REPORT TO

HEALTH INFRASTRUCTURE

ON

STAGE 2 GEOTECHNICAL INVESTIGATION

FOR

NEW COMMUNITY HEALTH CENTRE

AT

89-91 COWPER STREET, WARRAWONG, NSW

Date: 21 March 2023

Ref: 34300LXrpt2Rev1

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ATTACHMENTS

1 INTRODUCTION

1.1 Project Background

This report presents the results of a geotechnical investigation for the proposed new Warrawong Community Health Centre (WCHC) at the Port Kembla Hospital (PKH) site at 89-91 Cowper Street, Warrawong, NSW. The proposed WCHC is part of the New Shellharbour Hospital project being developed by Health Infrastructure NSW. The location of the site is shown in Figure 01 in the Attachments to this report. The investigation was commissioned by Health Infrastructure NSW and was carried out in accordance with the Health Infrastructure Consultancy Agreement (Contract No. HI21308BGE, dated 25 August 2021) and our Variation Request letters (Ref: 34300LXlet2 and 34300LXlet6) dated 12 September 2022 and 10 January 2023, respectively.

We understand that at least three options for the design of the new WCHC building were being considered as of April 2022, all to be located within the southern portion of the Port Kembla Hospital site. Of these, Options 2 and 3 were being evaluated as preferred options. These two options are indicated in the plan excerpts from the Cox Architects plans supplied with the brief (from April 2022), shown below.

Fig. 1 – Option 2 layout (pink shaded buildings within red boundary are to be demolished)

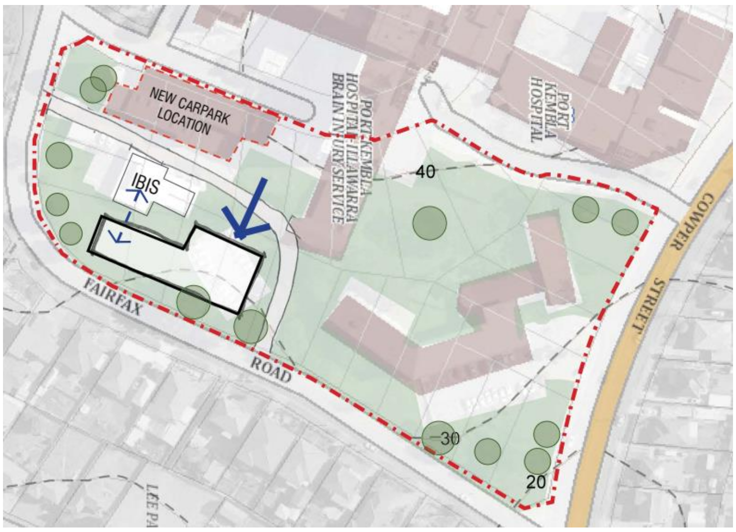


Fig. 2 – Option 3 layout, from April 2022 (pink shaded buildings within red boundary are to be demolished)

However, Option 3 was re-evaluated in December 2022 and a new location was proposed for the WCHC building at the northwest portion of the PKH site, where there is currently a car park area. The outline of the proposed WCHC building for this revised option is shown in Figure 3 below.

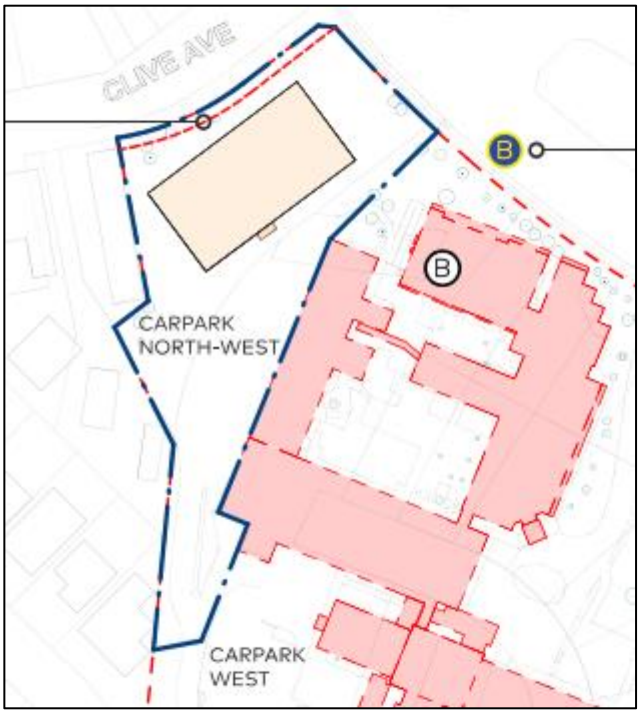


Fig. 3 – Revised Option 3 layout (December 2022)

Details of the proposed WCHC building were not available at the time of writing, but we understand the building may include at least one basement.

1.2 Scope and Objective of this Report

This report is an interpretive report that presents the factual data obtained from the geotechnical investigations carried out at the Port Kembla Hospital site, together with an analysis and interpretation of the information obtained. The interpretive section of this report provides the inferred geotechnical ground model of the site, the site classification as per AS2870:2011, seismic classification as per AS1170.4:2007, recommendations and/or discussion on geotechnical design parameters, groundwater conditions, material characterisation for earthworks and construction, general recommendations for foundation and pavement design, slope stability and key potential geotechnical risks associated with the proposed development. A gap analysis section is also included that provides recommendations for additional, targeted investigations that can be undertaken when the detailed design is further developed.

This report should be read in conjunction with the Contamination Assessment reports (both Preliminary and Detailed) prepared by JK Environments (JKE) for the site. The JKE reports present the findings and recommendations associated with the contamination-related desktop study and field investigations carried out at the Port Kembla Hospital site concurrently with the geotechnical investigation.

2 SITE DESCRIPTION

2.1 Topography

The Port Kembla Hospital is located at the top of a gently undulating ridgeline that trends roughly north-south in the Warrawong suburb of Port Kembla, NSW. The northern shore of Lake Illawarra lies about 600m to the south of the hospital site. The main part of the hospital lies at an elevation of about 40m AHD at the top of the ridge, with the eastern half sloping to the north-east with the lowest point at about 24m AHD. The proposed WCHC site is proposed within the eastern, gently sloping half of the overall hospital site.

The hospital site is flanked by Fairfax Road along the eastern boundary, by Cowper Street along the north boundary, and Clive Avenue at the northwest boundary. An internal access road branching from Vermont Road in the south borders part of the south boundary and the western boundary of the site. Three-storey buildings and an at-grade car park area are located in the north-east quadrant of the site with a broad, grass-covered area between the buildings/car park and Fairfax Road. Some smaller, single-storey buildings and smaller car park areas are situated in the south-east quadrant of the site.

In the northwest part of the hospital site there is a main car parking area that slopes uniformly from south to north, from an elevation of about 40m AHD to 36m AHD at the north end. The car park is bounded by a wire fence along the western and northern perimeters. Beyond the fence there is a grass-covered slope grading down to Clive Avenue and Cowper Street. The slope increases in height southwards along Clive Avenue, to a maximum height of about 4 metres, and has an average grade of about 1V:3H becoming less steep towards Cowper Street.

The Google Earth street-view image below is from the corner of Fairfax Road and Cowper Street in the north-east corner, looking to the south-west. The area that is the subject of the investigations for the WCHC options shown in Figs. 1 and 2 is approximately 90m wide and 200m long, or about 1.7 ha across the eastern half of the hospital grounds. The northwest car park area (proposed for revised Option 3), as shown in Fig. 3, is approximately 120m in length and about 50m wide.

It is noted that the northeast car park and most of the boundary along Fairfax Road contains a concrete crib-type retaining wall, that varies in height from about 0.3m to about 1.5m (estimated), as shown below.

Photo 1. View from the corner of Fairfax Road and Cowper Street

2.2 Geology

Examination of the 1:100,000 scale Wollongong-Port Hacking geological map (Sheet 9029-9129, Ed. 1, 1985) indicates the entire hospital site is situated in an area underlain by the Permian-age, Dapto Latite Member of the Shoalhaven Group. The latite rock is described as a coarse-grained, melanocratic and porphyritic latite (latite being an extrusive volcanic rock type).

A previous investigation carried out by JK Geotechnics in 1986 at the hospital site (on the western half of the site) comprised a total of 9 boreholes, all of which encountered latite rock at shallow depth. The two boreholes nearest to the eastern half of the site encountered the latite rock at less than 0.5m depth.

3 INVESTIGATION PROCEDURE

3.1 Summary of Fieldwork

The geotechnical investigation at the eastern part of the Port Kembla Hospital site (for Options 2 and 3) was carried out during two different periods: 4 days between 26 and 29 September 2022, and a subsequent 4 days between 31 October and 3 November 2022. The contamination sampling by JK Environments was carried out and completed in conjunction with the geotechnical fieldwork during the first period on site.

An additional investigation, for the revised Option 3 layout within the northwest car park area, was carried out between 6 and 10 February 2023. The JK Environments contamination soil sampling was also carried out and completed during this week, in conjunction with the geotechnical fieldwork.

The geotechnical fieldwork within the eastern part of the PKH site comprised the drilling of ten boreholes (designated BH-1 to BH-10P) to depths ranging between 5.73m and 8.59m below existing surface levels. The location of the boreholes is shown on the Borehole Location Plan (Figure 02), included with this report.

The fieldwork at the northwest car park area comprised of additional five boreholes (designated BH-101P to BH-105P) to depths ranging between 5.62m and 9.10m depth. The location of these five boreholes is also shown on the Borehole Location Plan (Figure 02).

Prior to commencement of drilling, a specialist sub-consultant scanned the borehole locations for buried services using electro-magnetic techniques.

All boreholes were initially advanced through the soils and upper weathered bedrock using spiral auger drilling techniques and a Tungsten Carbide (TC) bit. The boreholes were then extended through the bedrock to the final depths by rotary diamond coring techniques, using an NMLC triple-tube core barrel and water flush. A summary of the boreholes drilled, including their coordinates and elevations is presented in Table 1 below.

The strength of the soils was assessed from Standard Penetration Test (SPT) 'N' values (where a test was possible) augmented by hand penetrometer tests carried out on cohesive samples recovered by the SPT split tube sampler or disturbed lump samples recovered from the auger. The strength of the bedrock in the auger portion was assessed from observation of the drilling resistance using the TC drill bit attached to the augers and tactile examination of rock cuttings. It should be noted that strengths assessed in this way are approximate and variances of at least one strength order should not be unexpected.

For the cored portion of the bedrock, the recovered core was returned to our laboratory for photographing and Point Load Strength Index (Is_{50}) testing. The Point Load Strength test results are summarised in the attached Tables C and C1 in Appendix B and on the borehole logs presented in Appendix A.

The borehole locations and surface level at the borehole locations were obtained by a Sokkia SHC5000 mobile differential GPS (DGPS) unit. The DGPS achieves a horizontal accuracy of 50mm or better, and vertical accuracy typically better than 100mm. The horizontal grid system used in the survey was GDA2020 / MGA Zone 56. The elevations are referenced to the Australian Height Datum (AHD). The grid coordinates and ground surface elevations for all the boreholes are shown in the table below.

