DECEMBER, 2018

JOB NO. GE18/144

WOOD AND GRIEVE ENGINEERS PTY LTD

ADDITIONAL GEOTECHNICAL INVESTIGATION

PROPOSED TWEED VALLEY HOSPITAL

LOT 102 ON DP870722

CUDGEN ROAD

KINGSCLIFF

(TSA MANAGEMENT PTY LTD)

MORRISON GEOTECHNIC

ISO 9001 Quality Management FS 610496

NATA WORLD RECOGNISED ACCREDITATION

CONSULT AUSTRALIA

TABLE OF CONTENTS

MORRISON GEOTECHNIC

Brisbane | **Gold Coast** | Maroochydore
Unit 1, 5 Brendan Drive (PO Box 2011), Nerang Q 4211 **P** (07) 5596 1599 **F** (07) 5527 2027
ABN 51 009 878 899
www.morrisongeo.com.au

Job No.: GE18/144
Ref: 18881
Author: Leigh Bexley/Dennis Riley

6th December, 2018

Wood & Grieve Engineers Pty Ltd
Level 6, Building B
207 Pacific Highway
St Leonards NSW 2065

ATTENTION MR IAN HARRIS
Email: ian.harris@wge.com.au
CC: sfolliotte@tsamanagement.com.au

Dear Sir,

RE: ADDITIONAL GEOTECHNICAL INVESTIGATION - PROPOSED TWEED VALLEY HOSPITAL, LOT 102 ON DP870722, CUDGEN ROAD, KINGSCLIFF

1.0 INTRODUCTION

This report presents the results of an additional geotechnical investigation carried out for the proposed new Tweed Valley Hospital to be constructed at Lot 102 On DP870722, Cudgen Road, Kingscliff (the 'site'). The work was commissioned by Mr. Ian Harris of Wood & Grieve Engineers Pty Ltd (the 'Client'). It should be noted that this report is an updated report which includes information from our original preliminary report (dated 26th September 2018) as well as additional information from the current investigation. It should also be noted that the original preliminary report assessed the existing geotechnical conditions of the site and provided preliminary advice for the Stage 1 works based on the original Masterplan Site Plan. Since the original investigation, a new Masterplan Concept Plan has been produced which includes areas which were not assessed in the original report. This additional geotechnical investigation is therefore based on the new Masterplan Concept Plan attached.

1.1. Overview

On 13 June 2017, the NSW Government announced the allocation of $534 million for the development of a new state-of-the art hospital on a greenfield site in the Tweed, to be known as Tweed Valley Hospital (Project). The Project is located on a portion of 771 Cudgen Road, Cudgen, legally described as Lot 102 DP 870722 (Project Site).

The geotechnical investigation was carried out to inform the preparation of an Environmental Impact Statement (EIS). The EIS has been prepared to accompany a State Significant Development Application for the Tweed Valley Hospital which will be assessed under Part 4 Division 4.1 of the Environmental Planning and Assessment Act. The project has been established based on the following supporting documentation:

- Tweed Valley Hospital Business Case.
- Tweed Valley Hospital MasterPlan.
- Tweed Valley Hospital Concept Proposal and design.

The Tweed Valley Hospital Project for which a staged approval is sought consists of:

- Delivery of a new Level 5 major referral hospital to provide the health services required to meet the needs of the growing population of the Tweed-Byron region, in conjunction with the other hospitals and community health centres across the region;
- Master planning for additional health, education, training and research facilities to support these health services, which will be developed with service partners over time. These areas will be used initially for construction site/ compound and at-grade car parking;
- Delivery of the supporting infrastructure required for the new hospital, including green space and other amenities, campus roads and car parking, external road upgrades and connections, utilities connections, and other supporting infrastructure.

The development application pathway for the Project consists of a staged Significant Development Application under section 4.22 of the Environmental Planning and Assessment Act 1979 (EP&A Act) which will consist of:

- A concept development application and detailed proposal for Stage 1 (early and enabling works); and
- A second development application for Stage 2 works which will include detailed design, construction and operation of the Tweed Valley Hospital (Project Application)

A detailed description of the proposed staging of the development is provided in the following sections.

1.1.1. Concept Proposal and Stage 1 Early and Enabling Works

This component (and EIS) seeks approval for a Concept design of the Tweed Valley Hospital and Stage 1 early and enabling works.

The Concept Proposal is informed by service planning to 2031/32 and has an expected gross floor area in the range 55,000m^2 to 65,000m^2. The hospital is expected to include (with more detail to be confirmed/provided at Stage 2) the following components/ services:

* A main entry and retail area
* Administration Services
* Ambulance ServicesRadiation
* Acute and Sub-Acute in-patient units
* Paediatrics
* Intensive Care Unit
* Close Observation Unit
* Mental Health Services
* Maternity Unit
* Renal Dialysis
* Pathology
* Pharmacy
* Cancer Services including Day Oncology and Radiation Oncology
* Emergency Department
* Intergrated Interventional Services
* Interventional Cardiology
* Medical Imaging
* Mortuary
* Back of House Services
* Car Parking
* Future Expansion Areas

Stage 1 includes:

- Early and enabling works (for site clearance and preparation), generally comprising:

  - Construction Compound for Stage 1 works
  - Augmentation and connection of permanent services for the new facility (water, sewer, electricity, telecommunications)
  - General clearance of site vegetation within the footprint of construction works, including tree stumps

- Chipping of cleared vegetation (excluding weed species) to use on site for ground stabilisation/ erosion control, or off-site disposal (as required)
- Bulk earthworks to establish the required site levels and create a stable landform in preparation for hospital construction
- Piling and associated works
- Stormwater and drainage infrastructure for the new facility
- Rehabilitation and revegetation of part of the wetland area
- Construction of internal road ways for use during construction and in preparation for final road formations in Stage 2
- Retaining walls.

1.1.2. Stage 2: Hospital Delivery - Main Works and Operation

Stage 2 (which will be subject to a separate application) would include the detailed design, construction and operation of the Tweed Valley Hospital. Stage 2 will be subject to a separate application following Stage 1.

1.1.3. Subsequent Stages: Potential Future Expansion

Any subsequent stages would be subject to a separate application(s) as required and would be related to works for potential future expansion of the facility. Details of this are unknown at this stage and would be developed as required.

From the information provided in a brief by Bonacci Group (NSW) Pty Ltd, Health Infrastructure NSW have identified a site for the construction of the new Tweed Valley Hospital. The proposed new hospital development is likely to involve the construction of a multi-level building with associated access driveways, car parking areas and bio-retention basins. The delivery of a new Level 5 major referral hospital is to provide the health services required to meet the needs of the growing population of the Tweed-Byron region, in conjunction with the other hospitals and community health centres across the region. Master planning for additional health, education, training and research facilities to support these health services will be developed with service partners over time. These areas will be used initially for the construction site/compound and at-grade car parking. Delivery of the supporting infrastructure required for the new hospital, including green space and other amenities, campus roads and car parking, external road upgrades and connections, utilities connections, and other supporting infrastructure will also be developed.

No information with regards to working loads on foundations have been provided at this stage. A geotechnical investigation is therefore required to assess the subsurface conditions within the footprint of the proposed new Tweed Valley Hospital and associated driveways and car parking areas at the site.

1.2. Report Deliverables

We have been advised that moderate cuts ranging up to approximately 5.0m to 10.0m in depth and moderate fills ranging up to approximately 5.0m in thickness are proposed. Based on the topography of the site, it is anticipated that the cut earthworks would be carried out in the slightly elevated southern portion of the site with the fill earthworks expected in the mild sloping terrain to the north. It is anticipated that material won from excavations carried out on site will be used as controlled fill material in the areas of the site to be filled, where appropriate.

This report contains the results of fieldwork and laboratory testing, together with advice and recommendations relating to:-

- Description of subsurface materials in the depth range of the boreholes in accordance with AS1726-2017 including strength of encountered materials, weathering, defect spacing and defect descriptions of cored rock as well as photographs of the rock core, where encountered. A brief geological history of the site has also been provided;

- Groundwater conditions including levels;
- Preliminary site reactivity in accordance with AS.2870-2011, including predicted ground movement (y_s) and site reactivity at selected borehole locations;
- Earthworks recommendations including excavation conditions (suitable excavation equipment), suitability for the reuse of excavated material as structural fill, compaction standards and site preparation as well as stripping depths.
- Recommendations for filling;
- Trafficability of subsoil material;
- Safe temporary and permanent batter angles for earthworks and temporary retention options including characteristic geotechnical parameters for design;
- Retaining wall design parameters including earth pressure coefficients K_a, K_p and K_o;
- Alternative footing systems, including allowable bearing pressures for high level footings and ultimate end-bearing pressures and shaft adhesion for piles. Estimated settlements for foundation systems would also be provided as well as recommended foundation depths. Allowable peak pressures for live and wind earthquake loads would also be provided;
- Ultimate lateral bearing pressures for piles and high level footings in rock;
- Short term and long term Young's modulus for the pile design;
- Comments with regards to geotechnical strength reduction factors;
- Pavement design parameters and recommendations based on results of soaked CBR testing including:-
    - estimate of the subgrade Young's Modulus for short term and long term loading;
    - subgrade preparation and compaction requirements;
    - type of material and placement specification for pavement materials;
- Earthquake site sub-soil classification in accordance with AS1170.4-2007;
- Site management and construction issues including problems associated with recommended foundation systems;
- Comments with regards to permeability of surficial soils within proposed detention basin area based on results of permeability testing; and
- Slope stability assessment for the proposed development in accordance with the Australian Geomechanics Society (AGS) "Practice Note Guidelines for Landslide Risk Management" Vol 42 No1 March 2007.

In accordance with the NSW Acid Sulfate Soils (ASS) assessment guidelines and the Tweed Local Environmental Plan (TLEP) 2014 clause 7.1, an ASS investigation is only required where works in a Class 5 area that occur below 5 metres AHD or are likely to lower the water table below 1 metre AHD on adjacent Class 1, 2, 3 or 4 land will trigger the requirement for an ASS assessment and may require management. Levels within the development area at the site indicate that the development and associated earthworks will occur well above RL5.0m AHD. A review of the Acid Sulfate Soils Risk map produced by the Department of Land and Water Conservation (DLWC) for NSW indicates that the development site is not located in an area which is assessed to contain Acid Sulfate Soils. On this basis and providing no earthworks are carried out below RL5.0m AHD or works do not lower the water table below 1.0m AHD (which is not expected to occur at the site) an ASS investigation will not be required.

A hydrological assessment of the site is also beyond the scope of this investigation.

2.0 METHODOLOGY

The additional geotechnical investigation involved the drilling of thirty (30) boreholes across the proposed development area of the site. This is in addition to the original twenty five (25) boreholes drilled as part of the original preliminary report. The additional geotechnical investigation boreholes BH29 and BH43 to BH55 were deep boreholes drilled within the footprint of the proposed hospital building or multi storey car park (BH29 and BH44) using a truck mounted Hydrapower Scout drilling rig and extended to depths ranging between 8.3m and 25.1m, below the existing ground surface. The additional geotechnical investigation boreholes BH26 to BH28 and BH30 to BH42 were shallow boreholes drilled within the footprint of the associated access driveways, car parking areas and other light weight single level buildings using a utility mounted drilling rig extending to depths ranging between 1.1m and 4.5m, below the existing ground surface.

The original boreholes BH1 to BH7 and BH25 were deep boreholes drilled within the footprint of the proposed hospital building using a truck mounted Hydropower Scout drilling rig and extended to depths ranging between 6.95m and 21.3m, below the existing ground surface. The original boreholes BH8 to BH21 and BH24 were shallow boreholes drilled within the footprint of the associated access driveways, car parking areas and other light weight single level buildings using a utility mounted drilling rig, extending to depths ranging between 0.8m and 3.0m, below the existing ground surface. Boreholes BH22 and BH23 were also shallow boreholes drilled to depths of 0.5m only for permeability testing within the proposed bioretention basin area.

The deep boreholes were each auger drilled using standard hollow flight 100mm diameter augers fitted with a tungsten carbide (TC) bit extending to depths ranging between 1.0m and 2.5m. Beyond this depth range washbore drilling techniques were carried out to the rock roller refusal depths where NMLC rock coring was then carried out to the borehole termination depths. It should be noted that in the original borehole BH3 no NMLC rock coring could be carried out due to the borehole not being straight due to the presence of boulders. Standard Penetration Tests (SPT) were carried out at regular depth intervals within each of the deep boreholes.

The shallow boreholes were drilled using a utility mounted Jehyco Digga drilling rig using 100mm solid flight augers fitted with a 100mm Tungsten Carbide (T.C) bit. Dynamic Cone Penetrometer (DCP) tests were performed adjacent to most boreholes, where appropriate.

Bulk disturbed samples were collected for Standard Compaction and Soaked Californian Bearing Ratio (CBR) tests to assist with pavement design. Undisturbed U50-tube samples of the natural clay soils were collected for Shrink/Swell Index testing for site reactivity. Pocket penetrometer estimates of q_u (unconfined compressive strength) were also carried out on recovered clay samples. Small disturbed samples were collected for Particle Size Distribution (PSD) and Atterberg Limits tests to assess the suitability of the excavated materials for reuse as structural fill. Point Load Strength Index tests of the recovered rock core was carried out to more accurately assess the strength of the encountered bedrock. These tests were carried out in addition to the tests from the original report.

