ACT Geotechnical Engineers

ACT Geotechnical Engineers Pty Ltd
ACN 063 673 530

5/9 Beaconsfield Street, Fyshwick, ACT,2609
PO Box 9225, Deakin, ACT, 2600
Ph: (02) 6285 1547
Fax: (02) 6285 1861

30 June 2023
Our ref: OB/C14369

Adhami Pender Architecture
Via email: nabil@adhamipenderarchitecture.com

Dear Sir

PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW

GEOTECHNICAL INVESTIGATION REPORT

We are pleased to present our geotechnical investigation report for a new proposed residence with a basement at 217a Beach Road, in Denhams Beach, NSW.

The report outlines the methods and results of exploration, describes site subsurface conditions and provides recommendations for building footing design, excavation conditions, preparation of subgrades, stability of cut and fill batters, and provides slope stabilisation advice.

Should you require any further information regarding this report, please do not hesitate to contact our office.

Yours faithfully,

ACT Geotechnical Engineers Pty Ltd

Olga Baruleva
Engineering Geologist
BSc (Geology) MPhil *MIEAust*

Reviewed by:

Jeremy Murray
Senior Geotechnical Engineer | Director
FIEAust CPEng Eng Exec NER RPEQ APEC Engineer IntPE(Aust)
Registered Professional Engineer of Queensland (RPEQ) #19719
NSW Professional Engineer Registration #PRE0001487

ADHAMI PENDER ARCHITECTURE

PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW

GEOTECHNICAL INVESTIGATION REPORT

JUNE 2023

ADHAMI PENDER ARCHITECTURE

PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW

GEOTECHNICAL INVESTIGATION REPORT

TABLE OF CONTENTS

ADHAMI PENDER ARCHITECTURE

PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW

GEOTECHNICAL INVESTIGATION REPORT

1 INTRODUCTION

At the request of the client, ACT Geotechnical Engineers Pty Ltd carried out a geotechnical investigation for a proposed new residence with basement at 217a Beach Road, in Denhams Beach, NSW

The project will comprise the construction of a new two-storey residence with a ~3m deep basement cut into the sloping site. The house will be located at the top of a steep cliff, and advice for stabilisation the cliff is also required. The aim of the investigation was to:

(i) Identify subsurface conditions including the extent and nature of any fill materials, soil strata, bedrock type and depth, and groundwater presence.

(ii) Advise on excavation conditions and suitability of excavated material for use as structural fill.

(iii) Provide site classification to AS2870 "Residential Slabs & Footings".

(iv) Advise on suitable footings systems, founding depths, allowable bearing pressures and design parameters for ground slabs.

(v) Provide guidelines for construction of controlled fill platforms.

(vi) Advise on stable batter slopes and retaining wall design parameters.

(vii) Stability assessment of the existing cliff face.

(viii) Advice and recommendations for stabilizing cliff face against slope instability and storm surges.

(ix) Drainage and other geotechnical advice.

2 SITE DESCRIPTION & GEOLOGY

The block is legally described as Lot 2 of DP1270021, in Denhams Beach, NSW. The site is ~1069m^2 in area and is bounded by Beach Road to the west, the residential blocks to the north and south and the coastline of Batemans Bay to the east. At the time of the investigation, the block was vacant.

The site is located on the coastal steep cliff. The residence will be built on the western side of the block that is relatively flat and dips gently east from RL26 to the top of the bank at RL24.6 above sea level. The eastern site of the block is on a steep bank of ~22m high and dips at the angles of ~40° with steep sections reaching 60°. The steep slope covered with bushes, mature trees and other dense vegetation. A stone and mortar retaining wall has been constructed along the upper face to support stairs to the beach. Figure 1 shows the site locality. Figure 2 is an aerial photo showing the present site layout.

The MinView Seamless Geology map (Reference 1) documents the site to be underlain by Late Ordovician age Bolongo Group bedrock, which includes mudstone plus minor quartz sandstone and chert; mafic volcanic rocks.

3 INVESTIGATION METHODS

To establish the subsurface conditions, a Hyundai excavator with a 300mm auger attachment was used to drill three (3) boreholes, designated BH1 to BH3, to ~3m target depth or earlier refusal on bedrock. Boreholes were excavated on 16 June 2023. The subsurface profiles were logged in accordance with AS1726-2017. The locations of the boreholes are shown on Figure 2, and the detailed logs are included in Appendix A.

The slope stability assessment was based on the guidelines on the AGS "Landslide Risk Management Concepts and Guidelines 2007". (Reference 2) and advice on slope stabilisation is provided in Section5.8.

Definitions of geotechnical engineering terms used in the report on the borehole logs, including a copy of the USCS chart, are provided in Appendix B.

4 INVESTIGATION RESULTS

4.1 Subsurface Conditions

The subsurface conditions of the proposed development were investigated by three (3) boreholes designated BH1 and BH3. The borehole logs in Appendix A can be referred to for more detail. Investigation boreholes found the subsurface profile to comprise:

Geological Profile	Typical Depth Interval	Description
TOPSOIL	0.0m to 0.5m	Gravelly Clayey SAND; fine to coarse sand, fine to coarse gravel, black, grass roots, dry to moist, concrete blocks, medium dense.
UNCONTROLLED FILL	0.0m/0.5m to 0.15m/1.6m/Below 2m	Sandy GRAVEL, COBBLES and BOULDERS, & Gravelly Sandy CLAY; fine to coarse sand, low to medium plasticity clay medium angular gravel to 20-25mm (DGB 20), grey, fine to coarse angular gravel of highly weathered siltstone, rounded quartz, brown, grey, concrete and ceramics fragments, dry to moist medium dense, dense, firm to stiff, stiff.
RESIDUAL SOIL	0.15m/1.6m to 1.2m/2.1m	Silty CLAY; low to medium plasticity fines, pale brown, pale yellow, trace of fine to coarse sand, dry to moist, stiff.
BEDROCK	1.2m/2.1m to Below >3m	SHALE; Extremely and Highly weathered, excavated as Sandy CLAY, extremely weak, pale grey, some gravel, fine grained.