Table 1: Census of Geotechnical Boreholes – WCHC Site

BH No.	Termination Depth (mbgl)	Standpipe Piezometer?	Easting	Northing	Reduced Level (mAHD)
BH-1	5.73		305507.264	6181866.095	27.31
BH-2P	7.26	Yes	305483.227	6181898.159	32.08
BH-3	5.81		305519.247	6181887.046	28.54
BH-4P	6.16	Yes	305534.879	6181919.422	26.91
BH-5P	8.15	Yes	305507.874	6181951.763	28.49
BH-6	5.90		305462.135	6181845.521	31.07
BH-7	6.11		305483.652	6181819.833	28.17
BH-8	5.91		305464.625	6181795.055	31.27
BH-9	5.95		305482.265	6181787.714	29.53
BH-10P	8.59	Yes	305454.787	6181777.116	32.24
BH-101P	5.66	Yes	305347.590	6181972.641	39.20
BH-102	5.62		305386.177	6182021.597	37.25

BH No.	Termination Depth (mbgl)	Standpipe Piezometer?	Easting	Northing	Reduced Level (mAHD)
BH-103	5.63		305349.963	6182017.319	37.47
BH-104	6.61		305333.881	6182009.850	37.21
BH-105P	9.10	Yes	305361.973	6182043.730	36.70

Notes: mbgl denotes metres below ground level. Coordinates referenced to MGA2020.

Laboratory testing of soil and rock samples commenced in the week subsequent to the completion of the borehole drilling on site. Laboratory testing was undertaken by Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories. The testing completed is detailed in Section 5 below. The results are provided in the attached Appendix B.

Groundwater observations were recorded in all boreholes during and on completion of auger drilling. Standpipe piezometers were installed in six boreholes to allow for longer-term groundwater monitoring. Details regarding the standpipe installations is presented in Section 3.2.

A JKG geotechnical engineer was present on a full-time basis during the fieldwork, to nominate testing and sampling and prepare the borehole logs. The borehole logs, which include field test results and groundwater observations, are presented in Appendix A together with a set of Explanatory Notes which describe the investigation techniques and their limitations and define the logging terms and symbols used.

3.2 Piezometer Installations

A total of 6 standpipe piezometers were installed in selected boreholes for the purpose of monitoring groundwater levels and for sampling groundwater for contamination testing. Their location is indicated by the suffix "P" in the borehole number in Figures 02. Typically, the standpipe piezometers were installed as per the following methodology:

- A 50mm diameter PVC pipe with a machine slotted screened interval was installed in the boreholes;
- Filter sand was placed in the annulus around the slotted screen section with a minimum 0.5m thickness bentonite seal above the sand filter in order to isolate the screened section thus forming a response zone for groundwater level monitoring;
- The borehole was then backfilled above the bentonite seal with sand up to the surface;

Each standpipe was capped with a flush-mounted Gatic cover, installed level with the surrounding ground surface.

The standpipe piezometer installation details are provided in Table 2 below and the piezometer installation details are detailed on the borehole logs which are attached in Appendix A.

Table 2: Standpipe Installation Details

Borehole ID	Reduced Level (m AHD)	Well Depth (m)	Slotted Screen Interval (mbgl)	Material in screened section (refer to logs for detailed description)	Sample collected for testing?
BH-2P	32.08	7.17	3.0-7.17	HW to SW Latite, defect spacing <200mm below 4.3m	No

BH-4P	26.91	6.10	1.0-6.1	SW to Fresh Latite, defect spacing >600mm below 2.5m	Yes
BH-5P	28.49	8.15	3.0-8.15	SW to Fresh Latite, non-intact below 4.4m and defect spacing >200mm below 6.5m	Yes
BH-10P	32.24	8.50	1.5-8.5	XW (to 1.8m) to HW Latite, defect spacing <200mm	Yes
BH-101P	39.20	5.60	1.1-5.6	SW to Fresh Latite, defect spacing > 600mm below 2.5m.	Yes
BH-105P	36.70	9.1	6.1-9.1	HW Latite, defect spacing < 200mm	Yes

After the completion of drilling and well installation, all standpipes except for BH-2P were developed by pumping out the drill water introduced to the borehole during drilling. At least 3 well volumes were removed from each well.

4 RESULTS OF THE INVESTIGATION

4.1 Summary of Subsurface Conditions

The results of the investigation revealed a generally consistent profile across the site, comprising clayey fill overlying relatively shallow bedrock. A silty sand fill layer was logged in the upper 0.3m to 1m in boreholes BH101, 102 and 105P in the northwest car park area. The fill appears to be poorly to moderately compacted and its thickness across the site ranges from 0.4m to 3m, except at the borehole BH-105P location where fill was logged to a depth of 4.5 metres. Underlying the fill across most of the site is weathered latite bedrock, which grades into less weathered to fresh latite with depth. For a detailed description of the subsurface conditions at each location reference should be made to the borehole logs in Appendix A.

4.1.1 Fill

Clayey fill was encountered at the surface in all boreholes and extended to depths typically ranging from 0.4m to 1.6m, but deeper at borehole BH-3 and BH-5 locations (2.4m and 2.6m, respectively), and significantly deeper at boreholes BH-104 and BH-105P locations. The clayey fill appears to be poorly to moderately compacted, well graded, and is assessed to be of medium to high plasticity with inclusions of fine to coarse grained sandstone, igneous, ironstone and latite gravel, ash and root fibres.

Boreholes BH-104 and BH-105P were located near the top of the existing slope bordering the car park at the northwest corner of the site. It is possible that the deep fill thickness at this location represents reworked, excavated material that was edge-tipped over a pre-existing slope during earthworks re-grading works at the time the car park was being constructed.

4.1.2 Dapto Latite Member

Weathered latite bedrock was encountered below the fill in all boreholes and typically comprises a fine-grained, porphyritic rock. In borehole BH-104 there is a thin (0.2m thick) layer of stiff, residual clay between the base of the fill layer and the weathered latite bedrock. In borehole BH-105, a thicker layer of very stiff residual clay was logged in borehole BH-105P between 4.5m and 6.15m depth.

The latite is a volcanic rock type, and stratigraphically it represents several volcanic flow events that form the Dapto Latite Member of the Shoalhaven Group. The latite ranges from extremely weathered to fresh latite and is typically high to very high strength, where it is moderately weathered to fresh. Various extremely weathered and non-intact zones were logged within the top 4m to 5m of coring with defect spacings typically in the range of 60mm to 200mm. Significant core loss zones were logged in boreholes BH-5P, BH-9, BH-10P, BH-102 and BH-103. The latite rock becomes generally slightly weathered to fresh below depths ranging from about 2.5m to 4.5m in twelve of the fifteen boreholes drilled, exception being boreholes BH-9 and BH-10P at the southern end of the site and borehole BH-105P at the northwest part of the site. Where the rock is slightly weathered to fresh, the rock is of very high strength with defects typically at more than 300mm spacing. The latite is typically dark grey, green grey, grey and red brown with a speckled texture.

Defects within the latite bedrock primarily comprise joints inclined at between 5° and 90° though typically in the range of 40° to 90°. Other defects encountered included extremely weathered seams and fractured zones. The fractured zones may be the result of fragmenting of the rock during the coring process due to the presence of several, closely-spaced intersecting, and orthogonally orientated joints.

Zones of significant core loss are inferred to represent zones within the latite rock that may be extremely weathered, possibly to a residual clay material, on account of the rock mass being highly fractured. The possible occurrence of localised shear zones (which are typically sub-vertical) or a weathered, intruded basaltic dyke within the latite rock mass could also account for the zones of significant core loss, particularly in boreholes BH-05, BH-102 and BH-103. Late Triassic-age basaltic dykes are reported in the geological literature that intrude into the earlier Shoalhaven Group volcanics.

4.1.3 Groundwater

All boreholes were generally dry during and on completion of auger drilling. Measurements were recorded in the standpipe piezometers installed on the site during the fieldwork. A summary of the groundwater measurements is presented in the table below. The initial set of readings on 30 September 2022, shown in Table 3, were taken two days after well development carried out for the purpose of groundwater analysis sampling (as part of the contamination investigation). A subsequent reading on 3 November 2022 from the piezometer installed in BH-2P was obtained prior to demobilising from the western part of the site.

Readings were obtained from piezometers installed in boreholes BH-101P and BH-105P, at the northwest car park area, during the week the fieldworks were being undertaken.

Table 3. Standpipe Piezometer Groundwater Measurements

Borehole ID	Date of Measurement					
	28 and 30 Sep 2022		3 Nov 2022		9 and 10 Feb 2023	
	mbgl	RL (mAHD)	mbgl	RL (mAHD)	Mbgl	RL (mAHD)
BH-2P	Not read	-	6.80	25.281	-	-
BH-4P	2.18	24.726	Not read	-	-	-
BH-5P	1.49	27.001	Not read	-	-	
BH-10P	Dry	<23.744	Not read	-	-	-
BH-101P	-	-	-	-	1.60	37.60
BH-105P	-	-	-	-	2.45	34.25

Note: mbgl denotes metres below ground level

Groundwater levels generally appear to grade down towards the northern part of the site, which is expected based on the overall site topography. In the eastern part of the site, the groundwater is shallowest in boreholes BH-4P and BH-5P at the northern end where the ground levels are lowest and with a steeper slope towards Cowper Street. Based on readings from boreholes BH-2P and BH-10P, the groundwater level across the most of the eastern part of the site and at the southern end is relatively deep, at 6.8m or deeper below ground level.

In the northwest car park area, the groundwater level as measured in boreholes BH-101P and BH-105P is relatively shallow, just below the base of the fill layer in BH101P and in the middle of the deep fill layer in BH-105P. It appears the groundwater level gradient is towards borehole BH-105P where the ground level is lower than in the other four boreholes.

5 GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory testing of soil and rock samples has been carried out in general accordance with the following standards and guideline:

- AS 4133 Methods of Testing Rocks for Engineering Purposes
- AS 1289 Methods of Testing Soils for Engineering Purposes

Tables 4 and 5 below summarise the geotechnical laboratory soil and rock testing carried out. The results of the laboratory testing are discussed in the sections below and presented in Appendix B – Laboratory Test Results.

Table 4: Geotechnical laboratory testing – Soil

Laboratory test		Test method	Number of tests included in report
Field Moisture Content		AS1289.2.1.1	7
Atterberg Limits including Linear Shrinkage		AS1289.3.3.1	4
	pH	APHA 4500-H+	2

Classification of Soil Aggression against Steel and Concrete	Cl	APHA 4110-B	2
	SO_4 (sulphate soluble)	APHA 4110-B	2
	Electrical resistivity	APHA 2510 (Ed. 22)	2

Table 5: Geotechnical laboratory testing – Rock

Laboratory test	Test method	Number of tests included in report
UCS Testing on Rock Core (no modulus)	AS4133.4.3.2	6
Point Load Strength Index Tests	AS4133.4.1	91

5.1 Laboratory Soil Test Results

Limited laboratory testing was scheduled on recovered soil samples, as the boreholes generally encountered silty clay soil, logged as fill material, that is generally less than 1.6 metres deep across the site. The soil layer was deeper in boreholes BH-3, BH-5P, BH104 and BH-105P (2.4m, 2.6m, 3.2m and 6.15m, respectively) overlying highly weathered latite.