The subsurface conditions encountered in the boreholes were logged and visually classified in accordance with *AS1726-2017* by a senior engineering geologist and geotechnical engineer from Morrison Geotechnic who directed all field sampling and testing and boxed and photographed the recovered rock core. Co-ordinates and surface levels for each borehole location were provided by B & P Surveys Consulting Surveyors.

Site photographs are presented in Appendix A of this report. The logs of the boreholes including the original logs are presented in Appendix B to this report whilst the results of the laboratory testing including the test results from the original investigation are presented in Appendix C. Cross sections of the boreholes are attached in Appendix D. The guidelines for Hillside Construction are attached in Appendix E. The approximate locations of the boreholes are shown on Site Plan attached to this report. A Masterplan Site Plan showing the proposed development is also attached to this report.

3.0 SITE DESCRIPTION

The site is located on the northern side of Cudgen Road opposite the Kingscliff TAFE College. The proposed development area is approximately 200m wide and 450m long and was recently being used as agricultural farmland.

The topography of the site typically comprises slightly elevated, flat or gentle sloping terrain of less than 5° within most of the proposed development area. This elevated flat and gentle sloping terrain grades into mild sloping terrain to the north which slopes downwards towards the north and north west. This mild sloping terrain then grades into flat, low lying terrain of less than 5° in slope angle forming a floodplain area along the northern boundary of the site. Most of the proposed development is to be located within the slightly elevated, flat or gentle sloping terrain adjacent to Cudgen Road within only a small portion of the proposed development encroaching into the mild sloping terrain to the north and northwest.

No major drainage features are evident at the site. Most of the site forms planar or convex terrain whereby most surface water drains as sheet flows across the surface as well as into small channels associated with drainage for the crops and then over the mild sloping terrain to the north and northwest and offsite into the floodplain area to the north.

The surface levels within the development area at the site range from RL25m to RL27m within the slightly elevated terrain adjacent to Cudgen Road down to RL14m to RL20m within the mild sloping terrain. Downslope to the north of the proposed development area, surface levels reduce to approximately RL5m at the base of the mild sloping terrain near the northern boundary and become less than 5° within the flat, low lying, floodplain area beyond the northern boundary.

The site has been cleared of native vegetation and currently supports agricultural vegetation or fields which were previously used for agriculture. In localised areas aligning the agricultural fields where the fields grade from the slightly elevated, flat or gentle sloping terrain into the mild sloping terrain, several lines of tree and shrubs are evident. The flat, low lying, floodplain area to the north supports a dense covering of native bush vegetation with some exotics. Some large mature exotic trees are also evident surrounding the farmhouse and associated farm sheds adjacent to Cudgen Road, in the central southern portion of the site.

4.0 SUBSURFACE CONDITIONS

The regional geology, local subsurface conditions and groundwater conditions are presented in Sections 4.1, 4.2 and 4.3 respectively.

4.1 Regional Geology

The regional geology of the area forms part of the Lamington Group which is thought to have been formed in the Tertiary Geological Time Period and comprises mainly basalt lava flows with members of rhyolite, trachyte, tuff, agglomerate and conglomerate. The basalt lava flows have weathered to form red clays and extremely weathered rock zones, including boulder zones within a clay matrix formed by spheroidal weathering. This basalt, most probably formed by fissure eruptions creating a series of horizontal flows, has capped the mountain ranges in the area and is most probably underlain at depth by the Neranleigh-Fernvale Group. The Neranleigh Fernvale Group is undifferentiated but thought to have been formed in the Silurian to Devonian Geological Time Period. It comprises greywacke, argillite, quartzite, chert, shale, sandstone and greenstone (Tweed Heads 1:250 000 Geological Survey of NSW, SH56-3).

4.2 Local Subsurface Geology

The subsurface conditions encountered in the boreholes throughout the site typically consists of slopewash soil at the surface comprising moist, stiff to hard, silty clay and sandy clay of medium and high plasticity extending to depths typically ranging between 0.1m and 1.0m. The slopewash soil is underlain by residual soil comprising moist, typically very stiff and hard, silty clay and sandy clay of low to high plasticity extending to depths typically ranging between 0.4m and 3.6m within the slightly elevated terrain to the south adjacent to Cudgen Road but extends to depths up to approximately 16.6m within the areas of the site to the north where the ground surface grades into mild sloping terrain. The residual soil is underlain by extremely weathered (XW), very low and low strength basalt rock which typically becomes slightly weathered (SW) and very high strength from a depth of approximately 1.0m to 2.0m in most boreholes within the slightly elevated terrain extending to depths of approximately 7.0m to 10.0m but with some lower strength layers. Beyond this depth, interbedded layers of XW and distinctly weathered (DW) basalt rock as well as residual clay are typically encountered extending to depths of approximately 14.0m to 15.0m where SW to Fresh (Fr) basalt is encountered extending to the termination depths based on the boreholes that extended beyond a depth of approximately 10.0m.

Several boreholes within the areas of the site to the north (ie BH25, BH29 and BH55 where the slope grades into mild sloping terrain are very inconsistent with interbedded residual clay as well as XW/DW basalt rock with minor SW layers encountered from near the surface, extending to a depth of approximately 15.0m to 20.0m with typically consistent SW and fresh basalt rock encountered below this depth.

To summarise and based on the encountered conditions in the boreholes, it is typically expected that within the slightly elevated, flat or gentle sloping terrain adjacent to Cudgen Road, slightly weathered (SW) basalt rock of very high strength with some interbedded XW and DW basalt layers is likely to be encountered below a depth of approximately 1.0m to 1.5m, extending to a depth of approximately 7.0m to 10.0m. This is typically consistent with residual corestone boulders which have weathered to form a matrix of XW basalt zones between and around the less weathered corestones. Below this depth interbedded XW and DW basalt rock as well as residual clay layers are likely to be encountered, extending to a depth of approximately 14.0m to 15.0m. Below this depth, SW and fresh basalt of very high strength is likely to be encountered, extending to depths in excess of 18.0m to 20.0m. Some boreholes (BH4 and BH44) did encountered SW and Fresh basalt near the surface which extended to the termination depths of approximately 8.0m and 10.0m with no weak or weathered zones.

Towards the mild sloping terrain to the north, similar conditions are expected, however there is likely to be more interbedded layers of XW rock and residual clay throughout with only minor SW basalt rock near the surface and the consistent level of SW basalt rock is expected to be encountered within a depth range of approximately 15.0m to 20.0m below the existing ground surface in this area.

Based on the results of borehole drilling for the original investigation as well as the recent investigation it appears that a consistent level of less weathered SW and Fresh basalt rock which is likely to form the foundation for the hospital is expected to be encountered at depths of around 14.0m to 15.0m adjacent to Cudgen Road and at a depth ranging between approximately 15.0m and 20.0m where the slope grades into the mild sloping terrain to the north.

As a general statement, it is expected that the subsurface conditions between the ground surface and RL9.0m to RL11.0m will be extremely variable ranging between clay soils, XW rock and SW to Fresh rock. Below RL9.0m to RL11.0m the subsurface conditions are likely to be more consistent comprising SW to Fresh basalt rock of at least high and very high strength. Local variations to the geotechnical model will occur on the site and must be expected.

The subsurface conditions encountered in the boreholes are summarised in Table 1 below. Borehole logs are presented in more detail as Appendix B to this report.

Table 1 – Geotechnical Summary of the Subsurface Profile

Borehole No.	Slopewash Clay (m)	Residual Clay (m)	XW/XW-DW Bedrock (m)	DW Bedrock (m)	DW-SW Bedrock (m)	SW/SW-Fr Bedrock (m)	Core Loss
BH1	0.00-0.40	0.40-1.20 10.05-11.60	1.20-1.40 7.10-7.20 7.40-10.05 12.10-14.20	1.40-1.60 2.20-5.00	5.00-6.00 6.40-7.10	1.60-2.20 6.00-6.40 7.20-7.40 14.20-17.40*	11.60-12.10
BH2	0.00-0.40	0.40-2.80	2.80-3.10 8.70-9.00 9.35-9.80*	6.70-6.90 7.25-7.50 7.65-7.90	NE	3.10-6.70 6.90-7.25 7.50-7.65 7.90-8.70 9.00-9.35	NE
BH3	0.00-0.60	0.60-3.60	3.60-4.70 4.90-6.80 7.00-7.30	4.70-4.90 6.80-7.00 7.30-7.80	NE	7.80-7.95*	NE
BH4	0.00-0.50	NE	NE	0.50-0.90 1.90-2.80	NE	0.90-1.90 2.80-10.10*	NE
BH5	0.00-0.70	0.70-1.10	1.10-1.30 3.41-3.57	1.70-1.95 2.30-2.52 4.95-5.25	NE	1.30-1.70 1.95-2.30 2.52-3.41 3.57-4.95 5.25-6.95*	NE
BH6	0.00-1.00	1.00-2.70	2.70-7.20 9.90-14.25 14.33-14.42	7.20-8.00	NE	8.00-9.90 14.25-14.33 14.42-14.50*	NE
BH7	0.00-0.70	0.70-1.50	1.50-1.90 2.70-3.50 4.10-6.70 10.80-15.40	1.90-2.00 6.70-8.90	8.90-10.80	2.00-2.70 3.50-4.10 15.40-19.05*	NE
BH8	0.00-0.10	0.10-1.10	1.10-3.00*	NE	NE	NE	NE
BH9	0.00-0.15	0.15-0.50	0.50-1.30*+	NE	NE	NE	NE
BH10	0.00-0.10	0.10-3.00*	NE	NE	NE	NE	NE
BH11	0.00-0.10	0.10-1.50	1.50-2.60*+	NE	NE	NE	NE
BH12	0.00-0.10	0.10-1.30	1.30-1.50*+	NE	NE	NE	NE
BH13	0.00-0.10	0.10-1.30	1.30-1.50*+	NE	NE	NE	NE
BH14	0.00-0.15	0.15-0.60	0.60-0.90*+	NE	NE	NE	NE
BH15	0.00-0.10	0.10-0.60	0.60-0.80*+	NE	NE	NE	NE
BH16	0.00-0.10	0.10-0.60	0.60-0.80*+	NE	NE	NE	NE
BH17	0.00-0.10	0.10-3.00*	NE	NE	NE	NE	NE
BH18	0.00-0.50	0.50-3.00*	NE	NE	NE	NE	NE
BH19	0.00-0.10	0.10-2.50	2.50-3.00*	NE	NE	NE	NE
BH20	0.00-0.10	0.10-1.30	1.30-2.60*+	NE	NE	NE	NE
BH21	0.00-0.10	0.10-3.00*	NE	NE	NE	NE	NE
BH22	0.00-0.05	0.05-0.50*	NE	NE	NE	NE	NE
BH23	0.00-0.05	0.05-0.50*	NE	NE	NE	NE	NE
BH24	0.00-0.10	0.10-0.40	0.40-0.50*+	NE	NE	NE	NE
BH25	0.00-1.20	NE	1.20-1.45 3.85-4.30 5.10-9.15 9.40-11.15 11.45-11.60 11.90-13.30 13.55-16.70 17.40-17.55 19.80-20.05	1.45-1.55 13.30-13.55	2.18-2.47	1.55-2.18 2.47-3.85 11.15-11.45 11.60-11.90 16.70-17.40 18.15-19.20 19.60-19.80 20.05-21.30*	4.30-5.10 9.15-9.40 17.55-18.15 19.20-19.60
BH26	0.00-0.50	0.50-1.90	1.90-2.10*+	NE	NE	NE	NE
BH27	0.00-0.40	0.40-1.10	1.10-1.50*+	NE	NE	NE	NE
BH28	0.00-0.50	0.50-3.00*	NE	NE	NE	NE	NE
BH29	0.00-0.30	0.30-16.60	NE	16.60-16.80 17.80-18.30 20.34-25.00*	NE	16.80-17.80 18.30-20.34	NE
BH30	0.00-0.40	0.40-3.00*	NE	NE	NE	NE	NE
BH31	0.00-0.50	0.50-3.00*	NE	NE	NE	NE	NE
BH32	0.00-0.40	0.40-3.00*	NE	NE	NE	NE	NE
BH33	0.00-0.30	0.30-3.00*	NE	NE	NE	NE	NE
BH34	0.00-0.40	0.40-3.00*	NE	NE	NE	NE	NE
BH35	0.00-0.40	0.40-3.00*	NE	NE	NE	NE	NE
BH36	0.00-0.40	0.40-3.00*	NE	NE	NE	NE	NE