Table 1 – Depth of Fill and Bedrock

Borehole No.	Depth of Fill or Unsuitable Material	Depth to Bedrock
BH1	0.15m	1.2m
BH2	>2.0m	Not encountered
BH3	1.6m	2.1m

In the absence of a Controlled Fill Report in accordance with AS3798, the fill encountered must be considered to be uncontrolled.

4.2 Groundwater

Groundwater was not encountered, and the soils and bedrock were dry to moist. However, temporary, perched seepages could be encountered following rainfall within the more pervious soils.

4.3 Slope Stability

The visual assessment of the slope instability was conducted during the site investigation. The steep slope at the eastern end of the site dips east at between 40° and 70°. The slope is approximately ~22m high, ranging from RL24 at the top of the bank to RL2 above the sea level at the slope base on the adjacent beach. Figures 3 and 4 show the site photographs.

Large landslides have recently occurred on the neighbouring blocks to the north and to the south from the site. Recent landslides formed several main scarp zones, on the top faces, undermining the property to the north and removing steps to the beach to the south. Exposed in the rupture surface soils comprises topsoil, clay residual soils and XW mudstone and shale bedrock. A few outcrops of bedrock on the slope top faces comprised flat bedded extremely to highly (XW/HW) weathered fine-grained mudstone and shale. The bedding dips steeply north-west from 55° to subvertical. The bedrock is highly fractured with open and closed fractures and partings spacing from 2cm to 4cm.

It has been assessed that the slope failure is active and will likely progress further as evidenced by steep angles on the slope top faces. The failures have occurred due to saturation of the overburden soils and fractured XW bedrock, which is then sliding down the slope.

The steepness of the slope and low cohesive strength of the overburden soil are contributing to the overall instability of the site. However, recent landslides have not occurred on the subject site yet and landslide processes have been slowed down due to the mature and dense of vegetation on the slope, which have been protecting the surface and holding it together.

The next landslide will occur once soils again become saturated with water sufficiently such that another slab of soil/debris can fall from the top faces. The next landslide of this location could occur in one year or in several years, but it is difficult to quantify. The existing house is founded at the top of the slope, and it is assessed that the small-scale slumps in the soil profile on the steep slope may undermine the house footing and potentially impact on the stability of the proposed house.

Slope instability could also be triggered by scour at the tow of the cliff due to wave action. The existing wave-cut platform has been formed as waves and tides have scoured and undermined the toe of the cliff. This has resulted in progressive cliff recession over many years.

5 DISCUSSION & RECOMMENDATIONS

5.1 Site Classification

Due to the presence of uncontrolled fill materials exceeding 0.4m depth, the site is designated as a Class "P" (problem) site in accordance with AS2870. If the fill is removed, or if footings are founded in the residual soil or extremely weathered material below the fill, a Class "S" (slightly reactive) category can be used in design of new footings. The characteristic ground surface movement "ys", as defined by AS2870 for the range of normal soil moisture conditions is estimated to be between 0mm to 20mm for the encountered subsurface profile described in Section 2.

Should earthworks (cut or fill) be undertaken on the site, or other activities which may cause abnormal moisture conditions to impact the soils within or near the building envelope beyond those addressed herein, the site classification shall be reassessed.

5.2 Building Footings

It is recommended that all footings are founded in XW/HW or better bedrock, which is expected at 1.9m/2.8m depth below existing ground surface. The proposed two-storey structure must be founded on pad footings or bored piers founding in the XW/HW or better bedrock. All footings should be taken below any topsoil, uncontrolled fill and/or disturbed ground.

AS2870 provides "deemed-to-comply" footing/slab designs, which for a Class "M" site includes stiffened rafts, stiffened footing slabs, waffle rafts, and strip and/or pad footings with above ground floors. Footings and slabs should be designed in accordance with the principles of AS2870.

For structures founded at existing grade, footings, including thickened sections of slabs forming footings should be founded in medium dense/very stiff residual soils, below any topsoil or uncontrolled fill soils. Shallow footings could be founded in newly placed or existing controlled fill following removal of any topsoil material (see Section 5.5). Alternatively, footings could be founded on piles extending to weathered bedrock. Piles could comprise bored piers, screw piles or driven piles. All piles/bored piers must be socketed at least 4 pile diameters into the bedrock.

A ~3m deep, single-level basement is expected to expose weathered shale bedrock over most of the foundation, and pad/strip footings founded in the bedrock would be suitable.

Table 2 below gives recommended allowable end bearing pressures for design purposes.

TABLE 2
Recommended Allowable End-Bearing Pressures for Footings

Foundation Material Type	Depth Below Existing Surface Level	Allowable End-Bearing Pressure			Allowable Shaft Adhesion on Bored Piers & Driven Piles	
		Strips	Pads	Piles	Downward Loading	Uplift
Colluvial/Residual Soil	0.15m/1.6m/>2.0m	100kPa	150kPa	200kPa	20kPa	10kPa
XW Bedrock	1.2m/1.6m/>2.0m	300kPa	400kPa	600kPa	60kPa	30kPa
XW/HW or less weathered bedrock	1.9m/2.8m	500kPa	600kPa	1000kPa	100kPa	50kPa

All footings should be inspected and approved by an experienced geotechnical engineer to confirm the foundation material and design values, and to ensure the excavations are clean and stable.

Ground slabs can be constructed on the natural soils or newly placed controlled fill, following the removal of any topsoil and uncontrolled fill material. Following excavation to required level, slab areas on soil should be proof-rolled by a pad foot roller to check for any weak, wet or deforming soils that may require replacement. Suitable replacement fill should be compacted in not thicker than 150mm layers to not less than 95%ModMDD.

Table 3 provides shear strength parameters of the soil and rocks layers at the site and can be used as a guide for foundation and retaining wall design.

