ENCM Ch174 and DIN4150)



Effects on Architectural Finishes

It has been found from experience that even with buildings bearing on rock, vibration levels as low as 10 mm/s VSPPV may cause minor defects such as cracks through rendering, cornices and skirtings. Management of vibrations may require a lowering of structural damage criteria to this architectural damage criterion, or negotiations with owners of affected buildings.

Effects on Humans

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s VSPPV and can be disturbing at levels above 5 mm/s VSPPV. Complaints from residents and building occupants are sometimes received when levels are as low as 1 mm/s VSPPV. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVz for human comfort. Management of vibrations may require a lowering of damage criteria to this human comfort criterion, or negotiations with occupants of affected buildings.

Vibration Dosage

A vibration limit based on a particle velocity allows real time control of excavation using warning systems (e.g. flashing lights) attached to vibration monitors. Occasional exceedances (vibration levels exceeding the allowed limit) are not damaging or disturbing and can be allowed but frequent exceedances should be avoided by changes in excavation methods. The difference between occasional and frequent is difficult to gauge on site but can be assessed using recorded vibration data, on the basis of experience or by application of a vibration dosage criterion.

A vibration dosage value (VDV) can be used to assess the affect of intermittent vibrations (e.g. from bursts of rock hammering) on humans over a defined period. Acceptable dosages (generally VDVz for vertical vibrations found most disturbing by humans) have been defined for occupants of residential, commercial and industrial buildings ("Assessing Vibration: a technical guideline", Department of Environment and Conservation, 2006). Estimates of VDV (eVDV) can be calculated from recorded vibration data and can be compared with recommended maxima of 0.4, 0.8 and 1.6 m/s for residential, commercial and industrial locations respectively, to assess the need to change excavation methods to restore human comfort.

The vibration dosage guideline does not relate VDV to structural damage however it is considered that if the VDV is acceptable from a human comfort viewpoint, vibrations leading to that VDV would be unlikely to cause damage to the corresponding residential, commercial or industrial structure. Management of vibrations may require addition of these vibration dosage criteria to other human comfort or damage criteria, if the frequency of vibration exceedances becomes difficult to assess on site or by experienced-based data review.

DATA SHEET 11-25

Design of braced excavations

Bracing generally consists of vertical sheet piling supported by a series of struts and waylings. The construction sequence is usually

(a) steel sheet piles are driven into the ground
(b) ground is excavated from inside the area enclosed by the piles
(c) walings and struts are installed and tightened as excavation proceeds.

Because of the method of construction and the rigid support given by the struts, pressures on the sheet piling cannot be predicted by traditional earth pressure theories. The usual design procedure follows the method proposed by Terzaghi, using rules similar to those given below:

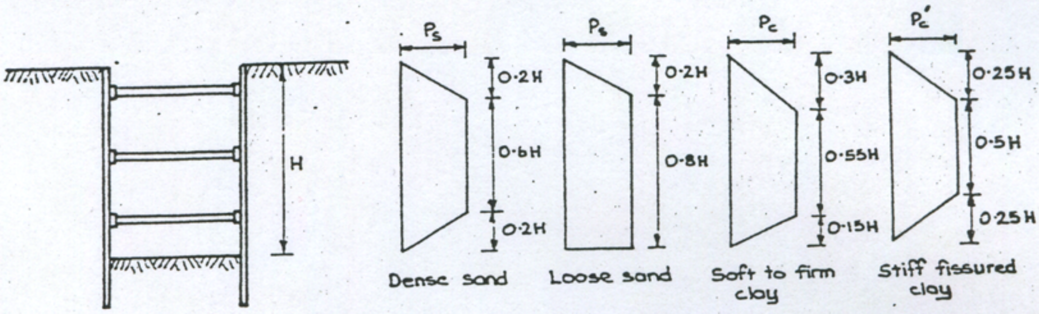


FIGURE 11-25-1 Terzaghi's rules for pressure distributions on braced excavations.

In sand, pressure $p_s = 0.8K_a\gamma.H\cos\delta$

where K_a is the active pressure determined from Data Sheet 11-2
γ is the average soil density
H is the depth of excavation
δ is the angle of wall friction, from Data Sheet 6-5.

In clay, pressure $p_c = \gamma H - 4c_u$

where c_u is the undrained shear strength.

In stiff fissured clays $p_c' = 0.4\gamma H$.

If movement can be reduced to a minimum and construction time is short, this can be reduced to

$p_c' = 0.2\gamma H$

The submerged density is used below the water table and pore water pressures are added to the calculated earth pressures.

In clays, creep effects cause a redistribution of stresses with time. From model tests carried out by Kirkdam, it was concluded that, for long-term design in clays, classical earth pressure theories should be used, based on effective stress parameters.

Reference:
Carter, M (1983) Geotechnical Engineering Handbook