Borehole No.	Slopewash Clay (m)	Residual Clay (m)	XW/XW-DW Bedrock (m)	DW Bedrock (m)	DW-SW Bedrock (m)	SW/SW-Fr Bedrock (m)	Core Loss
BH37	0.00-0.40	0.40-4.50*	NE	NE	NE	NE	NE
BH38	0.00-0.30	0.30-3.00*	NE	NE	NE	NE	NE
BH39	0.00-0.30	0.30-3.00*	NE	NE	NE	NE	NE
BH40	0.00-0.40	0.40-0.80	0.80-1.10*+	NE	NE	NE	NE
BH41	0.00-0.20	0.20-2.40	2.40-3.00*	NE	NE	NE	NE
BH42	0.00-0.30	0.30-0.80	0.80-1.40*+	NE	NE	NE	NE
BH43	0.00-0.50	0.50-1.20	6.30-6.85 8.90-9.00 9.77-11.20 12.10-14.90	1.20-1.50 4.85-5.00 7.05-7.35 7.80-8.00	5.30-6.00	1.50-4.85 5.00-5.30 6.00-6.30 6.85-7.05 7.35-7.80 8.00-8.25 8.35-8.90 9.00-9.77 14.90-20.40*	8.25-8.35 11.20-12.10
BH44	0.00-0.30	0.30-1.00	NE	1.00-1.15 2.05-2.20	NE	1.15-2.05 2.20-8.30*	NE
BH45	0.00-0.25	0.25-1.30	1.30-1.45 6.25-6.85 8.60-8.90 9.15-9.65 9.85-10.80 10.95-11.65 11.75-11.95 12.30-15.25	15.25-15.50	5.40-5.80	1.45-5.40 5.80-6.25 6.85-8.60 8.90-9.15 9.65-9.85 10.80-10.95 11.65-11.75 11.95-12.30 15.50-20.25*	NE
BH46	0.00-0.50	0.50-1.70 11.00-12.50	9.00-9.75	1.70-2.00 4.20-4.50 5.00-5.30 6.00-6.80 7.10-7.45 7.60-7.84	NE	2.00-4.20 4.50-5.00 5.30-6.00 6.80-7.10 7.45-7.60 7.84-9.00 12.50-17.40*	9.75-11.00
BH47	0.00-0.40	0.40-8.00	9.75-10.85	8.00-8.20	NE	8.20-8.85 9.25-9.75 10.85-16.00*	8.85-9.25
BH48	0.00-0.30	0.30-1.60	1.60-2.10 4.35-4.80 6.45-12.35	2.10-2.55 4.15-4.35 5.50-6.45	3.10-3.95	2.55-3.10 3.95-4.15 4.80-5.50 12.75-17.75*	12.35-12.75
BH49	0.00-0.35	0.35-1.05 7.50-8.30	1.80-2.05 2.70-4.35 4.65-6.00 8.30-8.75 9.20-9.55	1.05-1.20	8.75-9.20	1.20-1.80 2.05-2.70 9.55-14.55*	4.35-4.65 6.00-7.50
BH50	0.00-0.25	0.25-3.00 10.80-14.20	3.00-3.30 3.80-5.00 5.30-10.80 14.20-15.25	3.30-3.80 5.00-5.30	NE	15.25-20.25*	NE
BH51	0.00-0.30	0.30-3.10	3.10-3.50 4.00-4.40 5.40-6.30 6.70-10.60 12.00-12.45 13.60-15.15	4.40-5.40	3.50-4.00 6.30-6.70 10.60-10.80	10.80-12.00 15.15-20.00*	12.45-13.60
BH52	0.00-0.30	0.30-1.40 16.45-18.05	1.40-2.50 4.85-7.45 8.55-8.70 10.70-16.45	3.00-3.80 19.10-19.40	2.50-3.00 3.80-3.95	3.95-4.85 7.45-8.55 8.70-10.70 19.40-23.55*	18.05-19.10
BH53	0.00-0.50	0.50-2.80	2.80-3.20 7.30-7.45 7.60-8.40 8.75-9.55 9.85-10.00 10.15-10.90 11.40-12.10 13.90-15.15 15.50-16.00 17.15-17.45 19.50-19.70	3.20-3.90 4.90-5.40 5.60-6.55 17.45-17.80 19.00-19.50	17.80-19.00 19.70-20.85	3.90-4.90 5.40-5.60 6.55-7.30 7.45-7.60 8.40-8.75 10.00-10.15 13.55-13.90 16.00-17.15 20.85-25.10*	9.55-9.85 10.90-11.40 12.10-13.55 15.15-15.50

Borehole No.	Slopewash Clay (m)	Residual Clay (m)	XW/XW-DW Bedrock (m)	DW Bedrock (m)	DW-SW Bedrock (m)	SW/SW-Fr Bedrock (m)	Core Loss
BH54	0.00-0.30	0.30-5.00 12.12-12.80	5.00-5.20 6.05-8.45 9.30-10.50 11.85-12.12 12.80-13.61	5.70-6.05	5.20-5.30	5.30-5.70 13.61-19.00*	8.45-9.30 10.50-11.85
BH55	0.00-0.30	0.30-7.40 10.30-14.45	7.40-10.30	14.45-14.90	NE	14.90-19.95*	NE
Notes: * - Termination Depth; NE - Not Encountered; + - Borehole Terminated at Maximum T.C Refusal.							

4.3 Geotechnical Model

A typical geotechnical model of the deep boreholes within the footprint of the proposed hospital building is presented in Table 2 below. It should be noted that the presence and depth of Geological Units 3A to 3E vary throughout the site.

Table 2 – Typical Geotechnical Model Based on Deep Boreholes

Geological Unit	Origin	Material	Depth to Base of Layer (m)	Typical Design SPT 'N' Value	Characteristic Mass Modulus $E_m^{(1)}$ (MPa)	Typical Unconfined Compressive Strength (MPa)	Rock Mass Rating (RMR)
Unit 1	Slopewash Soil	Silty CLAY (CI/CH) – Stiff/Very Stiff	0.20-1.00	5-10	15-30	0.15	-
Unit 2	Upper Residual Soil	Silty CLAY (CI/CH) – Very Stiff/Hard	1.00-3.60	5-30	15-90	0.30	-
Unit 3A	Rock	Basalt (XW) – Very Low/Low Strength with interbedded SW Very High Strength layers	Approx 8.00-10.00	>50	1100	1.0	12
Unit 3B	Rock	Basalt (XW) – Very Low/Low Strength with interbedded DW Medium Strength layers	Approx 10.00-15.00	>50	1100	0.8	12
Unit 3C	Rock	Basalt (XW) – Very Low/Low Strength	Approx 10.00-15.00	40-100	1100	0.5	12
Unit 3D	Lower Residual Soil	Silty CLAY (CI) – Stiff/Very Stiff/Hard	Approx 10.00-15.00	5-30	15-90	0.3	-
Unit 3E	Rock	Basalt (DW/SW) – Medium to High/High Strength	Interbedded between 2.50-11.00	N/A	4000	10-50	34
Unit 4	Rock	Basalt (SW/Fr) – Very High Strength	Approx >15.00-20.00	N/A	7500	60-200	45
Note: Conditions over the site will be extremely variable and not all units will be encountered in all areas of the site. (1) Based on $E_m = 10^{(RMR-10)/40} \times 1000$ (MPa) for rock.							

4.4 Groundwater

During borehole drilling, groundwater was encountered in deep boreholes BH1, BH6, BH7, BH25, BH29, BH43 and BH45 to BH55 only at depths ranging between 6.5m and 14.9m below the existing ground surface. No groundwater was encountered in any of the shallow boreholes which extended to depths ranging up to 4.5m below the existing ground surface. On this basis, it is likely that groundwater would only be encountered below a level of approximately RL14.0m within the footprint of the proposed hospital building.

It is expected that consistent conditions comprising SW to Fresh basalt of at least high strength will be encountered below levels of RL9m to RL11.0m. Above this level range, extremely variable conditions will characterize the site ranging between clay soils and weak XW basalt to SW to Fresh basalt of high and very high strength.

Groundwater depths and levels are shown in Table 3 below.

Table 3 – Groundwater Depths and Levels

Borehole	Groundwater Depth (m)	Groundwater Level (RLm)
BH1	11.2	12.0
BH6	12.2	14.8
BH7	11.4	14.0
BH25	14.4	11.4
BH29	6.5	9.7
BH43	13.4	11.3
BH45	13.5	12.9
BH46	10.2	12.3
BH47	4.2	10.2
BH48	10.4	11.8
BH49	7.9	10.9
BH50	13.8	10.7
BH51	11.7	14.5
BH52	15.0	12.4
BH53	12.1	13.9
BH54	11.5	11.2
BH55	14.9	9.1

Seepages may be encountered at the natural soil/weathered rock interface, especially following rainfall events. The presence and depth to groundwater is dependent on rainfall, subsurface material and permeability, integrity of in-ground services, and the proximity to, type and density of vegetation.

5.0 RESULTS OF LABORATORY TESTING

Laboratory testing included Shrink/Swell Index tests for site reactivity, Standard Compaction & Soaked Californian Bearing Ratio (CBR) tests for pavement design and Particle Size Distribution (PSD) and Atterberg Limits tests to assess the reuse of excavated material as structural fill. Point Load Strength Index tests of the recovered rock core was also carried out to more accurately assess the strength of the encountered bedrock.

Undisturbed samples of natural clay soil taken in thin wall 50mm diameter steel tubes in boreholes BH2, BH4, BH12, BH18, BH26 and BH28 were tested to assess volume change capability in the Shrink/Swell Index test (*AS1289 7.1.1*). The results of the Shrink/Swell Index tests are summarised in Table 4.

Table 4 – Shrink/Swell Index Test Results

Borehole No.	Depth (m)	Soil Classification (1)	Swell (%)	Shrinkage (%)	Shrink-Swell Index, Iss (%)
BH2	0.15-0.24	Silty CLAY (CH)	0.0	2.3	1.3
BH4	0.10-0.29	Silty CLAY (CH)	0.0	2.5	1.4
BH12	0.50-0.76	Silty CLAY (CH)	0.0	2.2	1.2
BH18	0.50-0.70	Silty CLAY (CH)	0.0	6.3	3.5
BH26	0.50-0.80	Silty CLAY (CH)	0.0	2.6	1.4
BH28	0.50-0.85	Silty CLAY (CH)	0.0	3.6	2.0
Note: (1) Soil classification based on visual descriptions in the field					

These results indicate that the natural clay soils are slightly and moderately reactive to moisture content changes.

Disturbed, bulk samples were taken from boreholes BH8, BH10, BH17, BH28 and BH40. The samples were representative of the natural clay soils. Standard Compaction and Soaked Californian Bearing Ratio (CBR) tests were carried out on the samples to assess typical pavement design parameters. The results of the Standard Compaction and Soaked CBR tests are summarised in Table 5.

Table 5 – Standard Compaction and Soaked CBR Results

Borehole No.	Depth (m)	Material	Standard Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)	Field Moisture Content (%)	Soaked CBR (%)
BH8	0.10-1.10	Silty CLAY (CH)	1.579	25.5	28.0	12.0
BH10	1.00-1.50	Silty CLAY (CH)	1.358	36.7	38.2	6.0
BH17	0.30-1.00	Sandy CLAY (CH)	1.401	34.8	33.5	4.5
BH28	0.50-0.90	Silty CLAY (CH)	1.270	40.0	32.8	2.5
BH40	0.40-0.80	Silty CLAY (CH)	1.360	35.0	36.1	8.0

The results indicate that the natural clay soils have Soaked CBR values of 2.5%, 4.5%, 6.0%, 8.0% and 12.0%, following compaction to 100% Standard Maximum Dry Density (SMDD).

Disturbed samples were collected from boreholes BH3, BH7, BH10, BH17, BH28 and BH40 for Particle Size Distribution (PSD) and Atterberg Limits tests to assess the reuse of excavated material as structural fill. The results of the PSD and Atterberg Limits tests are presented in Table 6 below.

Table 6 - Quality of Materials Test Results

Borehole No.	Depth (m)	Particle Size Distribution			Liquid Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Material
		Silt/Clay (%)	Sand (%)	Gravel (%)				
BH3	1.50-2.50	50	16	34	47	14	8.5	Silty Gravelly CLAY (CH)
BH7	0.10-0.50	88	10	2	42	15	10.0	Silty CLAY (CH)
BH10	1.00-1.50	87	7	6	46	17	12.0	Silty CLAY (CH)
BH17	0.30-1.00	36	51	13	40	8	5.5	Sandy CLAY (CH)
BH28	0.50-0.90	86	8	6	56	15	7.0	Silty CLAY (CH)
BH40	0.40-0.80	76	7	17	50	16	11.5	Silty CLAY (CH)

Point Load Strength Index testing was carried out on samples of rock core from cored boreholes BH1 to BH2, BH4 to BH7, BH25, BH29, BH43 to BH55. Due to the amount of Point Load Strength Index tests carried out, the results of the tests have not been presented in tabular format but are attached as test certificates in Appendix C to this report.

6.0 PRELIMINARY SOIL REACTIVITY AND GROUND SURFACE MOVEMENTS

This section presents the results of the investigation in terms of the reactivity of the prevailing clay soils.