TABLE 3
Soil/Rock Strength Parameters and Earth Pressure Coefficients

Soil/Rock Property	Bulk Density γb (kN/m^3)	C (kPa)	Φ' (degrees)	Elastic Modulus (MPa)	Active K_a	At Rest K_o	Passive K_p
Uncontrolled Fill/Topsoil	19	0	20	10	0.49	0.66	2.1
Residual/ Colluvial Soil	20	5	25	25	0.41	0.58	2.45
XW/HW Bedrock	24	30	30	100	0.33	0.50	3.0

γ - Bulk unit weight, Cu - undrained shear strength, Ø – soil friction angle, c – Cohesion

The above values can be used in software programs for design; however, it is recommended that the values for lateral earth pressures in section 5.6 be used as a minimum.

5.3 Excavation Conditions & Use of Excavated Material

The development will have a one-level basement, and a maximum excavation depth of ~3m will be required for the proposed building. Such excavation would require excavation through uncontrolled fill, colluvial/residual soils, and XW and HW shale bedrock. The overburden soil and XW/HW, and HW bedrock can be dug by a medium to large excavator. However, HW/MW & MW, and less-weathered bedrock, which could be encountered below 2m/3m depth, will require heavy excavator or dozer (D8 or D9) ripping, and rock hammering.

The low and medium plasticity colluvial/residual soils can be used in controlled fill construction of building platforms. The weathered shale bedrock is also suitable for fill material, although rock particles should be broken down to <75mm size. The silty topsoil and slopewash material and any high plasticity clay should not be used in controlled fill construction, but could be used in non-structural applications such as landscaping.

If imported fill is required, a suitable select fill material would include a low or medium plasticity soil such as clayey sand or gravelly clayey sand, containing between 25% and 50% fines less than 0.075mm size (silt and clay), and no particles greater than 75mm size.

5.4 Stable Excavation Batters

Temporary site excavations to 1.5m depth should be cut back at no steeper than 1(H):1(V). If required and space allows, deeper temporary cuts can be formed at 1(H):1(V) in soils and at 0.5(H):1(V) in weathered shale bedrock. A geotechnical engineer should inspect all cut batters during construction to confirm stability. Exposed temporary batters should be protected from the weather by black plastic pinned to the face with link-wire mesh, or similar.

During construction, the following recommendations must be followed to maintain stability of all temporary unsupported excavations:

- All equipment/machinery/stockpiles/site sheds and containers are located 1(H):1(V) from the toe of the batters. Trucks and heavy construction plant/equipment and large soil stockpiles must not be located close to the top edge of the batters, especially with the motor idling. Trucks and heavy construction plant/equipment must be located outside the zone of influence (1(H):1(V)) of the excavation batter.

- A bund or dish drain must be constructed along the top edge of all cuts to intercept and divert surface water away from the batters.

- All batter faces must be trimmed when erosion occurs, and loose material cleaned from the face regularly. Therefore, it is recommended that the batter faces are monitored on a daily basis and cleaned of loose material when present.

- Regular inspections by a geotechnical engineer of the batters would be required. As a guide, these inspections by a geotechnical engineer must be conducted on a weekly basis, while a competent person representing the contractor should do daily checks.

- No work must be conducted close to the toe of the batters during rain and 24 hours after. The batters must be re-inspected by a geotechnical engineer following rainfall (about 20mm of rain, or enough rain that the batter faces become wet).

- If deterioration or significant weathering of the batter face occurs, stabilisation/remediation of the batter must be applied. A geotechnical engineer will confirm this recommendation.

Permanent cut & fill batter slopes should be formed at no steeper than 2(H):1(V) in soil and be protected against erosion by shotcreting, stone pitching or other suitable methods. Alternatively, permanent excavations can be supported by structural retaining walls.

5.5 Controlled Fill Construction

For construction of any new fill foundation platforms and road subgrades, it is recommended that:

- Areas be fully stripped of all topsoil and uncontrolled fill material. A stripping depth of up to 1.6m/>2.0m may be required. Stripped foundations should be proof-rolled by a vibratory pad-foot roller of not less than 9 tonne static mass to check for any weak or wet areas that would require replacement. No fill should be placed until a geotechnical engineer has confirmed the suitability of the foundation.

- Controlled fill comprising suitable site excavated or imported sand of not greater than 75mm maximum particle size, be compacted in not greater than 150mm layers to a Density Index of not less than 75% at about OMC.

- Fill placement and control testing be overviewed and certified by a geotechnical engineer at Level 1 involvement of AS3798 – 2007 "Guidelines on Earthworks for Commercial & Residential Developments" (Reference 3).

5.6 Retaining Walls

Retaining walls constructed in open excavation, with the gap between the excavation face and the wall backfilled later, can be designed for an earth pressure distribution given by:

$$\sigma_h = (K\gamma' h) + Kq$$

where,

σ_h is the horizontal earth pressure acting on the back of the wall, in kPa

K is the dimensionless coefficient of earth pressure; this can be assumed to be 0.4 when the top of the wall is unrestrained horizontally, and 0.6 when the top of the wall is restrained (i.e. by building slabs etc.)

γ' is the effective unit weight of the backfill, and can be assumed to be 20kN/m^3 for a lightly compacted soil backfill

h is the height of the backfill, in metres

q is any uniform distributed vertical surcharge acting on the top of the backfill, in kPa

Apart from structural restraints such as floor slabs, resistance to overturning and sliding of retaining walls is provided by frictional and adhesive resistance on the base, and by passive resistance at the toe of the wall. For a natural soil or controlled fill foundation, an ultimate base friction factor (tanδ) of 0.4, base adhesion (c) of 25kPa, and allowable passive earth pressure coefficient Kp=2.5 can be used for calculation of sliding resistance. For calculating sliding resistance of concrete on the weathered bedrock, an ultimate base friction factor (tanδ) of 0.6 can be used, with an ultimate base adhesion (c) value of 75kPa.