Laboratory classification testing was scheduled on a sample of the clayey fill material from boreholes BH-3 and BH-5 at the eastern part of the site, and on soil samples from boreholes BH-101P, BH-103, BH-104 and BH-105P in the northwest car park area. The results of these tests are summarised in Table 6 below and presented in the laboratory test reports Tables A to A2 in Appendix B.

The in-situ moisture contents of the clays were found to be significantly wetter than the plastic limit. The Atterberg Limit tests indicate they are high plasticity clays. The linear shrinkage values obtained (together with the corresponding plasticity indices) indicate these clay soils are highly reactive and may be classed as expansive soils. Reference should be made to the attached laboratory test reports in Appendix B for further details.

Table 6: Summary Table of Laboratory Test Results

BH ID	Sample Details			Plasticity				Natural Moisture Content (%)
	Depth (m)	Origin	Material	LS (%)	LL (%)	PL (%)	PI (%)	
BH-3	1.5-1.7	Fill	Silty Clay with sand and gravel	17.5	64	20	44	30.6
BH-5P	1.5-1.95	Fill	Silty Clay with sand and gravel	20.5	98	28	70	34.0
BH-101P	0.5-0.95	Fill	Silty Sand with gravel	-	-	-	-	16.0
BH-103	0.5-0.95	Fill	Silty Clay with gravel	-	-	-	-	20.7
BH-104	1.5-1.95	Fill	Sandy Clay with gravel	6.5	29	16	13	10.2
BH-105P	3.0-3.45	Fill	Silty Clay with gravel	15.0	63	20	43	38.6
BH-105P	4.4-4.95	Residual	Silty Clay	-	-	-	-	31.0

Two soil samples from selected boreholes were tested for aggressivity to buried concrete and steel. The following table summarises the soil aggressivity tests.

Table 7: Summary Table of Aggressivity Test Results

Borehole No.	Sample Depth	Sample Type	Soil pH (pH units)	Soil Chloride (mg/kg)	Soil Sulphate (mg/kg)	Resistivity (ohm m)
BH-104	1.5-1.95	Fill – Sandy Clay with gravel	6.6	<10	<10	560
BH-105P	3.0-3.45	Fill – Sandy Clay with gravel	7.0	10	260	43

Based on these results, the sandy clay fill material would be classified as having a 'Non-aggressive' exposure classification for both concrete and steel piles in accordance with Table 6.4.2(C) and 6.5.2(C) of AS2159-2009 'Piling – Design and Installation'.

5.2 Laboratory Rock Test Results

Point load strength index tests were carried out on the recovered rock core to assess the strength of the bedrock. The results are presented in the laboratory datasheets in Tables C and C1 in Appendix B. A summary of the results for the latite bedrock is provided in Table 8 below and graphically in Figure 07 in the Attachments to this report. The latite rock is typically of high to very high strength.

Table 8: Summary of Point Load Strength Index Testing

Unit	Range of $Is_{(50)}$ (MPa)	Average $Is_{(50)}$ (MPa)	Median $Is_{(50)}$ (MPa)
Dapto Latite	0.50 – 10.0	5.85	6.10

The unconfined compressive strength (UCS) was measured on six representative samples of intact latite rock core selected from six boreholes. The tests were carried out in general accordance with AS4133.4.3.2 test method. The laboratory UCS test results are summarised in Table 9 below and presented in the corresponding laboratory datasheet (Tables B and B1) in Appendix B.

The laboratory test datasheet (Tables C and C1) that shows the Point Load Index strength results also presents an estimated UCS strength obtained using a conventional correlation factor (K) equal to 20 as per AS1726-2017. However, for this project we have also assessed a site-specific correlation factor by comparing the six UCS test results with the closest Point Load Index results for each UCS test. The assessment indicates an approximate correlation factor (K) of 13 for the Lower Latite rock unit, although there is some scatter in the data. It is thus possible that the correlated UCS values presented in the laboratory test datasheets (Tables C and C1) may over-estimate the actual rock UCS strength values by as much as 50 percent. In any case, the adjusted correlated values would still correspond to a high to very high strength range.

Table 9: Summary of UCS Testing

Rock Unit	No. of UCS tests	UCS (MPa) values obtained	Rock strength classification as er AS1726
Dapto Latite (Lower Unit)	6	44; 58; 92; 96; 102; 117	High to Very High

Note: suitable core test specimens not available from Upper Latite unit, as rock was found to be too fractured.

6 COMMENTS AND RECOMMENDATIONS

No detailed drawings have been provided for the proposed development. The location of the site is also still being determined. Therefore, the following comments and recommendations are of a preliminary nature only and should be used to assist in the design development process. This report will need to be updated and revised to reflect the final scope of the proposed development, once it has been determined.

6.1 Site Classification

The site classification, with reference to AS2870-2011, of the WCHC site is largely determined by the type, thickness, and reactivity of the surficial soil units. Given that the encountered soils have been mostly logged as a fill material, likely to represent re-worked site-won material from when the hospital site was originally developed, and the site will be subject to abnormal moisture conditions due to trees and existing pavements, the site classification according to AS2870-2011 is Class P as the fill layer is more than 0.4m thick at most borehole locations.

Based on the reactivity potential of the clay fill material, we consider that shrink-swell movements in the range equivalent to a Class H2 site should be allowed for. The characteristic surface movement that can be expected in response to moisture changes in the reactive clay is dependent on its thickness and the moisture regime. Where the clay is thickest and potentially subject to significant changes in moisture content, the characteristic surface movement may be in the range 60mm to 75mm, in accordance with the estimates given in Table 2.3 of AS 2870 for H2 sites. Where the clay layer is thinner, lower characteristic movements would be predicted.

6.2 Ground Model

For the purpose of generating an interpreted ground model of the site, we have differentiated the encountered soils and bedrock into geotechnical units with similar properties. The Dapto latite bedrock has been subdivided into two zones (for simplicity designated as Upper and Lower Latite units) based on the degree of weathering, defect spacing and rock strength. Table 10 below shows the interpreted depth and reduced level of the top of each soil and latite rock unit used in developing the ground model.

Table 10: Depths and reduced levels of soil and rock units

BH No.	Fill Layer		Upper Latite		Lower Latite	
	Depth to top of unit (mbgl)	RL of top of unit (mAHD)	Depth to top of unit (mbgl)	RL of top of unit (mAHD)	Depth to top of unit (mbgl)	RL of top of unit (mAHD)
BH-1	At surface	27.31	0.70	26.61	2.65	24.66
BH-2P	At surface	32.08	1.20	30.88	4.50	27.58
BH-3	At surface	28.54	2.40	26.14	4.10	24.44
BH-4P	At surface	26.91	0.40	26.51	2.50	24.41
BH-5P	At surface	28.49	From 4.40 to 6.50	From 24.09 to 21.99	2.60 and 6.50	25.89 and 21.99
BH-6	At surface	31.07	1.55	29.52	3.30	27.77
BH-7	At surface	28.17	From 1.20 to 1.40 and 3.40 to 4.40	From 26.97 to 26.77 and 24.77 to 23.77	1.40 and 4.40	26.77 and 23.77
BH-8	At surface	31.27	1.40	29.87	3.10	28.17

BH No.	Fill Layer		Upper Latite		Lower Latite	
	Depth to top of unit (mbgl)	RL of top of unit (mAHD)	Depth to top of unit (mbgl)	RL of top of unit (mAHD)	Depth to top of unit (mbgl)	RL of top of unit (mAHD)
BH-9	At surface	29.53	0.80	28.73	Not reached	Not reached
BH-10P	At surface	32.24	0.50	31.74	Not reached	Not reached
BH-101P	0.02	39.18	Not present	Not present	0.12	39.08
BH-102	At surface	37.25	1.00	36.25	3.9	33.35
BH-103	0.02	37.45	1.50	35.97	4.1	33.37
BH-104	At surface	37.21	3.20	34.01	4.0	33.21
BH-105P	At surface	36.70	6.15, also 8.80 to 9.10+	30.55	7.6 and 8.8	29.10 and 27.90

Note: mbgl denotes metres below existing ground level.

In boreholes BH-5P, BH-7 and BH-105P there is a more closely fractured and/or weathered section within the Lower Latite unit that has similar properties to the Upper Latite unit, even though it is shown in the cross-sections as being overlain by Lower Latite rock unit. For simplicity, these closely fractured and/or weathered sections have been described as Upper Latite in the ground model. In BH-5P it is assumed that the significant core loss between 5m and 6.45m is due to the presence of more closely fractured and/or weathered latite rock between these depths.

Four inferred stratigraphic cross-sections are presented in Figures 03 to 05 for the eastern part of the hospital site, two of which are aligned approximately north-south and roughly sub-parallel to each other. The other two cross-sections are aligned approximately west-east. Two cross-sections are shown in Figure 06, aligned approximately north-south and west-east, across the northwest car park area (for revised Option 3 layout).

6.2.1 Eastern Part of Site

The Upper Latite extends from the initial top of bedrock contact in all the boreholes. It is characterised by extremely weathered (rock with soil-like properties) to moderately weathered, being closely fractured with defect spacings typically in the range of 60mm to 200mm. The intact rock strength ranges from 'hard soil' to very high strength, however the rock mass quality is generally governed by the defects. The Upper Latite is typically variously coloured red brown, brown and grey. The thickness of the Upper Latite stratum is generally in the range of approximately 2m to 5m across the site, except at the south-eastern part of the site (where ground elevations are highest) and where the Lower Latite was not intersected in boreholes BH-9 and BH-10P at termination depths of 5.95m and 8.59m, respectively. The intact rock strength of the Lower Latite unit is generally found to be of very high strength, with correlated UCS values of up to 200 MPa, and a defect spacing generally greater than 300mm.

In boreholes BH-5P , BH-7 and BH-105P there is a more closely fractured and/or weathered section within the Lower Latite unit that has similar properties to the Upper Latite unit, even though it is shown in the cross-sections as being interbedded within the Upper Latite rock unit. In BH-5P it is assumed that the significant core loss between 5m and 6.5m is due to the presence of more closely fractured and/or weathered latite rock between these depths.

6.2.2 Northwest Car Park Area

The two inferred cross-sections (Figure 06) show that the depth of clayey fill material increases towards the edge of the platform that the car park is built on. It is thickest in boreholes BH-104 and BH-105 at the crest of the slope grading down to Clive Avenue and Cowper Street. As explained in a preceding section, it is possible that this fill represents reworked, excavated material that was edge tipped at the time of construction. The relatively low SPT N values (4 to 8) recorded in the fill layer in these two boreholes further support this inference.

The top of the Upper Latite and Lower Latite are relatively uniform across the car park (cross-section F-F), with the Upper Latite encountered at a depth between 1m and 1.5m below the fill. The Lower Latite occurs at about 4m across this alignment. However, in the south to north direction the latite bedrock appears to slope downwards towards Cowper Street as shown in cross-section E-E. Notably, the Upper Latite was absent in borehole BH-101P at the south end of the car part, with high strength Lower Latite rock unit beneath a relatively thin layer of fill about 1m thick.

6.3 Excavation Conditions

The following comments and recommendations are provided on the basis that excavation for the proposed development will not extend deeper than about 3m depth, which allows for a single basement level, if a basement level is to be part of the final detailed design for the WCHC buildings.