Shrink/Swell Index testing indicates that the natural clay soils encountered at the site are slightly and moderately reactive to moisture changes. Based on the results of testing and the encountered soil profiles, characteristic ground surface movements at the site may range up to 40mm due to moisture content changes assuming a H_s of 1.5m corresponding to a slightly/moderately reactive ‘Class S/M’ classification in accordance with *AS2870-2011*. These typical ground surface movements are estimated in the context of “**normal**” soil moisture variations, as defined in *AS2870-2011*.

At this site it is likely that **abnormal** soil moisture contents will prevail in the short to medium term due to the proposed earthworks involving moderate cuts and fills. Examples of abnormal soil moisture conditions are described in Section 1.3.3 of AS2870-2011.

The characteristic ground surface movements will change where:

- Further earthworks are carried out;
- Abnormal soil moisture conditions are allowed to develop; and
- Tree planting is carried out on site or trees are removed on or adjacent to the site.

Where the natural clays soils are excavated and placed in fill platforms in excess of 1.5m in thickness, potential characteristic ground surface movements ranging up to 60mm can be expected on this site due to "normal" variations in foundation soil moisture conditions which corresponds to a highly reactive 'Class H1' in accordance with *AS2870-2011*.

The design of new building footings should take account of the characteristic ground surface movement, y_s, and the potential surface movement resulting from tree induced suction changes, y_t, calculated as described in Appendix H of AS2870-2011.

Potential ground surface movements due to the effects of trees (y_t) can be assessed using the parameters in Table 7.

Table 7 – Ground Surface Movements Due to Effects of Trees

Design Suction Change Depth H_s (m)	Shrink/Swell Index Iss (%)	Single Tree		Tree Group	
		Maximum Extra Suction Change (pF)	Maximum Design Drying Depth (H_t)	Maximum Extra Suction Change (pF)	Maximum Design Drying Depth (H_t)
1.5	3.5	0.3	2.5	0.38	3.0

The design of the footings and floors of all residential type buildings must be carried out in accordance with AS2870-2011, using the parameters presented above.

7.0 BULK EXCAVATION CONDITIONS

The information and advice presented in this section of the report is based on the conditions encountered in the boreholes drilled within the development area only. Extreme variations in these conditions will occur across the site and must be expected.

Bulk excavations ranging up to approximately 5.0m to 10.0m in depth are expected in the slightly elevated central southern portion of the site as well as the area to the north where the slope grades into the mild sloping terrain.

Ripping depths can be significantly increased where the rock is bedded, laminated and highly jointed. The nature of the rock and inherent planes of weakness therefore play an important part in rock excavation assessment. Most of the rock comprised basalt which contained significant defects including jointing and extremely weathered (XW) basalt layers which are expected to improve excavation rates. There was, however several areas where SW and SW-Fr basalt bedrock was encountered and the defects were not as predominant. Excavation rates in this rock are expected to be slower.

Bulk excavations up to approximately 5.0m to 10.0m will require the removal of natural clay soil as well as extremely weathered (XW), distinctly weathered (DW), slightly weathered (SW) and slightly weathered (SW) to Fresh (Fr) basalt rock. It should be noted that the geotechnical investigations comprised the drilling of twenty two deep boreholes (BH1 to BH7, BH25, BH29 and BH43 to BH55) within the cut areas of the site with most of these boreholes extending below the proposed excavation depths.

Excavations extending to a depth of approximately 1.0m to 2.0m below the existing ground surface should be achievable using conventional earthmoving equipment such as small to medium sized dozers and excavators. In localised areas (BH3, BH6, BH29, BH47, BH50, BH51, BH53, BH55) where mainly natural clay soils and XW basalt rock extend to a depth in excess of approximately 7.0m to 10.0m below the existing ground surface, excavations to this depth should also be achievable using conventional earthmoving equipment such as small to medium sized dozers (D6) or medium sized excavators (say ~15t to 20t).

Several boreholes (BH7, BH25 and BH54) also contain mainly natural soil and XW basalt rock with some localised, interbedded zones of SW basalt extending to depths of at least approximately 8.0m to 10.0m. In these areas excavations to depths of approximately 8.0m to 10.0m should be achievable using earthmoving equipment such as medium sized dozers (D6-D7) and medium to large sized excavators (20t) fitted with toothed buckets and ripper attachments.

Typically excavations below a depth of approximately 1.0m to 2.0m and extending to depths ranging between approximately 5.0m and 10.0m will most probably encounter slightly weathered (SW) to fresh (Fr) basalt rock (residual corestone boulders) with numerous defects and some interbedded XW weathered basalt zones. It is expected that heavy earthmoving equipment such as a Cat D9 dozer or larger in bulk excavation or large excavators exceeding 20t fitted with ripper attachments for confined excavations will be required to achieve the proposed excavation depths. The use of hydraulic rock breakers is likely to be required for confined excavations such as service trenches but also possibly for localised areas of bulk excavations, particularly at the location of boreholes BH2, BH4, BH43 to BH46 and possibly BH1 and BH5. Some budgeting should be allocated for the possibility of encountering localised zones of SW and Fr basalt rock with minimal defects which may require localised blasting or heavy ripping using a Cat D10/D11.

No significant groundwater issues are expected to be encountered in excavations under normal weather conditions in the slightly elevated cut areas of the site as excavations are typically not expected to extend below the expected water table.

8.0 SITE PREPARATION AND STRIPPING DEPTHS

Prior to the placement of fill there are several issues and requirements which must be addressed regarding the surficial soils at the site. These issues include crops, trees, grass, cobbles, boulders and topsoil at the surface as well as in ground elements associated with the existing structures on site.

This will most likely require the removal of the upper 0.1m to 0.35m of topsoil and disturbed slopewash soils which contain organics from crops across the site, but possibly up to about 1.0m where structures (houses/sheds) and large trees are located as well as in areas where cultivation has been carried out to form small drainage bunds. Most boreholes indicate the presence of stable natural clays which are stiff or better in consistency below depths of 0.15m to 0.35m. The following general treatment action is recommended within the development area prior to the placement of structural fill on the site:-

- Remove all trees, grass, topsoil, organics, uncontrolled fill and surface irregularities such as eroded and uneven areas and the cultivated areas from the existing ground surface which is expected to typically extend to depths ranging up to approximately 0.1m to 0.15m but possibly up to approximately 0.35m or more where existing structures are present.
- Depressions formed by the removal of vegetation, existing structures or other features should have all disturbed soil removed to expose competent natural soil and be backfilled with compacted and suitably moisture conditioned select fill material.

9.0 SUITABILITY OF EXCAVATED MATERIAL FOR REUSE AS STRUCTURAL FILL

Bulk excavations to a depth of approximately 5.0m to 10.0m below the existing ground surface are expected as part of the bulk earthworks on site. Excavations to this depth range are expected to encounter minor topsoil, slopewash clay of medium and high plasticity, residual clay of low to high plasticity and XW, DW, SW and SW-Fr basalt rock. Confined excavations for services may also encounter less weathered basalt rock in the deeper sections of trenches in the slightly elevated central portion of the site.

The following presents recommendations for the reuse of excavated material as controlled fill:

- The existing topsoils and disturbed soils for farming, which generally contain organic matter and tree roots, are not suitable for use as structural fill without selective sorting to remove all organics and deleterious material. This material could be stockpiled and used for landscaping or gardens.
- The in-situ soils and XW and DW basalt rock obtained from site cuttings, where free of organic and deleterious material, may be used for structural fill provided the moisture content of the soils on placement approximates the OMC (Optimum Moisture Content) required for compaction. This may require moisture conditioning to bring the soils to OMC.
- With the use of in-situ or imported reactive clay soils, proper control of moisture content during placement and compaction is required so as to minimise the potential for swelling and shrinkage movement. Moisture content within the range of OMC to OMC +2% is recommended. The natural clay based soils may present difficulty in achieving the required compaction density. Foundation design must reflect the use of the potentially reactive clays if they are used as structural fill. We do not recommend the use of these reactive clay soils as fill within 1.5m of the final surface level.
- The weathered basalt rock, where broken down on extraction, may be used as structural fill provided no rock over 75mm in greatest dimension is included above a level, 500mm below the design subgrade/platform level. Below this plane, rock fragments up to 100mm greatest dimension may be used. These rocks should not represent more than 15% of the fill make-up. Rocks over 100mm greatest dimension, which cannot be broken down, should be removed or stockpiled for use as landscaping or for the fill in non - structural areas. It is difficult to satisfactorily compact rocks greater than 100mm size.
- All boulders should be removed or stockpiled for use as landscaping, retaining walls or for the fill in non - structural areas. Boulders up to 300mm to 400mm in size may possibly be used at the base of the deep fills.
- Consideration should be given to the exclusion of rock pieces with greatest dimensions of 50mm to 75mm or greater from areas where trenching and piling may occur, as large rock pieces could impede the trenching and piling operations. It is expected that the XW and DW basalt rock could generally be expected to be broken down to maximum aggregate size of 50mm or less, during extraction and compaction.
- The SW and SW-Fr basalt rock which was inconsistently encountered in most boreholes in the upper 7.0m to 10.0m depth profile is expected to present difficulties in being broken down to a maximum aggregate size of 100mm or less, during extraction and compaction. This less weathered basalt rock may therefore need to be removed from site or possibly a crushing plant could be set up to break the rock down to a maximum aggregate size of 100mm or less. This less weathered basalt rock could then be used as pavement gravel or possibly blended with the excavated natural soils to form general fill material.
- Provided the placement moisture content of any imported fill or select in-situ material approximates the Optimum Moisture Content for compaction, suitable compaction should be achievable using typical compaction machinery. This may require moisture conditioning to bring the soils to OMC. The fill materials should be compacted in layers not exceeding 200mm loose thickness. However, layer thicknesses will be dependent on the compaction plant type and size, use of vibration, material type and condition. Final maximum placement layer thicknesses should be assessed when compaction plant, as well as material type and conditions, are known.
- Imported select fill material, if required, should be a good quality select fill material with a soaked CBR of at least 10%, a maximum aggregate size of 50mm and have a maximum Shrink/Swell Index of 1.0%.

- With the use of in-situ or imported reactive clay soils, proper control of moisture content during placement and compaction is required so as to minimise the potential for swelling and shrinkage movement. Moisture content within the range of OMC to OMC +2% is recommended. Foundation design must reflect the use of the potentially reactive clays if they are used as structural fill.
- The clay based soils may present difficulty in achieving the required compaction density. These soils typically require a moisture content very close to OMC to achieve compaction. Close control of the moisture content of these soils would therefore be required if used as structural fill.

The rock materials encountered within the cut areas may present difficulties when being reused as structural fill. Over sized rock material must be crushed to the appropriate size (preferably less than 50mm but up to 100mm) and moisture conditioned to within ± 2% of optimum moisture content (OMC) with the following procedure recommended:

- The excavated material is to be spread in thin layers not exceeding 300mm.
- The oversized material is to be broken down and compacted using a machine of sufficient size.
- The excavated material should be uniformly compacted to 98% Standard Maximum Dry Density (SMDD) in accordance with AS3798-2007.
- Trial passes are recommended to visually assess the efficiency of the rock breaking procedure.
- A minimum of 12 passes using the rock breaking machine are recommended.

The possibility of encountering stronger zones of rock which may not break down using the successful trial procedure must not be discounted. Consideration must be given to sourcing larger plant (or crushing plant) to break down the rock. Alternatively, the rock may be unsuitable for use as structural fill. If the SW and SW-Fr basalt can be broken down using a larger plant or a crushing plant, this material may possibly be used as pavement gravel. Compaction and Soaked CBR testing would be required to be carried out on this crushed material to assess the design soaked CBR values.

10.0 RECOMMENDATIONS FOR FILLING

Based on the topography of the site, bulk fill earthworks are expected in the lower northern portion of the site as well as within pavement and car parking areas. Broad recommendations for filling are as follows:-

- In areas which are to be filled, all trees, grass, weed zones, uncontrolled fill, wet and weak soils, organic matter and debris must be removed from the existing ground surface to expose competent natural soil or weathered rock.
- Depressions formed by the removal of vegetation or other features should have all disturbed soil removed to expose competent natural soil and be backfilled with compacted and suitably moisture conditioned select fill material.
- The fill used in the construction of any proposed batters and behind retaining walls is placed and compacted to at least 98% Standard Maximum Dry Density and is “Controlled Fill” in accordance with A.S. 2870 (Clause 6.4.2 (a)) – “Residential Slabs and Footings” and A.S. 3798.
- Where the ground surface slopes at 8^{o} or more, the foundation is benched prior to filling and the fill supported by engineered retaining walls or battered to the appropriate angles described in Section 11.0.
- Fill batters should be over-constructed and trimmed back to ensure compaction in the outer zones.

- Vertical heights of natural soil and fill batters should incorporate a 1.0m wide bench at every 2.5m height interval when battered to the maximum allowable slope angle and a 2.5m wide bench incorporated at every 5.0m height interval.
- Earthworks should take account of the sloping terrain and be limited to cuts and fills which can be adequately constructed by medium sized civil contractors without resorting to specialised contractors.

11.0 TEMPORARY AND PERMANENT BATTERS

Maximum safe batter angles for the different materials encountered on site are shown below in Table 8 for unsurcharged batters less than 3.0m high. Where surcharges are located within H(m) (height of the batter) of the top of the batter, then some reduction in the design angle will be required.