Free-draining granular backfill or synthetic fabric drains should be installed behind all walls. These should connect to weep holes and/or a collector drain, and ultimately to the stormwater system. Granular backfill should be wrapped in a suitable filter fabric to minimise infiltration of silt/clay fines

5.7 Passive Resistance

The horizontal passive resistance provided by socketed sections of piles in soil below excavation floor level can be calculated as:

$\sigma_p = 25z$ (Existing Fill socket only)
$\sigma_p = 60z$ (Residual soil socket only)
$\sigma_p = 100z$ (XW & XW/HW bedrock socket only)

where,

σ_p is the allowable passive pressure acting on the front of the pier/footing at depth z, in kPa

z is the pier socket length below excavation level in soils, in metres

The effective width of a socketed pier for calculation of allowable passive resistance can be assumed to be equivalent to twice its actual width, except where the centre-to-centre distance between the piers is two diameters or less, in which case the soldier piers can be considered to act as one continuous wall.

5.8 Slope Stabilization

To maintain and/or reduce the risk level of slope instability the following options are recommended to be implemented by the relevant stakeholder, which may be the house owner.

It has been indicated that the client intends to stabilise the steep slope by forming ~2m high terraces, supported by low retaining walls (concrete sleepers with cantilevered pots, or similar). Suitable footings for the low retaining wall must be founded on bored piers socketed into weathered shale bedrock (Section 5.2). Such retaining walls can be designed based on the advice in Sections 5.6 and 5.7.

Further recommendations for the slope stabilisation include the following measures;

- It is strongly recommended that footings for all structures (including for the residence and all retaining walls) must be founded in weathered shale bedrock. This may require bored pier footings for the retaining walls.

- Any sections of steep slope not supported by terraced retaining walls must be stabilised using a geofabric such as 'Terramat". Vegetation can then be re-established on the slope to provide long term protection against small scale slumps in the soil and erosion. The geofabric must be installed as per the manufacturer's recommendations, and typically comprises rolling out the fabric and pinning to the slope. The geofabric should extend from the patio edge.

- Maintain adequate drainage of the site and ensure drains are free-flowing. The house downpipes should be checked regularly to ensure they are not blocked or leaking, and stormwater drains must be checked to make sure they are all directed to the street frontage. It is also important that there are no leaking water pipes in the backyard, and that surface water is not allowed to pond on the patio or flow down the slope.

- It is strongly recommended that a deep subsoil drain be installed along the upslope boundary of the site to intercept any subsurface seepages that could lead to slope instability. Given that the development includes a ~3m deep basement on the upslope side of the property, the subsoil drain could be incorporated into the drainage behind the basement retaining walls.

- If it is not practical to install a geofabric on the slope, then vegetation must be re-established on the slope to at least protect against erosion.

- To limit future scour at the toe of the slope due to wave action, it is recommended that some for of wave dissipator be installed along the toe of the cliff. This could include large boulders (preferably strong, durable, volcanic rock), or some form of retaining wall. It is advised that the boulders or retaining wall are founded on bedrock to prevent undermining.

- Periodic monitoring inspections of the slope for signs of erosion and scour should be conducted. Photographs of the slope from one position can be taken as part of the monitoring routine and then compared with each other. This will allow early detection of signs of any erosion progress and/or any movements of the material on the slopes. We recommend that the owner of the property carry out these monitoring inspections on a yearly basis, while a qualified geotechnical engineer should re-inspect every 5 years.

- The following features should be recorded during monitoring inspections:
    - Tension cracks on the top edge of the slope
    - Moisture and seepages on the surface of the slope
    - ‘Lumpiness’ or signs of small scale slumps in the soil
    - Signs of scour or erosion of the slope
    - State of the patio footing with any signs of undermining

5.9 Earthquake Site Factor

Table 2.3 of AS1170.4 “Minimum Design Loads on Structures - Part 4: Earthquake Loads” (Reference 4) lists the earthquake acceleration coefficients for major centres to be considered in structural design. The Batemans Bay area has an acceleration coefficient of 0.08.

Section 4.2 of AS1170.4 “Minimum Design Loads on Structures – Part 4: Earthquake Loads” lists the site sub-soil classes to be considered in structural design. The site is classified as a “Class C_e – Shallow Soil Site”.

5.10 Site Drainage

Suitable surface drainage should be provided to ensure rainfall run-off or other surface water cannot pond against buildings or pavements. Drainage should be provided behind all retaining walls, and subsoil drains should be installed along the upslope sides of access roads and carparks. All drainage must be directed away from the steep slope.

It is strongly recommended that a deep subsoil drain be installed along the upslope boundary of the site to intercept any subsurface seepages that could lead to slope instability. Given that the development includes a ~3m deep basement on the upslope side of the property, the subsoil drain could be incorporated into the drainage behind the basement retaining walls.