Based on the encountered subsurface conditions, excavation is expected to be predominantly through soils and the weathered, more fractured Upper Latite rock unit. Excavation of the soils and any extremely weathered latite should be achievable using conventional earthmoving equipment, such as the buckets of hydraulic excavators.

Excavation of latite bedrock of low or higher strength will require rock excavation techniques such as rock saws, rock grinders and/or hydraulic impact hammers attached to large excavators. In general, it is not expected that the Lower Latite unit will be rippable, on account of its high to very high strength and the generally wider defect spacings. If possible, it is recommended that any excavations be limited in depth to the top of the Upper Latite, as indicated on the interpreted ground model cross-sections presented in this report. Where excavation for the proposed buildings encounter high to very high strength latite then this will produce hard rock excavation conditions. Excavation through such material will be slow and abrasive for excavation equipment. Specialised rock excavation equipment will be required which may include rock hammers and/or drill-and-split techniques.

Depending on the excavation technique employed by the building contractor, some techniques could induce ground vibration levels that exceed the maximum tolerance levels for a hospital building. Advice should be sought from the building contractor on the anticipated vibration levels that could generated for the excavation techniques employed on site. In addition, we attach to this report some guidelines on safe building vibration limits based on German standard DIN 4150.

In terms of bulking factors for excavations in the Upper Latite rock unit (which is likely to be the most prevalent rock unit excavated at the site) a bulking factor between 1.60 and 1.70 is generally recommended for basaltic-type igneous rocks.

The Upper Latite, where excavated, may be re-used on site as rockfill material (although it may require screening and/or secondary crushing) depending on earthworks material requirements for the project.

6.4 Groundwater

If excavations are proposed that will extend below the groundwater level, this will produce an added complexity to the design of basements and the associated drainage requirements. Measures may need to be adopted to reduce inflow rates into the excavation, and/or basements may need to be designed as tanked structures.

Given the groundwater levels measured in the four installed piezometers at the site, and assuming excavations will be limited to the upper 3 metres, it is not anticipated that excavations will extend below the level of the groundwater table at the eastern part of the site. At the northwest car park area, excavations down to 3m depth may intersect the groundwater table towards the base of the excavation, however it is expected that any seepage into the excavation can be managed by conventional sump pumping from the base of the excavation.

Water inflows into excavations above the groundwater table can be expected to be minimal and mostly due to rainfall infiltration in non-paved areas of the site . If deemed appropriate, a more thorough assessment of likely groundwater inflows into excavations can be undertaken once details of proposed basement footprints and depth extents are known.

6.5 Batter Slopes

All excavations should be carried out with reference to the latest version of *'Excavation Work – Code of Practice'* by SafeWork NSW.

6.5.1 Temporary Batter Slopes

We anticipate that sufficient space will be available for forming temporary batters for excavations up to 3m deep. As a preliminary guide, temporary batters formed through clay fill layer may be formed no steeper than 1 Vertical (V) in 1 Horizontal (H).

Steeper temporary batters should be feasible where better quality bedrock is exposed in the excavation, although the stability of such batters will largely be governed by the defect spacing and orientation. As a guide, for excavations into the latite bedrock we recommend the temporary and permanent batter slopes shown in Table 11 below.

Table 11: Recommended maximum temporary batter slopes for cuttings in rock

Material	Temporary Batter
Upper Latite	1V:1H
Lower Latite	1V:0.5H to Vertical

The following should be noted with regard the above recommended batters:

- Where defects are found to be unfavourable, flatter batters may be required and/or stabilisation measures such as rockbolts, shotcreting, etc. installed;
- If space is not available to flatten batters, then a generous budget allowance for additional stabilisation measures should be allowed;
- All cuttings should be inspected and assessed by an experienced geotechnical professional during construction, progressively as the excavation is advanced and at not more than 1.5m depth intervals;
- The above batter slopes are for un-supported cuttings that are above the groundwater table and not affected by seepage;
- No discharge of stormwater should be permitted along the crest zone of temporary batters;
- The recommended batters assume there are no structures or surcharges near the crest zone of the cuttings.

Where retaining walls are to be constructed in front of temporary batters, care will need to be exercised in backfilling between the temporary batter slope and the new retaining wall. Uncontrolled backfilling will lead to large settlements which may adversely affect pavements, structures or landscaping areas behind the walls. It is often difficult to achieve adequate compaction of backfill due to limited access and the need to use small hand compaction equipment. We recommend therefore the use of a single-sized durable gravel, such as “blue metal” gravel or crushed concrete (free of fines and with less than 10% brick), which does not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34, or similar, should be placed as a separation layer immediately above the cut batter slope to control subsoil erosion. Provided the gravel backfill is placed as recommended above, density testing of the gravel backfill would not be required. The geotextile should then be wrapped over the surface of the gravel backfill and capped with at least a 0.5m thick compacted layer of clayey engineered fill.

6.5.2 Permanent Batters

Permanent batter slopes will likely be suitable for transitioning between existing and proposed surface levels around landscaped and pavement areas. The formation of permanent batters will be dependent on the height of the cut and the materials exposed. As a guide, we suggest permanent batters through the surficial soils should be battered at not steeper than 1 Vertical (V) in 3 Horizontal (H). Permanent batters through the latite bedrock should be battered at not steeper than the slopes indicated in Table 12 below.

Table 12: Recommended maximum permanent batter slopes for cuttings in rock

Material	Permanent Batter
Upper Latite	1V:1.5H
Lower Latite	1V:0.5H

The stability of batters cut into rock, and in particular the Upper Latite unit, will be largely governed by the defect spacing and orientation. For this reason, the above recommended batters are subject to detailed inspections and assessments by a geotechnical engineer that are carried out progressively as the excavation advances. The following should also be noted with regard the above recommended permanent batter slopes:

- Where batters are higher than 3m, a 2m wide mid-height bench should be provided as a minimum, however the actual batter slope will need to be specifically designed by the geotechnical engineers;
- Some localised instability due to fretting or erosion may occur in the more fractured Upper Latite, and may require installation using additional measures such as shotcrete and/or mesh;
- For batters graded at or steeper than 1V:1H in rock, the stability of the rock cutting will largely be dependent on the joint spacing and configuration. Isolated rock bolts, or pattern bolting, may be required where there is a potential for joint-defined wedges or blocks to become dislodged.

Any permanent batters will need to be fully protected from erosion, in the long term, by suitable and approved erosion protection measures. Suitable measures could include re-vegetation (in soil slopes) or shotcrete. Where re-vegetation is proposed for soil batters, consideration should be given to flattening the permanent batters even further than recommended above to assist with initial vegetation and topsoil establishment, to reduce the risk of topsoil washing from the face during heavy rainfall, and to provide for ease of maintenance. Erosion protection may not be required if batters will be located within undercroft areas, subject to approval from the geotechnical engineers.

In the case of rock batters cut into very fractured rock (such as the Upper Latite unit) that is susceptible to fretting, long term or permanent protection will be required if progressive fretting erosion is not acceptable.

6.6 Retaining Structures

Where steep cuts are proposed, primarily in soils, such as for temporary excavations (pits and trenches) then a suitable retention system may be required (as determined by a specific geotechnical assessment). The retention system adopted may, in certain cases, be incorporated into the permanent structure (for example a lift shaft). There are various types of retention systems that are commonly used locally and that could be considered at the WCHC site, including: soldier pile walls with shotcrete infill panels; concrete walls (including infilled cellular panel walls); gravity walls and various types of concrete piled walls. Depending on required stiffness of the retention system (to limit ground deformation behind the wall) the retention systems may additionally be braced and/or anchored.

Consideration will need to be given to whether it is a temporary or permanent retention system. Temporary excavations, such as for trenches or pits, can adopt appropriately-designed shoring boxes to provide temporary support to vertical sidewalls. A summary is provided in Table 13 on the following page that outlines the key advantages and disadvantages of the more commonly used retention systems. This is not a comprehensive summary, but is provided here as a guide.

It should be noted that all retention systems will experience some lateral movement towards the excavation. The magnitude of this movement is typically up to about 0.3% of the retained height for a non-braced system that has been adequately designed. Factors such as groundwater, adjacent surcharge loads, and nature of the material being retained all have an impact on the behaviour of the retention system. For movement-critical cases, a more thorough geotechnical analysis (for example using numerical modelling) will be needed to assess the expected ground response.

The recommended earth pressure coefficients for the design of retaining systems are shown in Table 14 below. The values for the Upper Latite unit are estimated lower-bound values assuming it is extremely weathered and closely fractured (<100mm spacing). It should be noted that the earth pressure values shown assume that the ground behind the wall is effectively level.

Table 14: Recommended earth pressure coefficients for design of retaining systems

Material	Bulk Unit Weight (kN/m^3)	Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)	At Rest Earth Pressure Coefficient (K_0)
Clayey Fill	18.0	0.4	3.0	0.7
Upper Latite	22.0	0.3	3.5	0.5

Retaining structures should be designed to support full hydrostatic groundwater pressures acting against the back of the retention system unless an effective drainage system has been provided to reduce this pressure. Even allowing for provision of a drainage system, it is considered prudent to allow for temporary groundwater pressures behind the retaining structure of up to one-third the wall height. Surcharge loads must also be accounted for in the design.

In the case of reactive soils, it will be important to consider the potential for swelling pressures acting at the back of permanent retaining walls. Swelling pressures are high and cannot be economically retained. Therefore, particular care in protecting the backfill or the reactive soils being supported from “wetting up” will be important. Allowing for any backfilling behind the wall to include free-draining engineered fill and provision of an appropriate drainage system will help reduce the risk of excessive swelling pressures impacting the wall.

The design of any retention system will also need to consider the potential impact of any ground movements behind the retention system and how these could affect nearby structures and services.

Table 13: Common retaining wall systems – key advantages and limitations

Retention System	For Permanent / Temporary Support?	Construction Type	Key Advantages	Key Limitiations
Mass Gravity	Normally permanent structures	Various types: from gabion walls, crib walls; masonry walls; to reinforced concrete walls	Generally economical to build; Do not generally require heavy, specialist plant	Generally limited to about 3m height; Requires pre-excavation to construct; Foundation area requires preparation; Generally require imported backfill; Higher lateral deflections compared to more rigid systems; Generally not watertight
Cantilever	Can be for both	Various types, including: sheet piling; reinforced concrete; infill panel walls (e.g. Dinsel)	Can be built post-excavation; Additional lateral support can be provided by bracing, tie-back anchors, walers and props; Greater heights (25m+) can be achieved with back-anchoring; Good watertightness can be readily achieved	Generally require access for heavier plant; Sheet piling not suitable where obstructions or penetrating into bedrock
Piled Wall (embedded)	Normally permanent structures	Various types, including: Steel soldiers; concrete soldiers; contiguous and secant bored piles	Suitable for deep basements; Good watertightness can be achieved with secant piles or addition of shotcrete; Lateral deflection can be minimal (with addition of anchoring, if required); Can also support axial surcharge loads	Generally require access for heavier plant;
Shoring systems	Normally for temporary support only (e.g. services trenches)	From simple systems using timber, to designed steel systems with hydraulic jacking	Economical and quick to install; Proprietary systems available from specialist suppliers	Limititations as to height and length retained; May not be suitable for certain ground conditions (saturated sands); Require adequate design checks and regular inspection

6.7 Earthworks

Earthworks recommendations in this report should be read in conjunction with AS3798-2007: '*Guidelines on Earthworks for Commercial and Residential Developments*' which should also be adopted.