Table 8 – Safe Batter Angles for Cuts

Material	Short Term	Long Term
New Controlled Fill Material (1)	45^0 (1V:1H)	26^0 (1V:2H)
Medium Dense or Better Natural Sand Soils	45^0 (1V:1H)*	26^0 (1V:2H)*
Stiff or Better Natural Clay Soils	45^0 (1V:1H)*	26^0 (1V:2H)*
XW, DW, SW & SW-Fr Basalt	60^0 (1.5V:1H)*	45^0 (1V:1H)*

Note:
*Subject to inspection by experienced geotechnical engineer/engineering geologist.
(1) In accordance with AS3798-2007

Structural fill batters should be overconstructed and trimmed back at no steeper than 2(H):1(V) or 26^0.

All exposed permanent batters should be properly vegetated and mulched to minimise erosion. Properly maintained vegetation should reduce the occurrence of surface erosion by impingent rainfall. It is essential that permanent cut and fill batters and embankments are suitably protected from erosion and scour by appropriate drainage. Upslope runoff should also be directed away from the batters limiting the ingress of water into the fill. The surface water should not be allowed to discharge directly across the batters. Surface drains must be constructed and located behind the crests of all batters and benches. The pavement drainage system may be sufficient for some batters.

The option of battered excavations is feasible but dependent on the excavation location in relation to the existing roads, boundary lines and other infrastructure. For preliminary assessments, the batter angles given above can be adopted.

Unsupported vertical excavations should be assessed by a suitably qualified engineer or geologist prior to personnel entering the excavation.

Where the above angles cannot be achieved, temporary or permanent retention systems may need to be incorporated to achieve desirable earthworks outcomes.

Where spacial constraints prevent the use of battered or benched excavations near existing roads and underground infrastructure, the use of temporary retention systems is likely to be required for excavations exceeding approximately 1.0m to 2.0m. The temporary lateral support should be designed prior to the commencement of excavation. Temporary structural retention systems that may be considered include:-

- A shoring box with end plates (shallow excavations);
- Suitable propping (shallow excavations);
- Anchored contiguous bored piles (deep excavations); or
- A temporary shotcrete and nail system (shallow and deep excavations).

Depending on the construction methodology and the dimensions of the excavations, the use of sheet piles is not considered appropriate due to the potential presence of cobbles and boulders within the soil as well as shallow, less weathered rock at numerous locations.

The temporary retention systems should be designed by a suitably qualified and competent engineer using the parameters presented in Table 9, Section 13.0 of this report.

12.0 SITE TRAFFICABILITY

At the time of the field investigation, trafficability was considered to be fair, however site trafficability may be difficult during and after wet weather periods particularly following stripping of surface material to expose natural high plasticity clay soils.

The most likely trafficability problems which are expected to arise for earthworks and construction machinery are from:-

- Softening of the upper level clay soils or clay soils exposed after excavations during and after periods of rainfall particularly in the slightly elevated, flat portion of the site adjacent to Cudgen Road as well as the flat, low lying terrain forming the floodplain area to the north. This is expected to occur in the upper clay based soils and may require the installation of seepage cut-off drains or rock-fill working platforms should construction commence during or following an extended period of wet weather. or residual clay soils following excavation
- Disturbance of the upper level soil fabric with removal of vegetation and underground elements. Depressions could be formed resulting in potential water traps, which could cause further softening of exposed soils. The agricultural fields where lineal shallow trenches have been formed may also result in potential water traps which could also cause further softening of the exposed soils if not addressed.

To improve site trafficability, it is recommended that following stripping, clearing and grubbing, the exposed surface be proof rolled to encourage surface drainage and prevent the formation of boggy conditions. The ground surface should then be inspected and assessed. Areas which demonstrate excessive movement and/or do not improve sufficiently under proof rolling should be removed and replaced as required. The exposed platform surfaces should be protected from drying after excavation is complete. If left unprotected, the soils are susceptible to drying and cracking. Temporary drainage measures should also be implemented to direct surface water away from high traffic areas and to reduce the potential for pooling of surface water. Maintaining adequate drainage conditions is also essential. It should be ensured that runoff is diverted away from the construction area to prevent the ponding of water.

Potential trafficability problems with this site should not be underestimated. The site will very quickly become untrafficable if appropriate drainage control measures, together with construction practices appropriate for the site conditions, are not maintained, particularly in the flatter areas of the site. If weather conditions and time constraints do not allow for seepage drains to take effect to sufficiently improve the site to allow earthworks to proceed, then site trafficability may be improved by constructing haulage roads into and on the site. Consideration can also be given to the placement of rock along high traffic areas. Rock fill materials won from onsite excavations or imported rock fill materials could be used as a blanket course for the haulage roads.

Preferably, earthworks should be scheduled in dry weather following a period of not less than several weeks of little or no rainfall. The contractors should fully inform themselves of the ground conditions on site prior to commencement of earthworks. This requirement should be explicit in any earthworks specifications or contract.

13.0 RETAINING WALL DESIGN PARAMETERS

It is anticipated that potential retaining structures are likely to retain new structural fill, natural clay soils or XW, DW, SW or SW-Fr basalt rock or a combination of these materials.

Where lateral movements are tolerable and retaining walls are free to rotate, retaining walls should be designed for 'active' (K_a) lateral earth pressure conditions. Where movements are to be limited and the retaining walls are restrained at the heads, design for the 'at rest' (K_0) condition is required.

Where adjacent structures and/or underground services are present, design using 'at rest' lateral earth pressures is required. The design of permanent retaining walls with horizontal backfill can be based on the recommended design lateral earth pressure coefficients presented in Table 9.

Table 9 – Design Parameters for Permanent Retaining Walls

Typical Geological Unit	Material	Effective Strength Parameters		Bulk Density (kN/m^3)	Buoyant Density (kN/m^3)	Earth Pressure Coefficient		
		c' (kPa)	Ø (degrees)			Active K_a	At Rest K_o	Passive K_p
N/A	Controlled Clay Fill Material (1)	2	25	20	10	0.41	0.58	2.47
N/A	Controlled Gravel Fill Material (2)	-	36	21	11	0.26	0.41	3.85
Unit 1/2/3D	Stiff Natural Clay	4	25	20	10	0.41	0.58	2.47
Unit 1/2/3D	Very Stiff Natural Clay	5	26	21	11	0.39	0.56	2.56
Unit 2/3D	Hard Natural Clay	6	26	21	11	0.39	0.56	2.56
Unit 3C	XW Basalt (VLS/LS)	10	33	23	13	0.30	0.46	3.40
Unit 3B	XW-DW/DW Basalt (LS/LS-MS)	30	35	24	14	0.27	0.43	3.69
Unit 3E	DW-SW Basalt (4) (MS/HS)	100	38	25	15	0.24	0.38	4.21
Unit 4	SW/SW-Fr Basalt (4) (VHS)	200	45	25	15	0.17	0.29	5.83

Notes:
(1) Based on existing natural clay soil onsite being reused as structural fill material.
(2) Assuming gravel fill material won from excavations of basalt rock on site or Type 2.3 gravel will be re-used as Level 1 backfill.
(3) VLS-Very Low Strength; LS-Low Strength; MS-Medium Strength; HS-High Strength; VHS-Very High Strength.
(4) Assume minimal jointing or favourable orientated joints.

The generalized lateral earth pressure distribution is given as:

$$p = K\gamma H + Kq + \sigma L \text{ (kPa)}$$

- K is either K_a, K_o, or K_p for "active", "at rest" or "passive" earth pressure conditions, respectively;
- γ (kN/m^3) is the relevant density of the soil and weathered rock;
- H (m) is the distance down from the top of the wall;
- q (kPa) is any uniform surface surcharge load behind the wall; and
- σL (kPa) is the lateral pressure on the wall resulting from adjacent surcharges.

For a line load of Q (kN/m) acting within 2.0m of the retention system, σL (kPa) can be assessed as a trapezoidal pressure distribution, increasing linearly from zero at the ground surface to Q/H(kPa) at a depth of 0.2H(m) and then decreasing linearly to 0.15Q/H (kPa) at the toe of the wall, where H(m) is the depth of the excavation.

Below the design water table, the bulk density is replaced by the buoyant density described for each material in Table 9 above, and a hydrostatic pressure distribution given by $10h_w$ (kPa) added to the lateral earth pressure, where h_w (m) is the height of the design water table above the base of the retaining wall. The walls should be properly drained to prevent the buildup of groundwater pressure behind the walls.

As an alternative to using excavated material as backfill, imported material such as select granular fill or cement stabilized sand could also be used as a backfill material. The design parameters of the imported material are dependent on the properties of the material. The product supplier should be contacted for further information if this option is selected.

The design parameters for retaining walls presented in this section must be confirmed by on-site inspection when the subsurface conditions are exposed during construction. A factor of safety of 2.0 should be achieved for permanent support systems and drainage must be provided behind all retaining walls.

14.0 FOUNDATION DESIGN PARAMETERS

No information regarding the anticipated loads of the proposed hospital structures has been provided. Allowable bearing pressures for shallow footings and ultimate end bearing pressures for deep footings are given below in Sections 14.1 and 14.2 respectively.

14.1 High Level Footings

Depending on the design loads of the proposed new hospital buildings and associated light weight structures, high level stiffened rafts, pad and strip footings founding in new controlled fill material, stiff or better natural clay soils or weathered basalt rock may be adopted for the proposed structures.

If footings cannot be poured on the same day as the excavations, a concrete blinding layer of at least 50mm thickness is recommended.

The design of high level footings including pads, strips and the ground beams of stiffened rafts founding at a minimum depth of 0.5m below the finished ground surface can be based on the allowable bearing pressures presented in Table 10.

Table 10 – Allowable Design Parameters for High Level Footings

Geological Unit	**Material (2)**		**Allowable Bearing Pressure (kPa) (4)**	
			Strip Footings	**Pad Footings**
N/A	Controlled Fill (3)		100	100
Unit 1, 2, 3D	Natural Clay Soils	Stiff	100	125
		Very Stiff	150	175
		Hard	175	200
Unit 3A, 3B, 3C	XW Basalt Rock	VLS (5)	350	400
		LS (5)	400	500
Unit 3E, 4	DW/SW/SW-Fr Basalt Rock	MS/HS/VHS (5)	1000	1250

Notes:
(1) NR – Not recommended.
(2) All founding material should be verified by a suitably qualified geotechnical engineer or engineering geologist during construction.
(3) In accordance with AS3798-2007
(4) Based on a factor of safety of 3.0, and a strip footing width of 1.0m and a pad footing width of 2.0m.
(5) VLS-Very Low Strength; LS-Low Strength; MS-Medium Strength; HS-High Strength; VHS-Very High Strength

For short term loading in high level footings, an allowable peak edge pressure for live and wind or earthquake loads of 1.2 times the allowable bearing pressure is suggested.

Raft design stiffness parameters are presented in Table 11.

Table 11 – Design Parameters for Rafts

Geological Unit	Material		Modulus of Subgrade Reaction (kPa/mm)	Long Term Young's Modulus (MPa)	Long Term Poisson's Ratio
N/A	Controlled Fill (1)		25	15	0.40
Unit 1, 2, 3D	Natural Clays	Stiff/Very Stiff	25	20	0.40
		Hard	35	25	0.40
Unit 3A, 3B, 3C	XW Basalt Rock	VLS/LS (2)	100	500	0.30
Unit 3E, 4	DW/SW/SW-Fr Basalt Rock	MS/HS/VHS (2)	200	2000	0.25

Notes:
(1) In accordance with AS3798-2007 reusing the excavated clay material from onsite.
(2) VLS-Very Low Strength; LS-Low Strength; MS-Medium Strength; HS-High Strength; VHS-Very High Strength

The parameters shown in Table 11 above can be adopted for detailed raft analysis using finite element methods. Total contact pressures beneath the raft should be limited to avoid excessive differential settlements.

It should be noted that due to the variability within the strength of the encountered basalt rock within the upper profile of the boreholes, the DW/SW/SW-Fr Basalt Rock of medium, high and very high strength has been grouped together for the shallow foundation parameters to provide more conservative values as a precaution. It should also be noted that when adopting the parameters for the encountered materials shown in the Tables 10 and 11 above, there must be at least 2B (where B is width of footing) of the design foundation material present below the design foundation level for the footings.

Considering the materials encountered, preliminary settlement assessments indicate that the total settlements could typically range between 10mm and 60mm, depending on the fill heights and structural ground loads. Differential settlements of up to 50mm could therefore be expected. These settlement estimates are approximate. Finite element modeling using commercially available software would be required to refine these settlement estimates.

It is expected that some structures may found on a combination of fill, natural soils and possibly weathered basalt rock. It is recommended that a uniform foundation is adopted across the entire footprint of the proposed buildings to reduce differential settlements and reduce the potential for damage to the structures. Where the footings for the proposed structures found in different materials, consideration should be given to designing the structure to accommodate the possible differential settlements.