ACT Geotechnical Engineers Pty Ltd

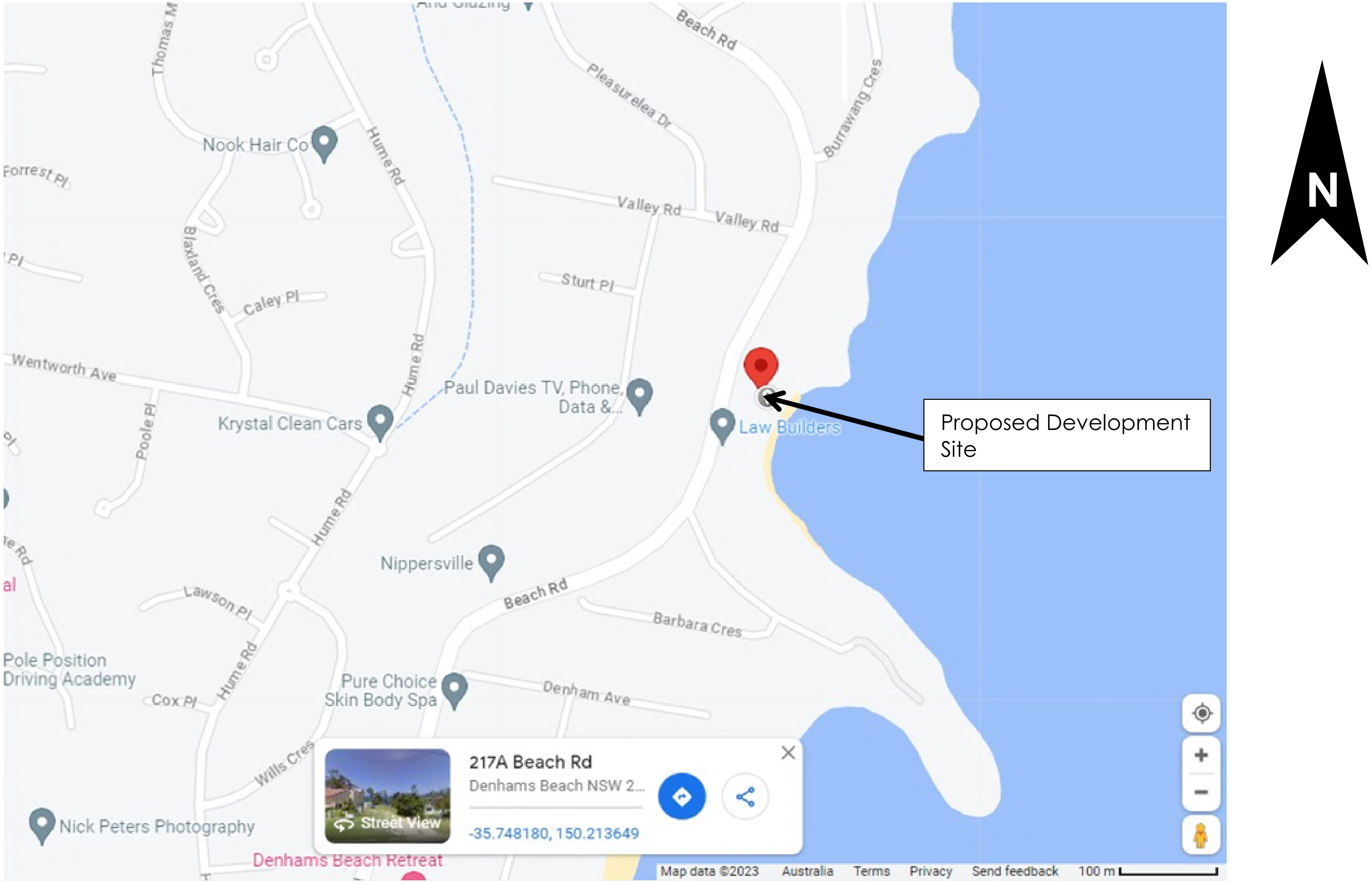
REFERENCES

1 MinView Seamless Geology Map https://minview.geoscience.nsw.gov.au/

2 Standards Australia, “AS2870 – Residential Slabs & Footings”, 2011.

3 AS3798, “Guidelines on earthworks for commercial and residential developments”.

4 Standards Australia, “AS1170.4 – 2007 – Minimum Design Loads on Structures – Part 4 Earthquake Loads”.



Proposed Development Site

ADHAMI PENDER ARCHITECTURE

PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW

SITE LOCALITY

ACT Geotechnical Engineers Pty Ltd	**C14369**	**FIGURE 1**

ADHAMI PENDER ARCHITECTURE
PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW
AERIAL PHOTO AND BOREHOLE LOCATIONS

ACT Geotechnical Engineers Pty Ltd	C14369	FIGURE 2

ADHAMI PENDER ARCHITECTURE
PROPOSED RESIDENCE WITH BASEMENT – 217a BEACH ROAD, DENHAMS BEACH, NSW
SITE PHOTOGRAPHS

ACT Geotechnical Engineers Pty Ltd
C13976
FIGURE 3

NOVARUM BATEMANS PTY LTD
PROPOSED APARTMENT DEVELOPMENT - 29 BEACH ROAD, BATEMANS BAY, NSW
SITE PHOTOGRAPHS – VIEW ON NEIGHBORING PROPERTIES

ACT Geotechnical Engineers Pty Ltd **C13976** **FIGURE 4**

APPENDIX A
Borehole Logs BH1 to BH3

Borehole Log

Borehole No. **BH01**

Sheet 1 of 1

CLIENT: Adhami Pender Architecture

Job No. C14369

PROJECT Proposed Residence with Basement
217a Beach Road, Denhams Beach, NSW

Location :

Collar Level : Not Known
Angle From Vertical : 0°
Bearing : N.A.

Equipment Type : Excavator with Auger Attachment
Hole Diameter :

Samples	Water	Casing	Depth Metres	Graphic Log	U.S.C.S.	Material Description, Structure Soil Type: Plasticity or Particle Characteristics, Colour, Secondary and Minor Components, Moisture, Structure	Consistency or Relative Density	Field Test Results	Geological Profile
	None Encountered		0.15		GS	DGB 20: Sandy GRAVEL; fine to coarse sand, medium angular gravel to 20-25mm, grey			FILL
			1.0 1.2		CL	Silty CLAY; low plasticity fines, pale yellow, pale grey, trace of sand, dry to moist.	Stiff		Residual Soil
			1.9			SHALE; Extremely Weathered, excavated as Silty CLAY, pale grey to white, pale yellow			Bedrock
			2.0 2.1			SHALE; Extremely to highly weathered, fine grained, foliated, highly weathered cobble.			Bedrock
			3.0 3.5			BOREHOLE TERMINATED AT 2.1m Refusal			

Logged By : OB Date : 6/16/23 Checked By : JM Date : 6/21/23

Borehole Log

Borehole No.	**BH02**
Sheet	1 of 1
Job No.	C14369

CLIENT: Adhami Pender Architecture

PROJECT Proposed Residence with Basement
217a Beach Road, Denhams Beach, NSW

Equipment Type : Excavator with Auger Attachment
Hole Diameter :

Location :

Collar Level : Not Known
Angle From Vertical : 0°
Bearing : N.A.