Earthworks on this site will need to consider a number of factors, some of which are highlighted below. These factors need to be considered as part of the overall design development for the buildings and in particular when considering earthworks operations on the site;

- The full extent of excavation and filling required, and whether it is more economical to re-use existing materials (with their adverse geotechnical characteristics discussed further below), to import fill materials, or to fully suspend structures.
- The reactivity of the existing fill and its potential for shrink-swell movements. We highlight that shrink-swell movements for a newly placed fill comprising reactive clay soils will be greater than the current insitu shrink-swell movements.
- The site is underlain by fill which is typically poorly to moderately compacted. This fill must be assumed to be uncontrolled and will not be suitable for support of structures. It may be suitable for support of pavements where some movements are permissible, but will need to be more specifically assessed and tested when further details are provided of the proposed development.
- The fill materials have moisture contents significantly greater than the plastic limit and its re-use will require moisture conditioning.

6.7.1 Subgrade Preparation

Subgrade preparation will depend to a large degree on the nature of the works and how the items in the section above are addressed. The recommendations below are of a general nature and will need to be refined when full details of the proposed development are available. The recommendations are for lightly loaded pavements where some differential settlement of the surface is acceptable. For a more stable surface, or to support structural loads, all fill material would need to be removed and replaced, or reworked if intended for general fill.

Following topsoil stripping and re-grading of the site during the construction phase, the existing subgrade should be prepared by removal of unsuitable materials. Unsuitable materials may include oversize materials, building waste and/or building rubble, organic material or deleterious materials. Some unsuitable material may be re-usable with approved treatment. Therefore, prior to disposal of any unsuitable material from site, further specific assessment should be undertaken by an experienced geotechnical engineer.

The prepared surface should be proof-rolled to verify that no soft pockets or areas of unsuitable materials are present. This would typically be carried out using a smooth, non-vibratory type roller typically of minimum weight of around 12 tonnes. The proof rolling should be witnessed by an experienced practitioner to detect any soft, wet or heaving areas. Where such areas are encountered, the areas should be improved by an appropriate method, which could include:

- Excavate and replace with suitable, engineered fill material;
- Moisture-conditioning of material that is too wet of optimum;
- Placement of geosynthetics (e.g. geogrids), in combination with granular materials, to form a bridging layer over localised soft zones.

Site won materials, from re-grading operations in other areas of the site, may be used as engineered fill in other areas of the site, provided they meet the acceptance criteria for general fill that will be specified for the project.

6.7.2 Material Re-usability

Site-won material reusability has been assessed in terms of TfNSW Specification Guide NR44 – Earthworks. Our assessment indicates that the clay fill material obtained from excavation at the site could be re-used as general fill only. Consideration would, however, need to be given to the material's moisture content and its swelling potential. The natural moisture content results in the fill material were between 6% and 19% wetter than the plastic limit, the exception being the fill layer in BH104 that was drier than the plastic limit. This could be attributed to its location at the crest of the slope in the northwest car park area. For most of the site, It is likely that the moisture content of the clay fill material will be wetter than the optimum moisture content for compaction. Site-won fill material intended for re-use may thus require moisture conditioning (probably drying out) prior to placement.

Given the clay fill is assessed as potentially highly reactive, consideration will be need to be given to the potential for shrink-swell movements as these could affect any services or structures that are built directly on them or within these soils. The re-use of these reactive soils should be used with caution and after consultation with the geotechnical engineer who can advise further on specific issues for the potential re-use and how movements may impact structures.

The Upper Latite rock unit, if it is to be excavated, could potentially be re-used on site depending on its final size grading after digging / ripping. Some secondary screening may be needed to remove oversize material. For compacting in layers up to 300mm thick, the recommended maximum size should not be greater than 75mm.

6.7.3 Engineered Fill Recommendations

Engineered fills (either site-won or imported soil materials) that are placed following subgrade preparation, where they are required as a support base for structures and/or pavements, should be compacted to between 98% and 102% of Standard Maximum Dry Density (SMDD), placed in layers not exceeding 300mm loose thickness. The engineered fill should be placed within 2% of optimum moisture content and compacted without delay using appropriate compaction plant. Imported rockfill and site-won rock materials, such as Upper Latite rock unit (following any secondary screening), should be compacted to not less than 98% of SMDD.

It is recommended that verification and placement of engineered fills be carried out under Level 1 geotechnical inspection and testing (GITA) as defined in AS 37978-2007 *Guidelines on earthworks for commercial and residential developments.*

6.8 Footings

6.8.1 Shallow Footings in Soils

Lightly loaded structures, such as single-storey buildings, may be founded on shallow pad or strip footings, or on reinforced concrete raft slabs within properly compacted engineered fill material (following reworking of any existing fill) that is free of unsuitable material. For all shallow pad/strip footings, edge and internal thickenings to raft slabs embedded into and founded on engineered fill material, an allowable bearing pressure of 100kPa may be adopted. The design of shallow pad/strip footings and raft slabs on engineered fill will depend on the type of engineered fill used. Where the existing clayey fill is used as engineered fill then further specific advice must be obtained from the geotechnical engineers in regard to potential shrink-swell movements. However, movements will likely be at least in the range similar to a Class H2 site in accordance with AS2870-2011. For footings within engineered fill, we recommend that Dynamic Cone Penetrometer (DCP) tests be completed to confirm the materials are well compacted, however detailed Level 1 records will also be necessary to verify that the engineered fill has been appropriately placed, inspected and tested. It should be noted that re-use of reactive clays within an engineered fill will increase the potential for shrink-swell movements and therefore due allowance must be made in the footings design for these additional movements.

Prior to pouring the footings, it is recommended that the founding stratum be inspected by a geotechnical engineer to ensure there has been no softening of the foundation material (due to water ingress) and that the foundation material is consistent with the design bearing pressure. Water should be prevented from ponding in the base of footing excavations as this will tend to soften the foundation material, resulting in further excavation and cleaning being required.

Consideration will need to be given to differential settlements that may occur between different sized footings or footings founded on different materials. Once detailed footing loads and sizes have been determined, we recommend that the geotechnical engineers be requested to provide specific advice on footing movements at each footing location so that a more thorough assessment of likely differential movements can be made. Where shallow footings are adopted movement control joints in structures is recommended, particularly where these light weight structures abut structures founded on the underlying bedrock.

6.8.2 Footings in Rock

For more heavily-loaded structures and higher column loads, or where supporting structures on engineered fill is not desirable (such as to avoid reactive movements), piled footings supported on the underlying latite bedrock will be required. The buildings should then be suspended from the piled footings, with void formers below all ground slabs and beams in order to isolate the structure from any potential reactive movements. Bored piles will be feasible for piled footings, however large piling rigs will be required to penetrate through even the Upper Latite rock unit. Any potential piling contractors should be provided with a copy of this geotechnical report (including the borehole logs and rock strength testing) so that they can confirm that they have suitable equipment to penetrate the rock. Slower than normal drilling progress and high bit wear should be expected. We recommend that pile sockets within the Lower Latite be limited as much as possible as penetration of such rock may be very problematic and may require coring buckets. However, the inferred cross-sections shown in Figures 03 and 04 suggest the thickness of the Upper Latite unit is between 2m and 2.5m across most of the site, except at the southern end of the site where it is considerably thicker. Furthermore, the upper part of this Upper Latite unit is likely to be extremely weathered exhibiting soil-like properties. It may thus be viable to extend pile foundations down to the Lower Latite unit in these areas.

Groundwater could be encountered near the base of the pile holes and piles may therefore need to be pumped free of water prior to pouring concrete or poured using tremie techniques.

Footing design parameters for the latite rock have been derived with reference to the general design parameters recommended by Pells (1988)[1] for Sydney sandstones and shales. Although the geology is different to the Sydney sandstones and shales, we consider that the paper provides a sound guide for design of footings on rock. By adopting a similar principle in regards to rock classification as presented in Pells 1988, we consider that the Upper Latite is comparable to a Class IV/V sandstone (in terms of defect spacing and weathering) while the Lower Latite is comparable to Class I/II sandstone.

The following table provides our recommended serviceability and ultimate bearing pressures for footings founded on rock, for compressive loads. These recommendations have taken into consideration the rock type, its strength, the rock classification, and reasonable proving requirements to confirm that an appropriate founding material has been reached. Where footings are designed to be founded on the Lower Latite unit, further cored boreholes are recommended to refine the likely founding depth to reduce the risk of excessive sockets through the higher strength bedrock.

Table 15: Geotechnical design parameters –Foundation in Latite rock units

Rock Unit	Serviceability Bearing Pressure (MPa)	Ultimate Bearing Pressures (MPa)	Serviceability Shaft Adhesion[1] (kPa)	Ultimate Shaft Adhesion (kPa)[1]	Elastic Rock Modulus (MPa)
Upper Latite	0.7	3	100	150	100
Lower Latite	6	60	400	2,000	2,000

Note:

1. Shaft adhesion values assume a rock socket roughness of R2 or better. Shaft adhesion should be included where piles are embedded at least 2 pile diameters into founding bedrock.
2. In the case of piled footings working in tension, a reduction factor of 0.5 is recommended on the ultimate and serviceability shaft adhesion values shown (for compressive loads).

The use of the serviceability end-bearing and shaft adhesion values recommended in Table 15 generally limit footing settlements under working load to around 1% of the minimum footing width or pile diameter

[1] Pells, P.N. et. al. (1988). Foundations on Sandstone and Shale in the Sydney Region. J. of Australian Geomechanics, December 1998.

For limit state design (as per AS 2159:2009), the design geotechnical capacity can be obtained by applying a geotechnical reduction factor (ϕ_g) to the ultimate capacity assessed using the ultimate-bearing pressure and ultimate shaft adhesion values shown in Table 15. Reference should be made to the AS 2159:2009 piling code for selection of an appropriate geotechnical reduction factor. Where limit state values are adopted, large settlements will occur, potentially greater than 5% of the minimum footing width or pile diameter. Therefore, specific settlement analysis is required to assess footing settlements where limit state design is carried out.

It should be recognised that accurate prediction of footings settlements is a function of several factors, including construction methodology, pile group effects, and assessed material properties.

Where shallow pad/strip footings or piles are founded in the Upper Latite we consider that all pad/strip footings and the drilling of all pile footings should be visually inspected by a geotechnical engineer. Where pad/strip footings or piles are founded within the Lower Latite unit, we consider that all pad/strip footings and the drilling of all piles should be visually inspected by the geotechnical engineers and in additional one third of all pad/strip footings should also be spoon tested. These inspections are necessary to confirm that a suitable founding stratum is being achieved.

The cross-sections in Figures 03 to 06 indicate that some variability in founding conditions may be expected across the site, particularly at the southern end of the site in the eastern part. Careful attention to likely differential settlements as a result of the size of shallow footings or piles and the possibility that they will be founded on different founding stratums needs to be carefully considered by the design engineers. Ideally all footings would be founded on the same or similar founding stratums, however on sloping sites this is not always feasible.