14.1.1 Ultimate Lateral Resistance of High Level Footings

The ultimate long term lateral resistance of strip and pad footings founding in the natural clay soils and weathered rock can be based on the parameters presented in Table 12 and the equation below:-

$$\sigma L \text{ (kPa)} = K_p \gamma H + q\tan 0.7\text{Ø} \text{ (kPa)}$$

- K_p is the passive earth pressure coefficient;
- γ (kN/m^3) is the bulk density of the soil or weathered rock;
- H (m) is the distance down from the top of ground surface;
- q (kPa) is the pressure on the base of the footing;
- Ø friction angle of the soil or rock; and
- σL (kPa) is the ultimate lateral resistance.

Table 12 – Ultimate Lateral Resistance Parameters for High Level Strips and Pads

Material	γ (kN/m^3)	K_p	Base Friction Factor as Tan 0.7Ø
Stiff Clay	20	2.47	0.32
Very Stiff Clay	21	2.56	0.33
Hard Clay	21	2.56	0.33
XW Basalt	23	3.40	0.43
DW/SW/Fr Basalt	25	4.21	0.50

Note: γ based on the bulk density above the water table

14.2 Deep Footings

If loads for the hospital or associated buildings are considered to be excessive for high level footings or the potential differential settlements cannot be tolerated or accurately estimated, due to the varying foundation material, a deep footing system may be considered. Considering the materials encountered, bored piles or CFA grout injected piles founding in the less weathered Unit 4 basalt rock below RL9.0m to 11.0m are considered to be a suitable system for deep footings at this site. For lighter structures associated with the hospital, bored piles or CFA grout injected piles founding in the natural clay soils or Unit 3A, 3B or 3C XW basalt rock may also be considered to be a suitable system for deep footings at this site.

It is recommended that all piles be socketed at least four pile diameters into the design founding material. Axially loaded bored piles and CFA grout injected piles can be designed using the ultimate geotechnical pressures presented in Table 13 below.

Table 13 – Ultimate Design Parameters for Bored or Grout Injected Piles

Geological Unit	Material		Ultimate Geotechnical Shaft Adhesion (kPa)	Ultimate Geotechnical End Bearing Pressure (kPa) (2,3)	
				L $\leq$ 4D (1)	L $\geq$ 4D (1)
Unit 2, 3D	Natural Clay Soils	Stiff	35	300(4)	450(4)
		Very Stiff	50	600(4)	900(4)
		Hard	70	1200(4)	1800(4)
Unit 3A, 3B, 3C	XW Bedrock	VLS (5)	100	1800	2400
		LS (5)	150	2400	3600
Unit 3E	DW/DW-SW Basalt	MS/HS (5)	500	3500	8000
Unit 4	SW/SW-Fr Basalt	VHS (5)	1000	6000	12000

Notes:
(1) L – Pier socket length; D – Pile diameter;
(2) Values apply above the groundwater table.
(3) Based on a minimum pile depth of 3.0m.
(4) Upward ultimate geotechnical shaft adhesion resistance and ultimate end bearing pressure should only be considered below a depth of 1.5m due to possible moisture variations where clay soils are present.
(5) VLS-Very Low Strength; LS-Low Strength; MS-Medium Strength; HS-High Strength; VHS-Very High Strength

It should be noted that when adopting the parameters for the encountered materials shown in the Table 13 above, at least 3.0m of the design foundation material must be present below the design foundation level for the footings.

For uplift loading the shaft friction values shown in Table 13 should be factored by 0.7.

A suitable geotechnical strength reduction factor (Φ_g) should be used when assessing the design geotechnical strength of piles. Refer to AS2159-2009 Section 4.3 for further advice regarding suitable geotechnical strength reduction factors. Without further information on the hospital, piling system and pile testing, a geotechnical strength reduction factor (Φ_g) of 0.45 is suggested but will need to be confirmed by the designer prior to construction.

The ultimate design geotechnical pressures presented in Table 13, in conjunction with a suitable strength reduction factor, are used to assess the "design geotechnical strength" (R_{dg}) of the pile, as defined in *AS2159-2009 Piling – Design and Installation* by considering the shaft and base areas. The design geotechnical strength must be greater than the "design action effect" (E_d).

The bases and sides of bored pile holes must be thoroughly cleaned of all loose soil and rock debris using a proper cleaning tool. The practice of adding water and spinning the auger is not acceptable. If there is any doubt as to the effectiveness of the base cleaning, the base resistance must be ignored.

Standing groundwater and seepages are likely to be encountered in bored pile holes below a level of approximately RL10.0m to RL15.0m. This seepage is likely to be significant, requiring the holes to be controlled by pumping or otherwise requiring the piles to be constructed under water or lined using bentonite liners or using tremie methods. The holes could also be lined with steel liners which will have to be socketed into low permeability material to achieve an impermeable seal against any water charged soils above. Shaft adhesion must be ignored for the portion of the pile that is permanently lined. As an alternative to bored piles, grout injected CFA piles could be adopted. Due to the presence of less weathered, high and very high strength rock zones near the surface a core barrel may be required to penetrate these zones.

Drilling piles is not only dependent on the subsurface material characteristics but also the type (power and size) of the bored pile drilling rig, drilling teeth, size of pile, etc. It is recommended that a specialist drilling contractor be consulted to be able to manage the above conditions and materials encountered.

During construction, all bored piles must be inspected by an experienced geotechnical engineer or engineering geologist to confirm the geotechnical strength parameters presented in Table 13 and to check the capacity of the piles.

The total long-term settlements of bored piles designed in accordance with the information given in this section should be limited to 30mm. Differential settlements should not exceed 50% of the total settlements.

14.2.1 Ultimate Lateral Resistance of Bored Piles

The ultimate lateral resistance parameters for bored piles in soils as well as modulus values for weathered rock for use in lateral load analysis of bored piles are presented in Tables 14 and 15 below.

Table 14 – Ultimate Lateral Resistance Parameters for Bored Piles in Soils

Material	Undrained Shear Strength C_u (kPa)	Ultimate Lateral Resistance (1) (kPa)
Stiff Clay	50	450
Very Stiff Clay	100	900
Hard Clay	200	1800

(1) *Note: For use in a Brom's type Analysis based on $q_{ult} = 9C_u$ (kPa)*

Table 15 – Modulus Values for Weathered Rock in Lateral Load Analysis of Bored Piles

Typical Geological Unit	Rock Weathering	Inferred RMR	NGI Rating, Q	UCS (MPa)	Rock Mass Modulus E_m (1) (MPa)
Unit 3A, 3B, 3C	XW/XW-DW	12	0.04	0.5-1.0	1100
Unit 3E	DW/DW-SW	34	0.61	10-50	4000
Unit 4	SW/SW-Fr	45	1.58	60-200	7500

(1) *Note: These values do not take account of possible vesicles in the basalt rock*

For rock socketed bored piles, the ultimate lateral resistance can be assessed using the following equation:-

$P_{ult} = 4\phi \times UCS \times B$ (MN/m)

- ϕ is the reduction factor which ranges between 0.3 for Unit 4 rock and 1.0 for Unit 3A, 3B or 3C rock;
- UCS (MPa) is the unconfined compressive strength of the intact rock; and
- B (m) is the pile diameter.

These resistance values are developed over a length of 3B below the top of the rock where the lateral resistance at the surface is zero and the resistance value at a depth of 3B is P_{ult}.

As an example for Unit 3A, 3B and 3C rock with a pile diameter of 0.75m, P_{ult} can be assessed as:-

$P_{ult} = 4 \times 1.0 \times 0.5 \times 0.75$
$P_{ult} =$ 1.5MN/m or 1.5kN/mm

For Unit 4 rock with a pile diameter of 0.5m, P_{ult} can be assessed as:-

$P_{ult} = 4 \times 0.3 \times 60 \times 0.5$ to $P_{ult} = 4 \times 0.3 \times 200 \times 0.5$
$P_{ult} =$ 36kN/mm to 120kN/mm

Analysis is usually carried out using commercial software such as Allpile by CivilTech Software in conjunction with p-y curves.

For the Unit 3E and Unit 4 rock, the strength of the pile will be less than the rock strength and will govern the ultimate lateral resistance of the pile.

15.0 PAVEMENT DESIGN PARAMETERS

Based on the subsurface conditions encountered on site, it is expected that natural clay soils of typically medium and high plasticity or the basalt rock will form the pavement subgrades for the majority of the site.

Laboratory testing was carried out on the natural clay soils from boreholes BH8, BH10, BH17, BH28 and BH40 to assess the typical soaked California Bearing Ratio (CBR). The CBR value represents the 'strength' of this material when compacted to 100% Standard Maximum Dry Density, under saturated conditions.

The soaked CBR tests produced results of 2.5%, 4.5%, 6.0%, 8.0% and 12.0% for the tested natural clay soils.

Based on the results of laboratory testing and testing of similar materials on nearby sites, the estimated pavement design parameters for new pavements to be constructed over new structural fill materials (either imported or re-used existing fill), natural clay soils, or the weathered basalt rock following compaction to 100% SMDD are shown in Table 16 below.

Table 16 – Estimated Pavement Design Parameters

Material [(1)]	Typical Design Soaked CBR Value	Modulus of Subgrade Reaction (kPa/mm)	Elastic Modulus, E (MPa)	
			Short Term	Long Term
New Structural Fill from Onsite [(2)]	2.0	25	8	7
Natural Clay (CI/CH) – Stiff to Hard	4.0	30	75	50
XW/DW Basalt (VLS/LS/MS) [(3)]	10.0	50	75	50
SW & SW-Fr Basalt (HS/VHS) [(3)]	>25.0	80	150	100

Note:
1) Soil classification based on visual descriptions in the field.
2) In accordance with AS3798-2007 reusing the excavated clay material from onsite.
3) VLS-Very Low Strength; LS-Low Strength; MS-Medium strength; HS-High Strength; VHS-Very High Strength.

Following the subgrade preparation, it is recommended that additional CBR testing be carried out to confirm the assigned design values.

The following general earthworks recommendations for pavement areas are made:-

- Perimeter drains to be incorporated at the pavement edges to prevent possible deterioration under wet weather.
- Pavements should be well drained during and upon completion of construction. Water should not be allowed to pond on or near pavement surfaces.
- Pavement gravel should comply with the Road and Maritime Services (RMS) quality specifications for sub base and base course materials or the relevant Australian Standards, local Council or the project specifications.
- Subgrades should be compacted to achieve a minimum density ratio of 100% SMDD at moisture contents about 2% wet of the OMC. We note that at present the natural clay soils are likely to be wet of OMC, based on the laboratory test results.
- Pavement materials should be compacted to the following minimum density ratios:
  - Sub Base 95% - AS 1289 5.2.1 (Modified).
  - Base 98% - AS 1289 5.2.1 (Modified).
- Inspections and testing should be carried out following general earthworks operations to confirm subgrade conditions.

16.0 EARTHQUAKE CONSIDERATIONS

In accordance with AS1170.4 – 2007 Section 4.2, the Site would be classified as Class C_e – Shallow soil site.

17.0 SITE MANAGEMENT AND CONSTRUCTION ISSUES

The main construction and site management issues are expected to be typically limited to the following:-

- Varying geology within the footprint of the multi storey hospital building including interbedded residual clay as well as XW, DW and SW-Fr basalt throughout the subsurface profile.
- Difficulties in penetrating the SW and Fresh basalt as well as the cobbles and boulders within the soil for pile foundations at various locations. The presence of cobbles and boulders within the volcanic clay soils can also pose other problems relating to construction as well as during the earthworks stage eg final trimming of pads and roads.

- Possible groundwater inflow issues during pile installation.
- Difficulties for excavations within the less weathered basalt rock.
- Problems for reuse of the excavated SW and SW-Fr basalt rock including difficulties in breaking down the rock to an aggregate size of 50mm or less.
- Retention of excavations during construction.
- Erosion and sediment control during construction.

The varying geology within the boreholes drilled for the proposed multi storey hospital building where the volcanic SW and Fresh basalt is encountered near the surface with some interbedded layers of XW basalt. The SW-Fresh basalt of very high strength then becomes XW and very low and low strength in numerous boreholes with some higher strength layers as well as some residual clay layers. This variation in strength and weathering of the basalt rock as well as the presence of residual clay will require careful consideration with regards to the foundation for the bored piles supporting the proposed hospital. To reduce the risk of settlements, it is expected that a conservative bearing pressure strength be adopted if the footings are founded within this variable material. Alternatively, all foundations can extend to the consistent levels of the SW and SW-Fr basalt which can support the multi-storey hospital. It is anticipated that the consistent level of the SW and SW-Fr basalt which can support the multi-storey hospital is typically encountered at a depth of approximately 14m to 15m in the slightly elevated, flat or gentle sloping terrain adjacent to Cudgen Road and up to approximately 20m in depth in several areas where the terrain grades into the mild sloping terrain to the north.

The presence of SW and Fresh basalt near the surface as well as cobbles and boulders within the volcanic clay soils can pose problems relating to construction as well as during the earthworks stage. Foundations for structures such as buildings or retaining walls may encounter basalt boulders up to 1.0m in size creating problems for the installation of footings such as bored piles. The presence of these basalt boulders may also pose problems during earthworks particularly for final trimming of pads and roads.