Samples	Water	Casing	Depth Metres	Graphic Log	U.S.C.S.	Material Description, Structure Soil Type: Plasticity or Particle Characteristics, Colour, Secondary and Minor Components, Moisture, Structure	Consistency or Relative Density	Field Test Results	Geological Profile
	None Encountered			▨		cobbles and boulders			FILL
			0.1	▨	SC	Clayey Gravelly SAND; fine to coarse sand, fine to coarse angular gravel, brown, some dark brown, grass roots, dry to moist.	Medium Dense		
			0.4	▨	CL	Gravelly Sandy CLAY; low to medium plasticity clay, fine to coarse sand, fine to coarse angular gravel, brown, dry to moist.	Firm to Stiff		
			0.9 1.0	▨	CL	Gravelly Sandy CLAY; low to medium plasticity clay, fine to coarse sand, fine to coarse angular gravel, brown, highly weathered siltstone, rounded quartz, concrete and ceramics, dry to moist	Firm to Stiff		
			2.0 3.0 3.5			BOREHOLE TERMINATED AT 2m Refusal			

Logged By : OB	Date : 6/16/23	Checked By : JM	Date : 6/21/23

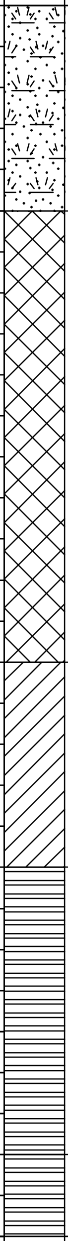
BOREHOLE/EXCAVATION LOG C14369 GINT.GPJ ACT GEO.GDT 6/21/23

ACT Geotechnical Engineers

asda

Borehole Log

Borehole No.	**BH03**
Sheet	1 of 1
CLIENT:	Adhami Pender Architecture
Job No.	C14369
PROJECT	Proposed Residence with Basement 217a Beach Road, Denhams Beach, NSW
Location :	
Collar Level :	Not Known
Angle From Vertical :	0°
Bearing :	N.A.
Equipment Type :	Excavator with Auger Attachment
Hole Diameter :	



Samples	Water	Casing	Depth Metres	Graphic Log	U.S.C.S.	Material Description, Structure Soil Type: Plasticity or Particle Characteristics, Colour, Secondary and Minor Components, Moisture, Structure	Consistency or Relative Density	Field Test Results	Geological Profile
	None Encountered		0.5		SG-SC	Gravelly Clayey SAND; fine to coarse sand, fine to coarse gravel, black, grass roots, dry to moist, concrete blocks	Medium Dense		Topsoil
			1.0 1.6		CL	Gravelly Sandy CLAY; low to medium plasticity clay, fine to coarse sand, fine to coarse angular gravel, highly weathered to moderately weathered siltstone, quartz, coal fragments, brown, grey, dark grey, dry to moist.	Stiff		FILL
			2.0 2.1		CL	Silty CLAY; low to medium plasticity fines, pale brown, trace of fine to coarse sand, dry to moist.	Stiff		Residual Soil
			2.8			SHALE; Extremely weathered, excavated as Sandy CLAY, extremely weak, pale grey, some gravel			Bedrock
			3.0 3			SHALE; Extremely to Highly weathered, fine grained, foliated, highly weathered cobble.			Bedrock
			3.5			BOREHOLE TERMINATED AT 3m Target Depth			

Logged By :	OB	Date :	6/16/23
Checked By :	JM	Date :	6/21/23

ACT Geotechnical Engineers

asda

APPENDIX B
Definitions of Geotechnical Engineering Terms

DESCRIPTION AND CLASSIFICATION OF SOILS

The methods of description and classification of soils used in this report are based on the Australian Standard 1726 – 1993, Geotechnical site investigations. In general, descriptions cover the following properties – soil type, colour, secondary grain size, structure, inclusions, strength or density and geological description.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (e.g. sandy clay) on the following basis:

Classification	**Particle Size**
Clay	Less than 0.002mm
Silt	0.002mm to 0.06mm
Sand	0.06mm to 2.00mm
Gravel	2.00mm to 60.00mm
Cobbles	60mm (63mm) to 200mm
Boulders	>200mm

Soils are also classified according to the Unified Soil Classifications System which is included in this Appendix. Rock types are classified by their geological names.

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

Consistency	**Shear Strength su(kPa) (Representative Undrained Shear)**	
Very soft	< 12	<2 (~SPT "N")
Soft	12 - 25	2-4
Firm	25 - 50	4-8
Stiff	50 – 100	8-15
Very Stiff	100 – 200	15-30
Hard	> 200	>30

Non-cohesive soils are classified on the basis of relative density, generally from the results of in-situ standard penetration tests as below:

Term	**Relative Density (%)**	**SPT Blows/300mm 'N'**
Very loose	< 15	<4
Loose	15-35	4-10
Medium dense	35-65	10-30
Dense	65-85	30-50
Very Dense	>85	>50

SAMPLING

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of 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 generally taken by one of two methods:

1. Driving or pushing a thin walled sample tube into the soil and withdrawing with a sample of soil in a relatively undisturbed state.

2. Core drilling using a retractable inner tube (R.I.T.) core barrel.

Such samples yield information on structure and strength in additions to that obtained from disturbed samples and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

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

PENETRATION TESTING

The relative density of non-cohesive soils is generally assessed by in-situ penetration tests, the most common of which is the standard penetration test. The test procedure is described in Australian Standard 1289 “Testing Soils for Engineering Purposes” Testing Soils for Engineering Purposes” – Test No. F3.1.

The standard penetration test is carried out by driving a 50mm diameter split tube penetrometer of standard dimensions under the impact of a 63 kg hammer having a free fall of 750mm.