Footings should be excavated/drilled, cleaned, inspected, and poured with minimum delay to avoid deterioration. If delays in pouring concrete are anticipated, we recommend that the base of shallow footings be protected with a layer of blinding concrete. Water should be prevented from ponding in the base of footing excavations and pile holes as this may lead to soften the foundation material, resulting in further excavation and cleaning being required.

It is recommended that all pile foundations be inspected by a geotechnical engineer, during construction, to verify that the rock quality at the base of the pile meets the design criteria and that a suitable depth of embedment has been achieved. The contractor should also ensure that the base of the pile excavation (for bored piles) has been cleaned out prior to concreting and there is no loose material at the base of the pile bore.

6.9 Basement/Ground Floor Slabs

Following earthworks, the subgrade below basement or ground floor slabs may likely comprise a combination of different strengths of latite bedrock, in-situ fill material or engineered fill. Once the subgrade has been exposed, we recommend a visual inspection by an experienced geotechnical engineer be carried out to assess the subgrade. Where clay fill material or engineered fill is exposed at subgrade level the subgrade will need to be prepared as discussed in Section 6.7.1 above. It is likely that clay fill will need to be fully stripped and reworked (or replaced) unless structures are designed as fully suspended on pile footings. Where bedrock is exposed or floor slabs are suspended, no particular subgrade preparation is required.

Provided the subgrade is prepared as discussed in Section 6.7.1 above, basement or ground floor slabs can be supported on engineered fill, on the assumption that they are constructed separately to the building footings, so that relative movement can occur. Movement control joints should be placed in basement or ground floor slabs where the subgrade transitions from soils to weathered latite bedrock.

Ground floor slabs should be supported on at least a 100mm thick sub-base of durable fine crushed rock such as DGB20 (RMS QA Specification 3051 unbound granular material) even if the exposed subgrade is a weathered rock material. The sub-base should be compacted to a minimum density ratio of 100% of SMDD. Adequate moisture conditioning to within 2% of SOMC should be provided during placement. The sub-base material will provide a separation layer between a rock subgrade and the slab and where a soil subgrade is exposed will provide more uniform slab support and reduce the risk of 'pumping' of subgrade 'fines' at joints due to vehicular movements.

Where relative movement between basement or ground floor slabs founded on engineered fill and structural footings on rock is not desirable, then ground floor slabs or basements will need to be designed as fully suspended. In the case of re-use of the clay fill material, consideration will also need to be given to the reactive potential of the soil. Void formers will need to be provided below footing beams and slabs (if they are suspended on piles) to allow for potential uplift from swelling soils. Initially we recommend void formers of at least 100mm thickness be allowed for.

All basement or ground floor slabs should also have adequate subfloor drainage to collect and appropriately dispose of any groundwater seepage. Where gravity drainage is not feasible, fail-safe sump and pump systems will need to be adopted. Basements may extend below the groundwater level and where this occurs and permanent long-term drainage of the basement is not allowed, the basements will need to be tanked structures designed for full hydrostatic uplift pressures.

6.10 Pavements

Following completion of bulk earthworks, we anticipate that the subgrade for pavements (such as access roads and car parking areas) will probably comprise clay fill material, extremely weathered latite or engineered fill. The subgrade will need to be prepared in accordance with the requirements of Section 6.7 above. We recommend that a preliminary design CBR of 3% or an estimated subgrade reaction modulus (for concrete slabs or pavements) of 25kPa/mm (750mm diameter plate) should be adopted for the clay fill material and extremely weathered latite, subject to specific site testing once the detailed design is further developed. Where imported material is used as an engineered fill below proposed pavements the design CBR of this material will need to be confirmed by additional testing to confirm suitability for the proposed pavement thickness design.

The existing high plasticity clay fill material is highly expansive, therefore in accordance with Austroads Pavement Design[2] guidelines, several techniques can be used to minimize the risk of volume changes (in response to changes in moisture content) in the subgrade beneath proposed pavement areas. These include, amongst other options:

- Provide a select fill layer above the expansive subgrade, at least 300mm thick;
- Ensure under-pavement drains do not penetrate into the expansive subgrade and that these divert all water away from the pavement subgrade;
- Provide adequate road drainage, including sealed shoulders and impermeable verge materials;
- Incorporate lime stabilisation (or similar) in the upper expansive subgrade to reduce the plasticity and potential for volume changes.

Further measures that can be implemented are detailed in Section 5.3.5 of the Austroads pavement design guidelines.

All base materials for flexible pavements or sub-base materials for rigid pavements should comprise good quality, fine crushed rock such as DGB20 (TfNSW QA Specification 3051). The base course material should be compacted using a large static smooth drum roller to at least 100% of SMDD. Adequate moisture conditioning to within 2% of Standard Optimum Moisture Content (SOMC) should be provided during placement.

Field density tests should be carried out on the unbound granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 50m length of lane or 1 per 500m^2 layer, whichever requires the most tests. At least Level 2 testing should be completed on pavement layers and the geotechnical inspection and testing authority (Level 1 GITA and Level 2 GTA) should be directly engaged by the client or their representative.

In order to protect pavements, subsoil drains should be provided beyond the perimeter of all proposed pavement areas. The drainage trenches should be excavated with a continuous longitudinal fall to appropriate discharge points to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system for disposal.

6.11 Earthquake Design Classification

Based upon AS1170.4-2007 “Structural design actions Part 4: Earthquake actions in Australia” the following parameters should be adopted for seismic design:

- Hazard Factor (Z) = 0.09;
- Sub-Soil Class C_e – Shallow soil site.

[2] Austroads Pavement Technology Series – Pavement Design. A Guide to the Structural Design of Road Pavements. Standards Australia.

7 GAP ANALYSIS

7.1 Identified Risks and Constraints

From a geotechnical perspective, we consider the site will be suitable for the scale of development proposed. A summary of some of the key geotechnical risks associated with the proposed development are outlined below. Risks associated with contamination and other environmental risks are discussed in the Detailed Site Investigation (DSI) report, prepared by JK Environments.

- If excavation for the proposed buildings encounters high to very high strength latite bedrock then this will produce hard rock excavation conditions. Excavation through such material will be slow and abrasive for excavation equipment. Specialised rock excavation equipment will be required.
- The existing clay fill material across the site is highly reactive and will exhibit high shrink-swell movements with changes in moisture content. As such, any structures will need to consider the reactive potential of these soils in any earthworks, footing and slab design.
- The existing clay fill is considered uncontrolled and not suitable for support of structural loads. Specific consideration needs to be given to earthworks operations.
- The depth and quality of the underlying bedrock for support of the structures may vary particularly at the southern end of the site, which may require deeper, piled footings.

7.2 Recommendations for Additional Investigations (Detailed Design development)

As the detailed design is further developed, and in consultation with the HI project team, additional field investigation works may be warranted, that are specifically targeted at key features or where a higher level of certainty is required. Such a requirement will be discussed between the various consultants (structural engineer, geotechnical engineer, architect, project management team) to agree a suitable scope and timing for the additional works. Additional geotechnical reporting may be required that specifically addresses the key elements of the developed detailed design.

8 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long-term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained.

If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

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

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

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

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

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

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

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

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

GROUNDWATER

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

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

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

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

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

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

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

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

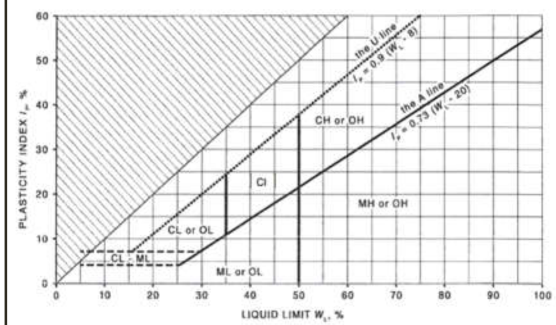
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



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LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	$w \approx$ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	$w \approx$ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	VL	VERY LOOSE	≤ 15	0 – 4
	L	LOOSE	> 15 and ≤ 35	4 – 10
	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30
	D	DENSE	> 65 and ≤ 85	30 – 50
	VD	VERY DENSE	> 85	> 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Abbreviation</th><th rowspan="2">Uniaxial Compressive Strength (MPa)</th><th colspan="2">Guide to Strength</th></tr>
<tr><th>Point Load Strength Index $I_{s(50)}$ (MPa)</th><th>Field Assessment</th></tr>
<tr><td>Very Low Strength</td><td>VL</td><td>0.6 to 2</td><td>0.03 to 0.1</td><td>Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.</td></tr>
<tr><td>Low Strength</td><td>L</td><td>2 to 6</td><td>0.1 to 0.3</td><td>Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.</td></tr>
<tr><td>Medium Strength</td><td>M</td><td>6 to 20</td><td>0.3 to 1</td><td>Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.</td></tr>
<tr><td>High Strength</td><td>H</td><td>20 to 60</td><td>1 to 3</td><td>A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.</td></tr>
<tr><td>Very High Strength</td><td>VH</td><td>60 to 200</td><td>3 to 10</td><td>Hand specimen breaks with pick after more than one blow; rock rings under hammer.</td></tr>
<tr><td>Extremely High Strength</td><td>EH</td><td>> 200</td><td>> 10</td><td>Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.</td></tr>
</table>

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

FIGURES

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Report No:	Revision No:	Figure No:
34300LX2	01	01