Recommendations with regards to the installation of piles such as problems associated with penetration into the SW and Fresh basalt which may require the use of a core barrel as well as groundwater issues are described in Section 14.0 above. Recommendations with regards to the retention of excavations during construction are also described above in Section 11.0.

Difficulties for excavations within the less weathered basalt rock as well as problems for reuse of the excavated SW and SW-Fr basalt rock including difficulties in breaking down the rock to a maximum aggregate size of 50mm or less is likely. Recommendations for excavations in this less weathered basalt rock as well as reusing this material as structural fill material are described in Sections 7.0 and 9.0.

An Erosion and Sediment Control Plan (ESCP) is outside the scope of this report, and as such, a proper description of erosion control techniques, including specific design requirements, maintenance issues and other information usually associated with an ESCP is not included. However, some general recommendations are made below.

It is important to remember that with proper erosion control in place before the excavation for the retention works, sediment control demands will be greatly reduced.

In general:-

- Erosion control methods should be favoured over sediment control measures;
- Drainage control is an effective means of erosion control;
- Protect and stabilise excavated/exposed soils;
- Stabilise excavation and construction traffic routes; and
- Control dust.

It is of utmost importance that external water flows are diverted around or away from the disturbed areas and stockpiles. Lined spoon drains or similar drainage channels are recommended to intercept and divert upslope water around the Site and should be positioned directly behind batter crests or the crests of retaining walls. The spoon drains should be connected by a lined surface and piped away from any excavations and to the proposed sediment basin. Spoon drain installation will have the effect of reducing the Site's erosion potential. As a temporary option, the spoon drains may be connected to holding tanks where required. Water quality testing will likely be required before transporting holding tank water offsite.

Sediment fences at the base of batters and surrounding the perimeter of all temporary soil stockpiles are recommended. It is also recommended that jute matting or similar geotextile protection is used between sediment fences and on sloping surfaces and in general areas of high surface water flows to reduce erosion and sediment runoff.

As a general rule, soil stockpiles and disturbed soil surfaces should not be left for long periods of time, unless properly covered and protected from wind and rainfall and by implementing the recommendations given in this report. Watering trucks should also be frequently used on site during excavation to limit the production of dust.

18.0 RESULTS OF PERMEABILITY TESTING

Constant head field permeability tests were carried out within a proposed sediment basin at the locations of boreholes BH22 and BH23 as part of the original investigation and also as part of a previous recent investigation. The tests were carried out within soils that appeared to be consistent with most of the conditions encountered across the site and in soils which contained some gravels and cobbles.

The constant head field permeability tests comprised the drilling of a 100mm diameter borehole to a depth of 0.5m. The hole was then saturated for approximately 30 minutes prior to the commencement of the test. The test was carried out using a permeameter setup on a tripod over the borehole.

Falling head permeability tests were also carried out as part of a previous recent investigation on soils obtained from the site which are expected to be typical of the soils used as fill in the construction of the proposed sediment basin bunds. The tests were carried out within soils that appeared to be consistent with most of the conditions encountered across the site, however no cobbles or large gravels were included.

The results of the permeability tests are presented in Tables 17 and 18 below with the permeability test certificates including time v permeability graphs presented in Appendix C.

Table 17 – Constant Head Permeability Test Results (Field)

Borehole	Location	Test Type	Material (m)	Test Depth (m)	Permeability (k) (m/s)
*BH22 (t1)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	1.90×10^{-04}
*BH23 (t1)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	5.70×10^{-05}
BH22 (t2)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	2.30×10^{-05}
BH23 (t2)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	1.70×10^{-05}
BH22 (t3)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	4.10×10^{-05}
BH23 (t3)	Mild Sloping Terrain (Proposed Sediment Basin)	Constant Head - Field	Silty CLAY (some gravel & cobbles)	0.10 – 0.50	2.20×10^{-05}

*(t1) - Denotes test 1; *Denotes test from previous investigation*

Table 18 – Falling Head Permeability Test Results (Laboratory)

Borehole	Location	Test Type	Material (m)	Test Depth (m)	Permeability (k) (m/s)
BH7	Slightly Elevated Terrain	Falling Head - Laboratory	Silty CLAY (minor gravel & no cobbles)	0.20 – 0.80	1.60×10^{-09}
BH10	Slightly Elevated Terrain	Falling Head - Laboratory	Silty CLAY (minor gravel & no cobbles)	0.20 – 0.80	9.60×10^{-10}

The results of the field constant head permeability tests indicate that the tested natural clay soils typically ranged between 10^{-4} and 10^{-5} m/s which is significantly higher than the usual range of the tested soil type (ie. Clay) which according to Taylor & Francis Group, LLC 2007 is typically in the range 10^{-8} and 10^{-12} m/s for a clay. This is most likely due to the insitu clay soils containing gravels and cobbles as well as the volcanic basalt clay soils being typically more permeable than clay soils associated with the nearby Neranleigh Fernvale Group.

The results of the laboratory falling head permeability tests indicate that the permeability of the tested soils obtained from onsite when compacted to 98% Standard Maximum Dry Density (SMDD) typically ranged between 10^{-9} and 10^{-10} m/s which is within the range of the tested soil type (ie. Clay) which according to Taylor & Francis Group, LLC 2007 is typically in the range 10^{-8} and 10^{-12} m/s for a clay soil. The variance between the field and laboratory permeability tests is most likely due to the insitu clay soil containing gravels and cobbles whilst the collected samples for the laboratory falling head permeability tests did not contain any cobbles or coarse size gravel.

19.0 SLOPE STABILITY ASSESSMENT

The entire development area including the proposed hospital, associated access driveways, car parking areas and other light weight single level buildings displays no evidence of recent past slope instability involving small-scale or large-scale movements of significant quantities of soil or rock in a short duration event such as slips, slumps, debris slides or a landslide. There are, however, localised areas within the mild sloping terrain which display minor evidence of slope instability in the form of creep movement of the surficial soil.

Most of the proposed development area is located within the slightly elevated, flat or gentle sloping terrain adjacent to Cudgen Road where the surface gradients are typically less than 5° and there is no evidence of slope instability. The northern end of the proposed hospital building as well as parts of the car parking area to the southwest slightly encroach into the mild sloping terrain where surface gradients are typically 10° but range up to 15° in localised areas. No major instability is evident within these areas of the mild sloping terrain other than minor creep movement at the surface. This minor creep movement is not expected to impact on the proposed development providing the recommendations of this report are followed.

The risk of slope movement in the form of deep seated failures through the soil or rock-mass below the surficial soil is considered to be low. This is due to the mild surface gradients, shallow depths to bedrock, no evidence of major instability and no evidence of groundwater within the sloping terrain.

Most of the site appears to be geotechnically stable at present with respect to slope stability. The main constraints or irregularities which would impact on the proposed development are typically limited to the areas of the development which are proposed to encroach into the mild sloping terrain. Drainage is important throughout the site, particularly in the mild sloping terrain where development is proposed. Drainage must therefore be addressed in these areas to reduce the potential for instability.

19.1 Site Specific Slope Stability Assessment

For the purpose of this assessment, the proposed development types have been categorised into the following areas and are shown on the attached Masterplan:-

Area 1 – Hospital (Slightly Elevated Flat and Gentle Sloping Terrain & part Mild Sloping Terrain)
Area 2 – Support building (Slightly Elevated Flat Terrain)
Area 3 – North eastern carpark & associated roads (Slightly Elevated Flat/Gentle Sloping Terrain)
Area 4 – South western carpark & associated roads/bays/service yards (Mild Sloping Terrain)
Area 5 – Main driveway areas (Slightly Elevated Flat Terrain)

A site specific slope stability assessment of each Area (Area 1 to 5) was carried out in accordance with the Australian Geomechanics Society (AGS) 'Practice Note Guidelines for Landslide Risk Management' Vol. 42 No1 March 2007. It must be noted that the Landslide Risk Rating assigned to each Area applies to the risk associated with the majority of the footprint of that Area.

On this basis, it can be expected that some parts of the Area footprint may contain higher Landslide Risk Ratings than the assigned rating for that Area footprint. Subsurface conditions have been obtained from borehole drilling at the site and onsite observations. Each individual Area footprint has been assigned Landslide Risk Ratings based on:-

- Ground surface slope angle and shape
- Geology
- Depth of soil cover and soil type (eg. slopewash, colluvium, residual)
- Presence of erosion features or surface irregularities
- Seepage and drainage conditions as assessed during the walkover survey carried out for this study

For the purpose of this assessment, each Area footprint has been assigned a number to identify each Area footprint as shown above. The results of the slope stability assessment for each Area footprint is presented in Table 19 below.

Table 19 – Landslide Risk Ratings – Individual Areas

Site Area	Type of Development	Likelihood of Instability Event	Consequences to Property	Landslide Risk Rating
Area 1	Proposed Hospital	Unlikely	Medium	Low
Area 2	Support building	Rare	Minor	Very Low
Area 3	North eastern carpark & associated roads	Unlikely	Minor	Low
Area 4	South western carpark & associated roads, bays and service yards	Unlikely	Minor	Low
Area 5	Main driveway areas	Rare	Minor	Very Low

The Landslide Risk Ratings for all of the proposed development at the site is assessed to be "Very Low or Low" in its existing condition (Areas 1 to 5). The risk maintenance and reduction strategies outlined in Sections 19.3 below are required to maintain the Landslide Risk Rating to a Level of "Low" or better for the long term.

19.2 Implications of Landslide Risk

The implications of Landslide Risk ratings are presented in the Australian Geomechanics Society (AGS) 'Practice Note Guidelines for Landslide Risk Management' Vol 42 No1 March 2007 and reproduced in Table 20.

Table 20 – Risk Level Implications (AGS)

Hazard Rating	Implications
VH (Very High Hazard)	Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low.
H (High Hazard)	Detailed investigation, planning and implementation of treatment options required to reduce risk to Low.
M (Moderate Hazard)	May be tolerated in certain circumstances but requires investigation, planning and implementation of treatment options to reduce the risk to Low.
L (Low Hazard)	Usually acceptable to regulators. Where treatment has been required to reduce risk to this level, ongoing maintenance is required.
VL (Very Low Hazard)	Acceptable. Managed by normal slope maintenance procedures.

19.3 Broad Recommendations for Development

Providing the recommendations for development outlined below are followed, the proposed development including the new hospital, other light weight single level buildings, new main driveway and new car parking areas are considered to be suitable and feasible for the long term development and should maintain a "Low" or better Landslide Risk Rating.

Recommendations for development are as follows:-

- No development should take place within any surface irregularities such as slumps, slopewash/colluvial soils, washouts, uncontrolled fill or erosion features. If development is to occur in these areas, surface irregularities must be removed prior to development and the drainage improved.
- All footings must found in the residual soil or weathered bedrock below all fill material, topsoil and slopewash soils which typically extend to a depth of approximately 0.35m but slightly deeper in some areas. Bored piles founding at least 4 pile diameters or at least 1.5m into the residual soil or weathered bedrock below the fill material or slopewash soils must be adopted in the mild sloping terrain. A suitably qualified structural engineer should be consulted with regards to sockets depths required to resist uplift and lateral forces.
- Any fill used in the construction of batters or behind retaining walls must be placed and compacted to at least 98% Standard Maximum Dry Density and be classified as "Controlled Fill" in accordance with A.S. 2870 (Clause 6.4.2 (a)) – "Residential Slabs and Footings" and A.S. 3798.
- Fill slopes should be no steeper than 2(H):1(V) and be over-constructed and trimmed back to the design geometry to ensure compaction in the outer zones. Natural soil batters should also be no steeper than 2(H):1(V).
- Prior to filling, all foundation slopes steeper than 8^0 (14%) must be benched to provide a suitable key for the fill materials.
- All permanent excavations exceeding 1.0m should be supported by adequately engineered retaining walls incorporating drainage, or battered at appropriate angles shown in Section 11.0.
- All retaining walls above 1.0m must be designed to have a factor of safety of at least 1.5 with respect to internal stability, including sliding and overturning. Retaining walls must also incorporate internal drainage behind the walls.
- All earthworks, if any, should be approved by a professional engineer or engineering geologist before being carried out.
- Vegetation clearing must be kept to a minimum and topsoiling with non dispersive topsoil should be adopted. Revegetation should commence immediately after the completion of earthworks to minimise the potential for erosion.
- All batters must be vegetated to minimise erosion. Properly maintained vegetation should reduce the occurrence of surface erosion by impingent rainfall.

- All development at the site must be designed, constructed and maintained in accordance with the attached Guidelines for Hillside Construction and the development examples of good and poor hillside practice.
- In sloping areas, surface contour drains should be constructed upslope of each building/site footprint to intercept and divert surface water flows into the stormwater system. This will reduce infiltration into the slopes and the potential for soil creep and erosion.
- All site runoff and roof water must be discharged into stormwater systems/drainage features via a system of pipe conduits or lined drains to minimise water infiltration into the slopes. Alternatively roof water can be discharged into water tanks for storage.
- All storage tank overflow water must be piped to the nearest drainage feature well downslope of the development or into the stormwater system. Uncontrolled discharge to the hill slope is not permitted.
- In the mild sloping terrain storage tanks must be located to the side or downslope of the development area.
- If batters and or retaining walls are to be constructed, all upslope surficial water flows should also be directed away from any batters and or retaining walls limiting the ingress of water into the fill or over and behind the retaining walls.