The “N” value is determined as the number of blows to achieve 300mm of penetration (generally after disregarding the first 150mm penetration through possibly disturbed material). The results of these tests can be related empirically to the engineering properties of the soil.

The test is also used to provide useful information in cohesive soils under certain conditions, a good quality disturbed sample being recovered with each test. Other forms of in situ testing are used under certain conditions and where this occurs, details are given in the report.

DEFINITIONS OF ROCK, SOIL, AND DEGREES OF CHEMICAL WEATHERING

GENERAL DEFINITIONS – ROCK AND SOIL

ROCK In engineering usage, rock is a natural aggregate of minerals connected by strong and permanent cohesive forces.

Note: Since “strong” and “permanent” are subject to different interpretations, the boundary between rock and soil is necessarily an arbitrary one.

SOIL In engineering usage, soil is a natural aggregate of mineral grains which can be separated by such gentle mechanical means as agitation in water, can be remoulded and can be classified according to the Unified Soil Classification System. Three principal classes of soil recognized are:

Residual soils: soils which have been formed in-situ by the chemical weathering of parent rock. Residual soil may retain evidence of the original rock texture or fabric or, when mature, the original rock texture may be destroyed.

Transported soils: soils which have been moved from their places of origin and deposited elsewhere. The principal agents of erosion, transport and deposition are water, wind and gravity. Two important types of transported soil in engineering geology and materials investigations are:

Colluvium – a soil, often including angular rock fragments and boulders, which has been transported downslope predominantly under the action of gravity assisted by water. The principle forming process is that of soil creep in which the soil moves after it has been weakened by saturation. It may be water borne for short distances.

Alluvium – a soil which has been transported and deposited by running water. The larger particles (sand and gravel size) are water worn.

Lateritic soils: soils which have formed in situ under the effects of tropical weathering include all reddish residual and non residual soils which genetically form a chain of material ranging from decomposed rock through clay to sesqui-oxide rich crusts. The term does not necessarily imply any compositional, textural or morphological definition; all distinctions useful for engineering purposes are based on the differences in geotechnical characteristics.

ROCK WEATHERING DEFINITIONS

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

The degrees of rock weathering may be gradational. Intermediate stages are described by dual symbols with the prominent degree of weathering first (e.g. EW-HW).

The various degrees of weathering do not necessarily define strength parameters as some rocks are weak, even when fresh, to the extent that they can be broken by hand across the fabric, and some rocks may increase in strength during the weathering process.

Fresh drill cores of some rock types, such as basalt and shale may disintegrate after exposure to the atmosphere due to slaking, desiccation, expansion or contraction, stress relief or a combination of any of these factors.

AN ENGINEERING CLASSIFICATION OF SEDIMENTARY ROCKS

This classification system provides a standardised terminology for the engineering description of the sandstone and shales in the Sydney area, but the terms and definitions may be used elsewhere when applicable. Where other rock types are encountered, such as in dykes, standard geological descriptions are used for rock types and the same descriptions as below are used for strength, fracturing and weathering.

Under this system rocks are classified by Rock Type, Strength, Stratification Spacing, Degree of Fracturing and Degree of Weathering. These terms do not cover the full range of engineering properties. Descriptions of rock may also need to refer to other properties (e.g. durability, abrasiveness, etc) where these are relevant.

ROCK TYPE DEFINITIONS

ROCK TYPE	DEFINITION
Conglomerate:	More than 50% of the rock consists of gravel sized (greater than 2mm) fragments.
Sandstone:	More than 50% of the rock consists of sand sized (0.06 to 2mm) grains.
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of silt or clay sized particles and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay sized particles and the rock is laminated.

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

STRATIFICATION SPACING

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

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks.

Term	**Description**
Fragmented:	The core is comprised primarily of fragments of length less than 20mm, and mostly of width less than the core diameter
Highly Fractured:	Core lengths are generally less than 20mm – 40mm with occasional fragments.
Fractured:	Core lengths are mainly 30mm – 100mm with occasional shorter and longer section.
Slightly Fractured:	Core lengths are generally 300mm – 1000mm with occasional longer sections and occasional sections of 100mm – 300mm.
Unbroken:	The core does not contain any fracture.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Society of Rock Mechanics.

Term	**Point Load Index Is(50) MPa**	**Field Guide**	**Approx qu MPa***
Extremely Weak:	0.03	Easily remoulded by hand to a material with soil properties.	0.7
Very Weak:	0.1	May be crumbled in the hand. Sandstone is “sugary” and friable.	2.4
Weak:	0.3	A piece of core 150mm long x 50mm dia. May be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	7
Medium Strong:	1	A piece of core 150mm long x 50mm dia. can be broken by hand with considerable difficulty. Readily scored with knife.	24
Strong: (SW)	3	A piece of core 150mm long x 50mm dia. core cannot be broken by unaided hands, can be slightly scratched or scored with knife.	70
Very Strong (SW)	10	A piece of core 150mm long x 50mm dia. may be broken readily with hand held hammer. Cannot be scratched with pen knife.	240
Extremely Strong (Fr)	>10	A piece of core 150mm long x 50mm dia. is difficult to break with hand held hammer. Rings when struck with a hammer.	>240

The approximate unconfined compressive strength (qu) shown in the table is based on an assumed ration to the point load index of 24:1. This ratio may vary widely.