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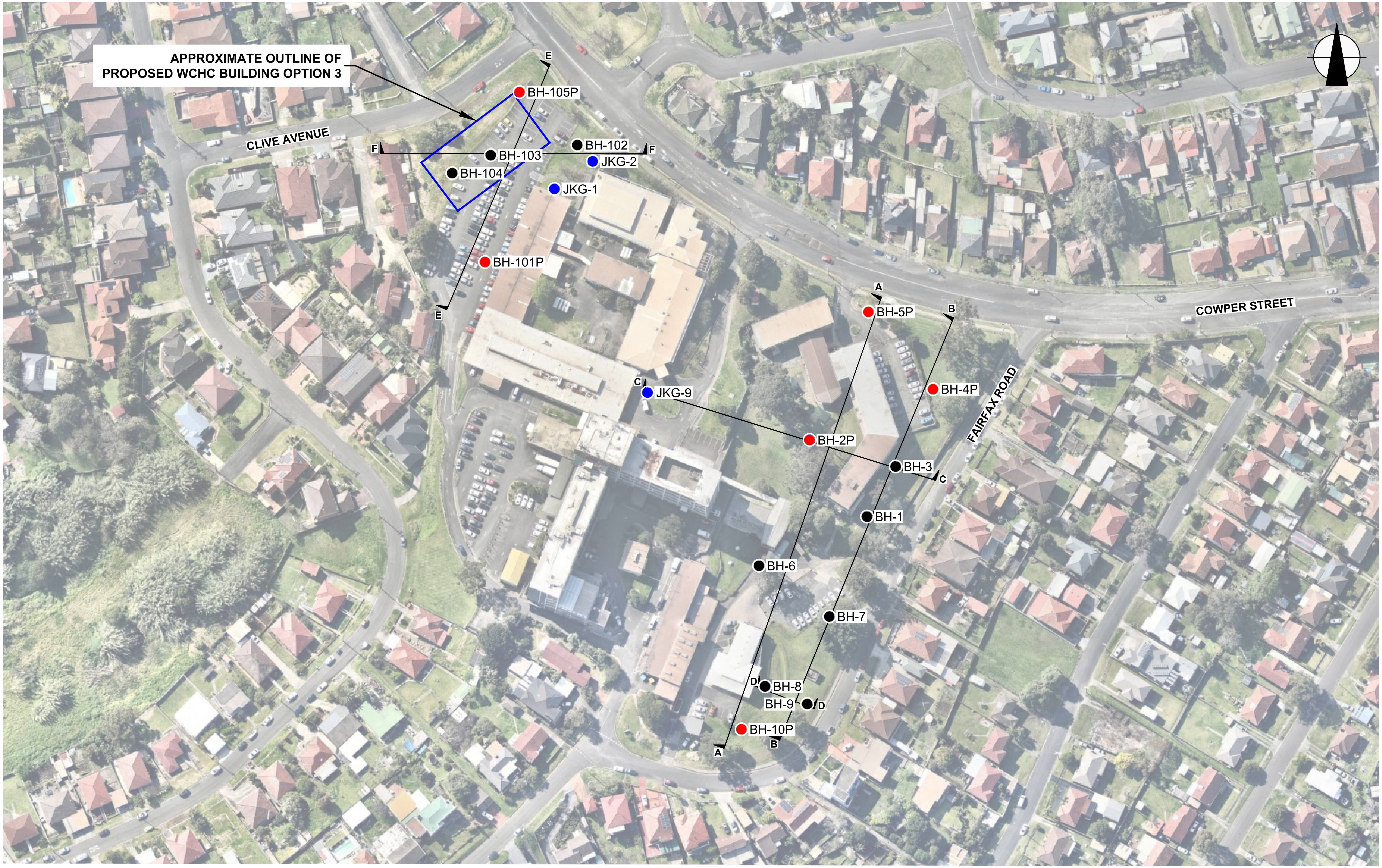
PLOT DATE: 13/03/2023 8:57:12 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34300LX DUNMORE\12. CAD\34300LX2\34300LX2.DWG



LEGEND

- BOREHOLE (CORED)
- BOREHOLE WITH STANDPIPE PIEZOMETER
- JKG BOREHOLE (1986)

NOTES:
1. REFER TO FIGURE 3 FOR CROSS SECTION A-A.
2. REFER TO FIGURE 4 FOR CROSS SECTION B-B.
3. REFER TO FIGURE 5 FOR CROSS SECTION C-C AND D-D.
4. REFER TO FIGURE 6 FOR CROSS SECTION E-E AND F-F.

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 15 30 45 60 75

SCALE 1:1500 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	BOREHOLE LOCATION PLAN
Location:	PORT KEMBLA HOSPITAL, WARRAWONG, NSW
Report No: 34300LX2	Revision No: 01 / Figure No: 02

JKGeotechnics

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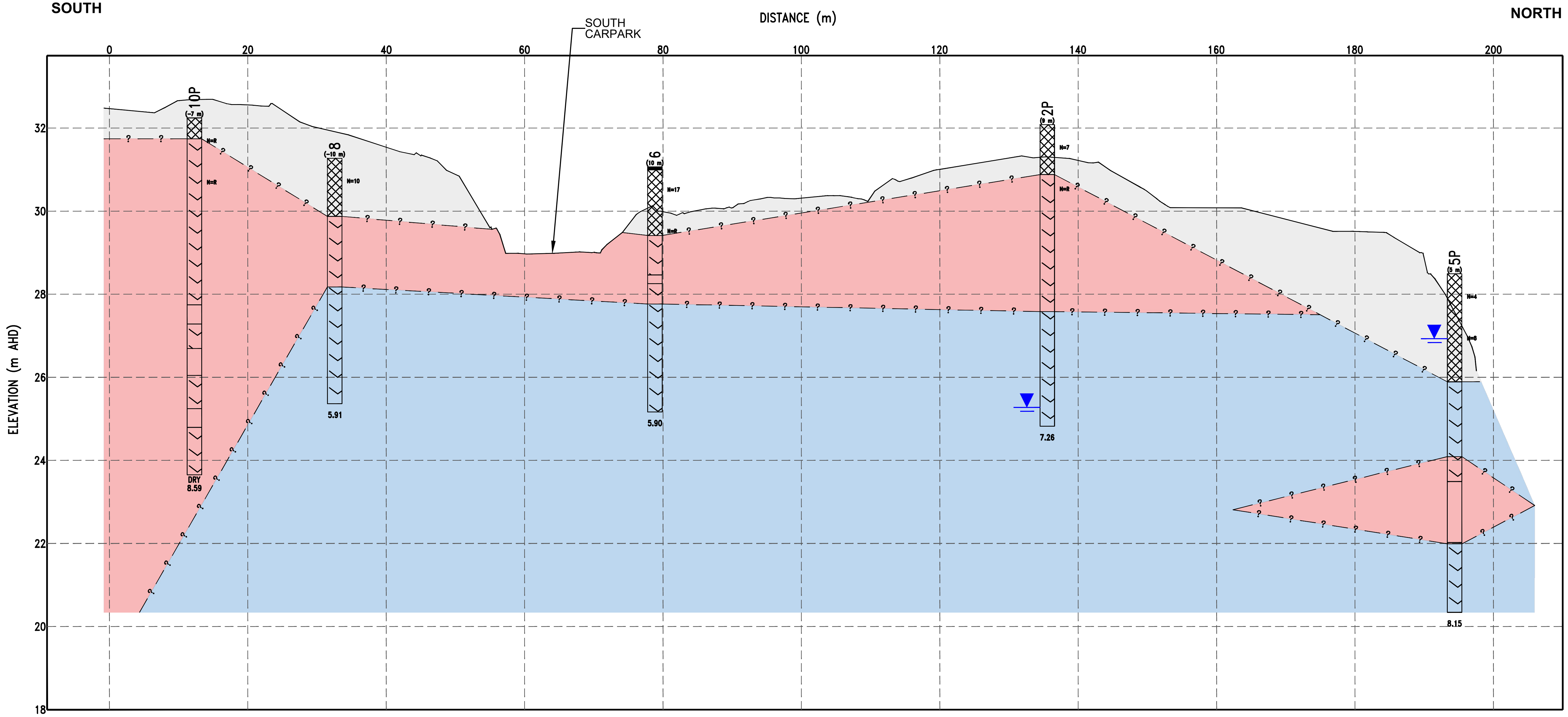


MATERIAL GRAPHIC

FILL

NO CORE

LATTITE (UPPER)

LATTITE (LOWER)

STANDING WATER LEVEL (IN PIEZOMETER)

0	6	12	18	24	30
H: SCALE		1:600 @A3			METRES
0	1	2	3	4	5
V: SCALE		1:100 @A3			METRES

This plan should be read in conjunction with the JK Geotechnics report.

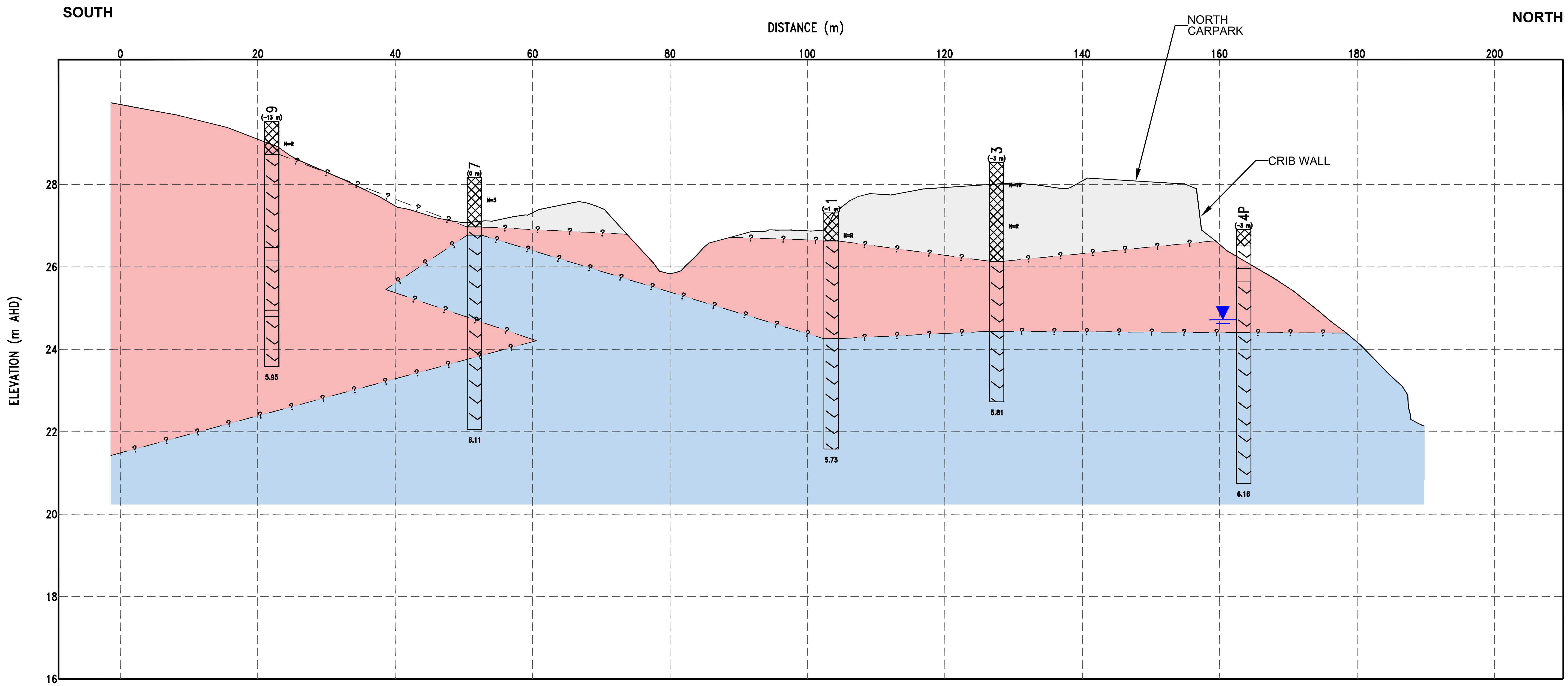
Title: **INFERRED GROUND PROFILE SECTION A-A**

Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Report No: 34300LX2 | Revision No: 01 | Figure No: 03

JKGeotechnics

PLOT DATE: 13/03/2023 8:59:53 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34300LX DUNMORE\12. CAD\34300LX2\34300LX2.DWG