It should be noted that positive drainage must be maintained at all times to maintain the long term performance of the site in terms of slope stability and preventing, slips, soil creep and erosion.

The site is considered satisfactory for development in relation to slope stability and the Landslide Risk posed to the property can be maintained to a level of “Low” or better for the long term (at least 70 years) if the recommendations outlined above are followed and implemented, which is a tolerable level of risk.

20.0 LIMITS OF INVESTIGATION

This Report has been prepared by Morrison Geotechnic Pty Ltd, and may include contributions from Morrison Geotechnic’s officers and employees, sub-contractors, sub-consultants or agents.

This Report is for the sole benefit and use of Wood & Grieve Engineers Pty Ltd and associated parties for the sole purpose of providing geotechnical advice and recommendations in respect to the Proposed Tweed Valley Hospital Development at Cudgen Road, Kingscliff. The Report is only intended to address those issues expressly described in the scope of work in the Proposal Letter and this Report.

This Report should not be used or relied upon for any other purpose without Morrison Geotechnic’s prior written consent. Morrison Geotechnic and the Contributors do not accept any responsibility or liability in any way whatsoever for the use or reliance of this Report by anyone other than the Client or by anyone for any purpose other than that for which it has been prepared.

Except with Morrison Geotechnic’s prior written consent, this Report may not be:

(a) released to any other party, whether in whole or in part (other than to the Client’s officers, employees and advisers);

(b) used or relied upon by any other party; or

(c) filed with any Governmental agency or other person or quoted or referred to in any public document.

Morrison Geotechnic and the Contributors, do not accept any liability or responsibility whatsoever for, or in respect of, any use or reliance upon this Report by any third party. Morrison Geotechnic is not obliged to enter into discussions with any third party in respect of this Report.

The information (including technical information and information obtained through discussions) on which this report is based has been provided by the Client and third parties. Morrison Geotechnic and the Contributors:

(a) have relied upon and presumed the accuracy of this information;

(b) have not verified the accuracy or reliability of this information (other than as expressly stated in this Report);

(c) have not made any independent investigations or enquiries in respect of those matters of which it has no actual knowledge at the time of giving this Report to the Client; and

(d) make no warranty or guarantee, expressed or implied, as to the accuracy or reliability of this information.

Morrison Geotechnic and the Contributors do not accept responsibility or liability for any incorrect assumptions related to this Report. For the avoidance of doubt, this Report:

(a) cannot predict the ground conditions encountered at any untested location because the ground conditions surrounding a test sampling location (or between any two test sampling locations) may be different from the test samples we have obtained;

(b) is not an environmental, contamination or hazardous materials assessment; may be invalid, incomplete or inaccurate (including errors in the scope of work, investigation methodology, observations, opinions and advice) where the information provided to Morrison Geotechnic was invalid, incomplete or inaccurate;

(c) is limited to observations of those parts of the site that were accessible at the time of the field investigation and is not based on observations about areas of the site which were inaccessible to the investigation equipment (including slopes, heavily vegetated areas or service corridors); and

(d) is not a comprehensive representation of the actual site conditions and may only show a reasonable interpretation of conditions encountered at discrete test locations along with general site observations.

No warranty or guarantee, whether express or implied, is made in respect of the geotechnical data, information, advice, opinions and recommendations present in this Report. In recognition of the limited use to be made by the Client of this Report, the Client agrees that, to the maximum extent permitted by law, Morrison Geotechnic and the Contributors shall not be liable for any losses, claims, costs, expenses, damages (whether in statute, in contract or tort for negligence or otherwise) suffered or incurred by the Client or any third party as a result of or in connection with the information, findings, opinions, estimates, recommendations and conclusions provided in the course of this Report.

If further information becomes available, or additional assumptions need to be made, Morrison Geotechnic reserves its right to amend this Report.

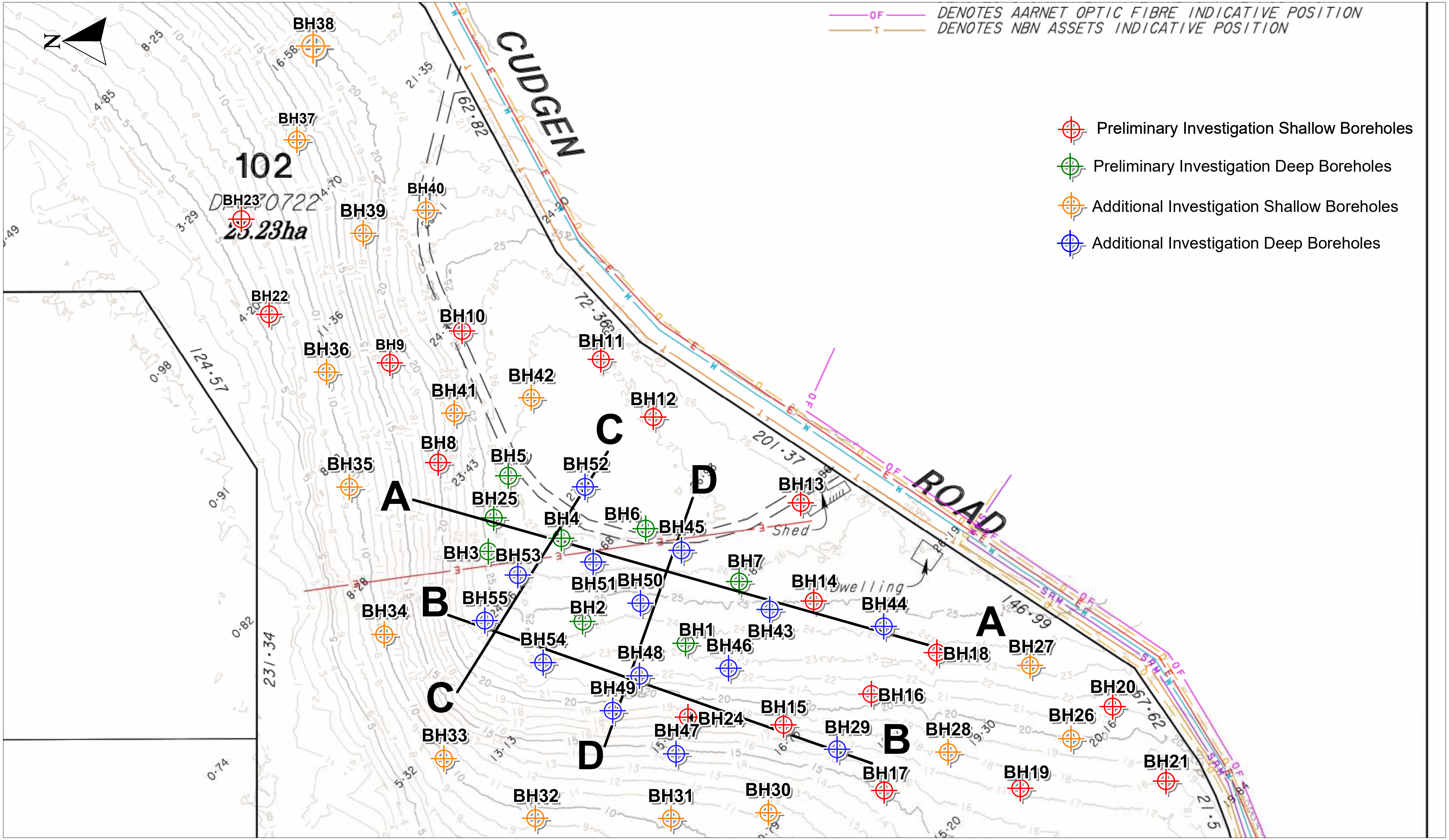
Please do not hesitate to contact the undersigned if you require any further information.

Yours faithfully

L. BEXLEY
for and on behalf of
MORRISON GEOTECHNIC PTY LIMITED

D. RILEY (RPEQ 5641)
for and on behalf of
MORRISON GEOTECHNIC PTY LIMITED

Encl Site Plan Showing Borehole Locations
Masterplan Concept Plan
Appendix A – Site Photos
Appendix B – Borehole Logs
Appendix C – Laboratory Test Results
Appendix D – Cross Sections of the Boreholes (Section A, B, C and D)
Appendix E – Guidelines for Hillside Construction
Important Information about your Geotechnical Engineering Report



MORRISON GEOTECHNIC

ABN: 51 009 878 899
Unit 1/5 Brendan Drive Nerang 4211 Ph: 5596 1599
Email: goldcoastlab@morrisongeo.com.au Fax: 5527 2027

Engineers: D.Riley, J.Daly, D.Dragun, R.Maxwell & B.Elsmore
Geologists: L.Bexley, J.Grentree & R.Howchin

Map Description :	SITE PLAN AND BOREHOLE LOCATION
Client :	WOOD & GRIEVE ENGINEERS
Project :	TWEED VALLEY HOSPITAL, KINGSCLIFF
Project No :	GE18/144 Date: 30/11/18 Scale : 1:3000 (A4)

NOTES

LANDSCAPE INDICATIVE ONLY - REFER LANDSCAPE ZONAL PLAN L-MP-12

LEGEND

- SITE BOUNDARY
- MAXIMUM PLANNING ENVELOPE
- TREE TRUNK LINE
- APZ OFFSETS
- PROBABLE MAXIMUM FLOOD LEVEL
- INDICATIVE BUILDING ENTRIES

N

DIMENSIONS

USE FIGURED DIMENSIONS, DO NOT SCALE. CONTRACTORS MUST VERIFY ALL DIMENSIONS ON THE SITE BEFORE COMMENCING ANY WORK OR MAKING ANY SHOP DRAWING WHICH MUST BE SUBMITTED AND REVIEWED BEFORE MANUFACTURE.

FIXTURES, FITTINGS & EQUIPMENT SPECIFICATIONS

SUBSTITUTE FF&E EQUIPMENT SPECIFICATIONS
THE FIT-OUT DESIGN AND DOCUMENTATION HAS BEEN COMPLETED ON THE BASIS OF FF&E AND EQUIPMENT ADVISED TO THIS OFFICE AT THE TIME OF BRIEFING THE DESIGN. THE DESIGN PROVISIONS FOR FF&E AND EQUIPMENT INCORPORATES SPATIAL ALLOCATIONS, SERVICING, LOADING AND ACCESS CLEARANCES AND WHERE APPROPRIATE SERVICES REQUIREMENTS, HAVING DUE REGARD FOR SURROUNDING FIXTURES AND FITTINGS. IT SHOULD BE NOTED THAT SUBSTITUTE FF&E OR EQUIPMENT WITH ALTERNATE SPECIFICATIONS SHOULD NOT BE PROCURED PRIOR TO VALIDATING THOSE SPECIFICATIONS AGAINST THE ITEM CONTROL SCHEDULE AND DESIGN PROVISIONS IN THE MODEL. THIS OFFICE ACCEPTS NO RESPONSIBILITY FOR THE PROCUREMENT OF SUBSTITUTE FF&E AND EQUIPMENT WHICH HAS NOT BEEN REVIEWED AND VALIDATED AGAINST THE ORIGINAL DESIGN PROVISIONS.

SERVICE POINTS DISCLAIMER

THE SERVICE POINTS IDENTIFIED ON THESE DRAWINGS HAVE BEEN OVERLAYED FROM THE BUILDING SERVICES ENGINEERING MODEL, AND ARE PROVIDED FOR INFORMATION AND SET-OUT PURPOSES ONLY. WE HAVE BEEN ADVISED BY THE SERVICES ENGINEERS THAT THE POINTS REPRESENTED ON THESE DRAWINGS ARE CORRECT AT THE TIME OF PUBLICATION. THIS OFFICE DOES NOT WARRANT THE ACCURACY OF THIS DATA. REFER TO THE BUILDING SERVICES ENGINEERING DOCUMENTATION FOR THE INSTALLATION OF THE SERVICES POINTS, AND THE INTERFACE BETWEEN THE SERVICES POINTS AND THE RESPECTIVE ENGINEERING SYSTEMS.

ISSUED FOR INFORMATION

ARCHITECTS

STH BATESSMART™

LEVEL 4, 89 YORK STREET,
SYDNEY, NSW, 2000
ABN: 73 561 269 057 PH: (02) 8299 4600
email syd@batessmart.com.au

43 BRISBANE STREET,
SURRY HILLS, NSW 2010
PH: (02) 8354 5100
email natalie@sth.com.au

PROJECT MANAGER

TSA MANAGEMENT

CLIENT

NSW GOVERNMENT | Health Infrastructure | Health Northern NSW Local Health District

PROJECT

TWEED VALLEY HOSPITAL
771 Cudgen Road, Cudgen

DRAWING TITLE

MASTERPLAN
CONCEPT PLAN

SCALE	DATE	DRAWN BY	CHECKED
1:2500 @ A3	19/09/2018	CE	MH

PROJECT No	DRAWING No	REVISION
10363	AR-SKE-10-006	04