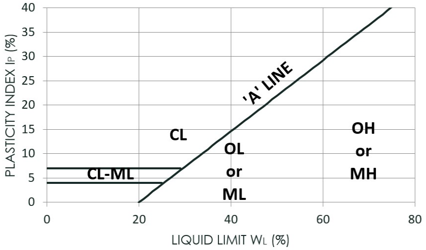
Unified Soil Classification System (Metricated)
Data for Description Indentification and Classification of Soils

MAJOR DIVISIONS				DESCRIPTION			
				Group Symbol	Graphic Symbol	TYPICAL NAME	DESCRIPTIVE DATA
COARSE GRAINED SOILS	More than 50% by dry mass, less than 60mm is greater than 0.06mm.	GRAVELS	More than 50% of coarse grains are greater than 2.0mm	GW	▨	Well graded gravels and gravel-sand mixtures, little or no fines	Give typical name, indicate approximate percentages of sand and gravel, maximum size, angularity, surface condition and hardness of the coarse grains, local or geological name and other perfinent descriptive information, symbols in parenthesis. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. EXAMPLE: Silty Sand, gravelly, about 20% hard, angular gravel particles, 10mm maximum size, rounded and sub angular sand grains coarse to fine, about 15% non-plastic fines with low dry strength, well compacted and moist in place, light brown alluvial sand, (SM)
				GP	▨	Poorly graded gravels and gravel-sand mixtures, little or no fines	
		GRAVELLY SOILS		GM	▨	Silty gravels, gravel-sand-silt mixtures	
				GC	▨	Clayey gravels gravel-sand-clay mixtures	
		SANDS	More than 50% of coarse grains are greater than 2.0mm	SW	▨	Well graded sands and gravelly sands, little or no fines	
				SP	▨	Poorly graded sands and gravelly sands, little or no fines	
		SANDY SOILS		SM	▨	Silty sand, sand-silt mixtures	
				SC	▨	Clayey sands, sand-clay mixtures	
FINE GRAINED SOILS	More than 50% by dry mass, less than 6omm is less than 0.06mm	Liquid Limit less than 50%		ML	▨	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, colour in wet condition, odour if any, local or geological name and r pertinent descriptive information, symbols in parenthesis. For undisturbed soil add information on structure, stratification, consistancy in undisturbed and remoulded states, moisture and drainage conditions. EXAMPLE Clayey Silt, brown, low plasticity, small percentage of fine sand, numerous vertical root-holes, firm and dry in place, fill, (ML).
				CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
				OL	▨	Organic silts and organic silty clays of low plasticity	
		Liquid Limit more than 50%		MH	▨	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.	
				CH	▨	Inorganic clays of high plasticity, fat clays.	
				OH	▨	Organic clays of medium to high plasticity.	
				Pt	▨	Peat muck and other highly organic soils.	

FIELD IDENTIFICATION								
				GRAVELS AND SANDS				Group Symbol
				GRADATIONS		NATURE OF FINES	DRY STRENGTH	
Determine approximate percentages of material over 60mm size, maximum size, shape, surface texture, hardness of material, geological description Identify on estimated percentage mass of the various fractions.	COARSE GRAINED SOILS	More than half of the material less than 60mm is larger than 0.06mm	0.05mm is about the smallest particle visible to the naked eye	GOOD	Wide range in grain size	"Clean" materials (not enough fines to band coarse grains)	None	GW
				POOR	Predominantly one size or range of sizes			GP
				GOOD TO FAIR	"Dirty" materials (Excess of fines)	Fines are non-plastic (1)	None to medium	GM
						Fines are plastic (1)		GC
				GOOD	Wide range in grain size	"Clean" materials (not enough fines to band coarse grains)	None	SW
				POOR	Predominantly one size or range of sizes			SP
				GOOD TO FAIR	"Dirty" materials (Excess of fines)	Fines are non-plastic (1)	None to medium	SM
						Fines are plastic (1)		SC

				SILT AND CLAY FRACTION			
				Fraction smaller than 0 20mm AS sieve size			
				DRY STRENGTH	DILATANCY	TOUGHNESS	
	FINE GRAINED SOILS	More than half of the material less than 50mm is less than 0.06mm		None to low	Quick to slow	None	ML
				Medium to high	None to very slow	Medium	CL
				Low to medium	Slow	Low	OL
				Low to medium	Slow to none	Low to medium	MH
				High to very high	None	High	CH
				Medium to high	None to very slow	Low to medium	OH
Readily identified by colour, odour, spongy feel and generally by fibrous texture							Pt*

LABORATORY CLASSIFICATION						
Group Symbol		% [2] < 0.06mm	PLASTICITY OF FINE FRACTION			NOTES
GW	Use the gradation curve of material passing 60mm for classification of fractions according to criteria given under "Major Division".	0-5	-	>4	Between 1 and 3	1. Identify Fines by the method given for fine grained soils. 2. Borderline classifications occur when the percentage of fines (fraction smaller than 0.06mm size) is greater than 5% and less than 12%. Borderline classifications require the use of dual symbols eg SP-SM GW-GC
GP		0-5	-	Fails to comply with above		
GM		12-50	Below 'A' line and Ip >7	-	-	
GC		12-50	Above 'A' line and Ip > 7	-	-	
SW		0-5	-	>6	between 1 and 3	
SP		0-5	-	Fails to comply with above		
SM		12-50	Below 'A' line or Ip < 4	-	-	
SC		12-50	Above 'A' line and Ip > 7	-	-	
ML		More than 50% passing 0.06mm	Below 'A' line			
CL			Above 'A' line			
OL			Below 'A' line			
MH			Below 'A' line			
CH			Above 'A' line			
OH			Below 'A' line			
Pt*	*Effervescence with H2O2					



PLASTICITY CHART

FOR CLASSIFICATION OF FINE GRAINED SOILS

ACT Geotechnical Engineers

Limitations in the Use and Interpretation of this Geotechnical Report

Our Professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject development and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive borehole and test pit logs, cross- sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory bore holes, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory bore holes and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between conducting this investigation and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and the recommendations considering the changed conditions and time lapse.

The summary bore hole and test pit logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the test holes progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The bore hole and test pit logs and related information depict subsurface conditions only at the specific locations and at the particular time designated on the logs. Soil conditions at the other locations may differ from conditions occurring at these bore hole and test pit locations. Also, the passage of time may result in a change in the soil conditions at these test locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, bore holes or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report: nor can our company be responsible for any construction activity on sites other than the specific site referred to in this report.