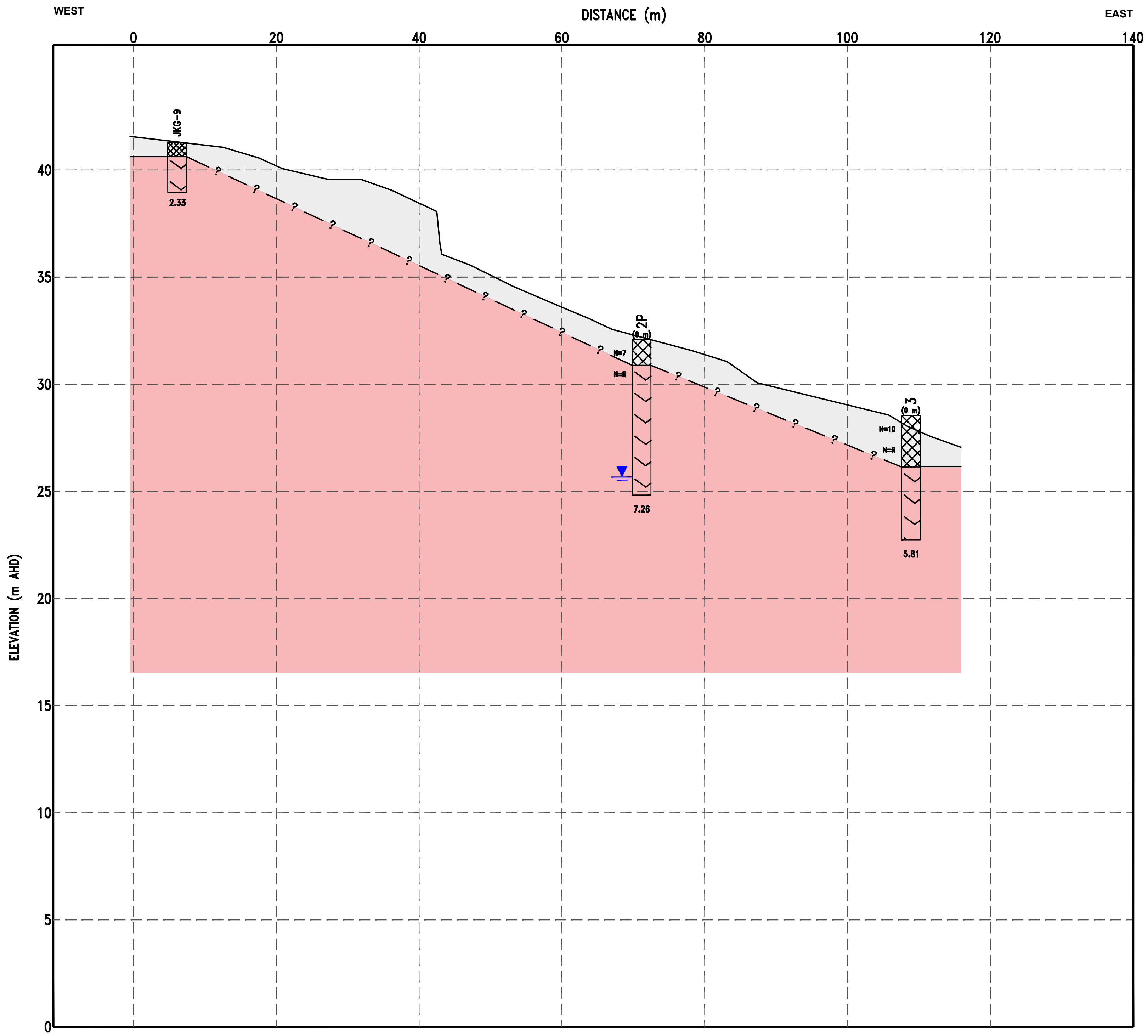
MATERIAL GRAPHIC

- FILL
- LATTITE (UPPER)
- NO CORE
- LATTITE (LOWER)
- STANDING WATER LEVEL (IN PIEZOMETER)

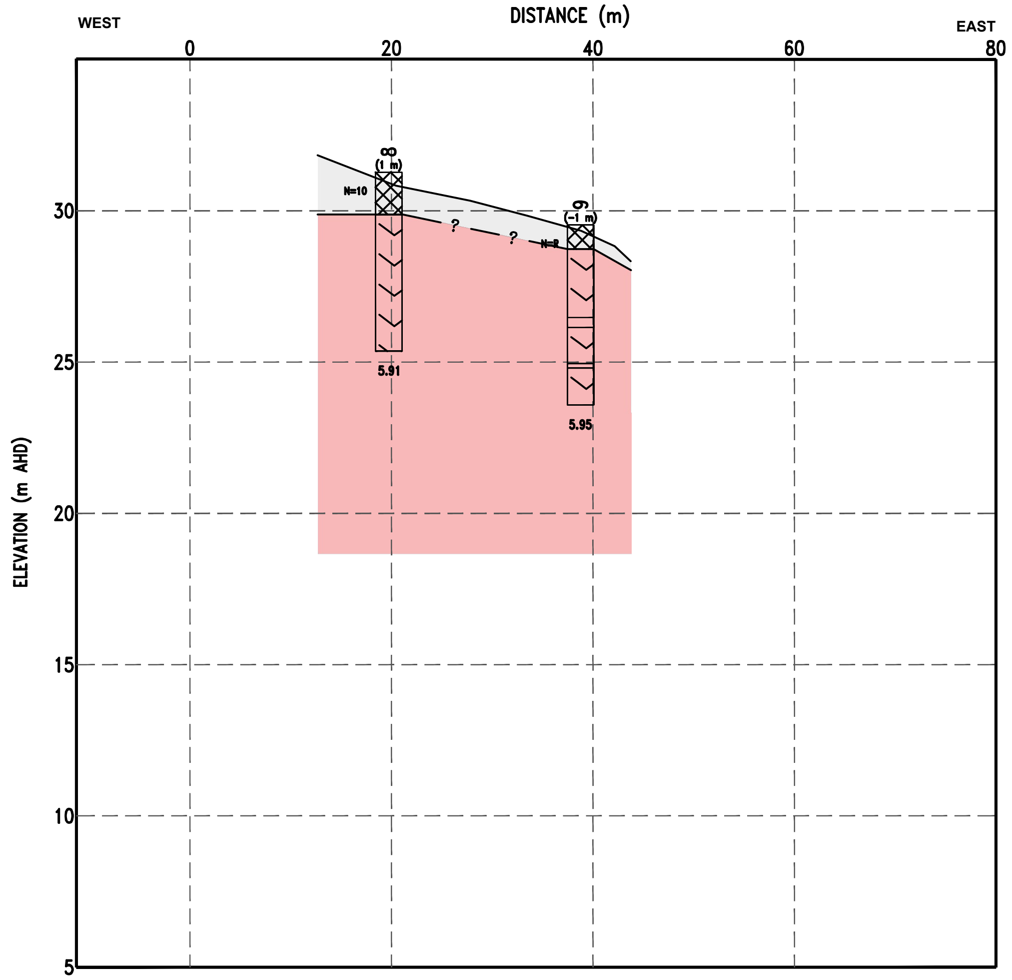
H: SCALE 1:600 @A3 (0 6 12 18 24 30 METRES)	Title: INFERRED GROUND PROFILE SECTION B-B
V: SCALE 1:100 @A3 (0 1 2 3 4 5 METRES)	Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW
This plan should be read in conjunction with the JK Geotechnics report.	Report No: 34300LX2 — Revision No: 01 — Figure No: 04
	JKGeotechnics

PLOT DATE: 13/03/2023 9:00:16 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34300LX DUNMORE\12. CAD\34300LX2\34300LX2.DWG

SECTION C-C



SECTION D-D



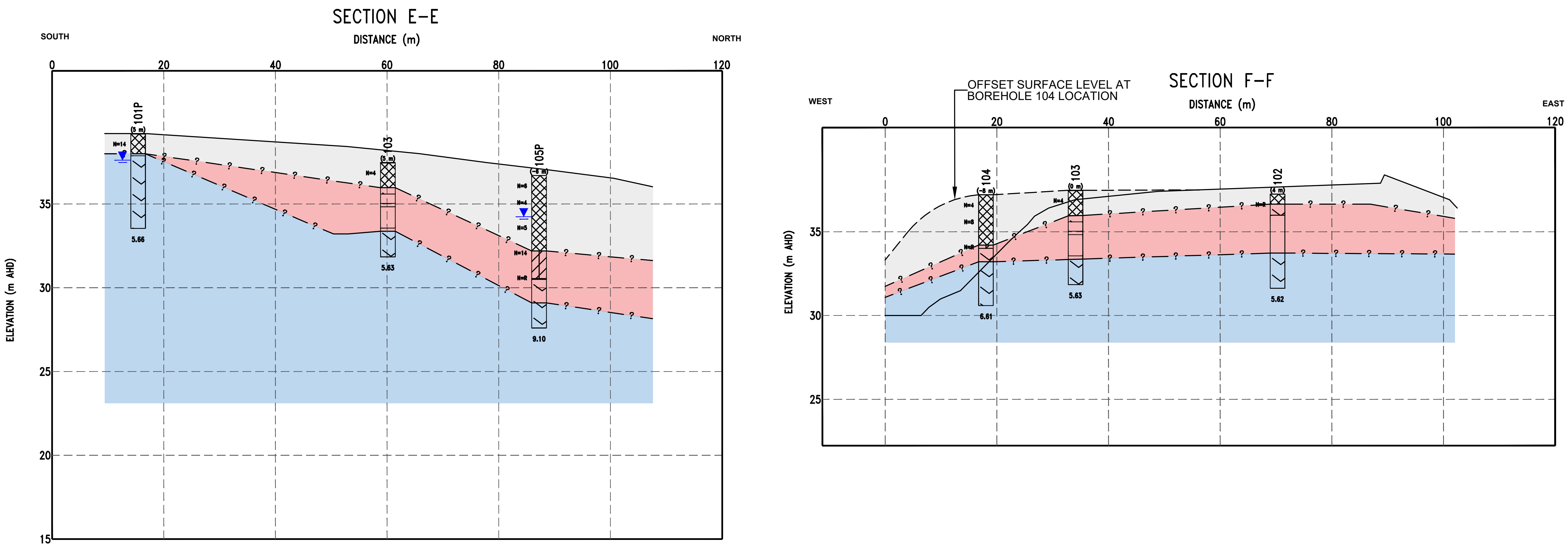
MATERIAL GRAPHIC

- FILL
- LATTITE (UPPER)
- NO CORE
- LATTITE (LOWER)
- STANDING WATER LEVEL (IN PIEZOMETER)

H:SCALE 1:750 @A3 METRES (0, 7.5, 15, 22.5, 30, 37.5)	Title: INFERRED GROUND PROFILE SECTION C-C AND SECTION D-D
V:SCALE 1:250 @A3 METRES (0, 2.5, 5, 7.5, 10, 12.5)	Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW
This plan should be read in conjunction with the JK Geotechnics report.	Report No: 34300LX2 · Revision No: 01 · Figure No: 05

JKGeotechnics

PLOT DATE: 13/03/2023 9:00:43 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34300LX DUNMORE\12. CAD\34300LX\34300LX2.DWG



MATERIAL GRAPHIC

FILL
NO CORE
LATTITE (UPPER) - INCLUDING RESIDUAL CLAY
LATTITE (LOWER)
STANDING WATER LEVEL (IN PIEZOMETER)

H: SCALE 1:750 @A3 METRES (0, 7.5, 15, 22.5, 30, 37.5)

V: SCALE 1:250 @A3 METRES (0, 2.5, 5, 7.5, 10, 12.5)

This plan should be read in conjunction with the JK Geotechnics report.

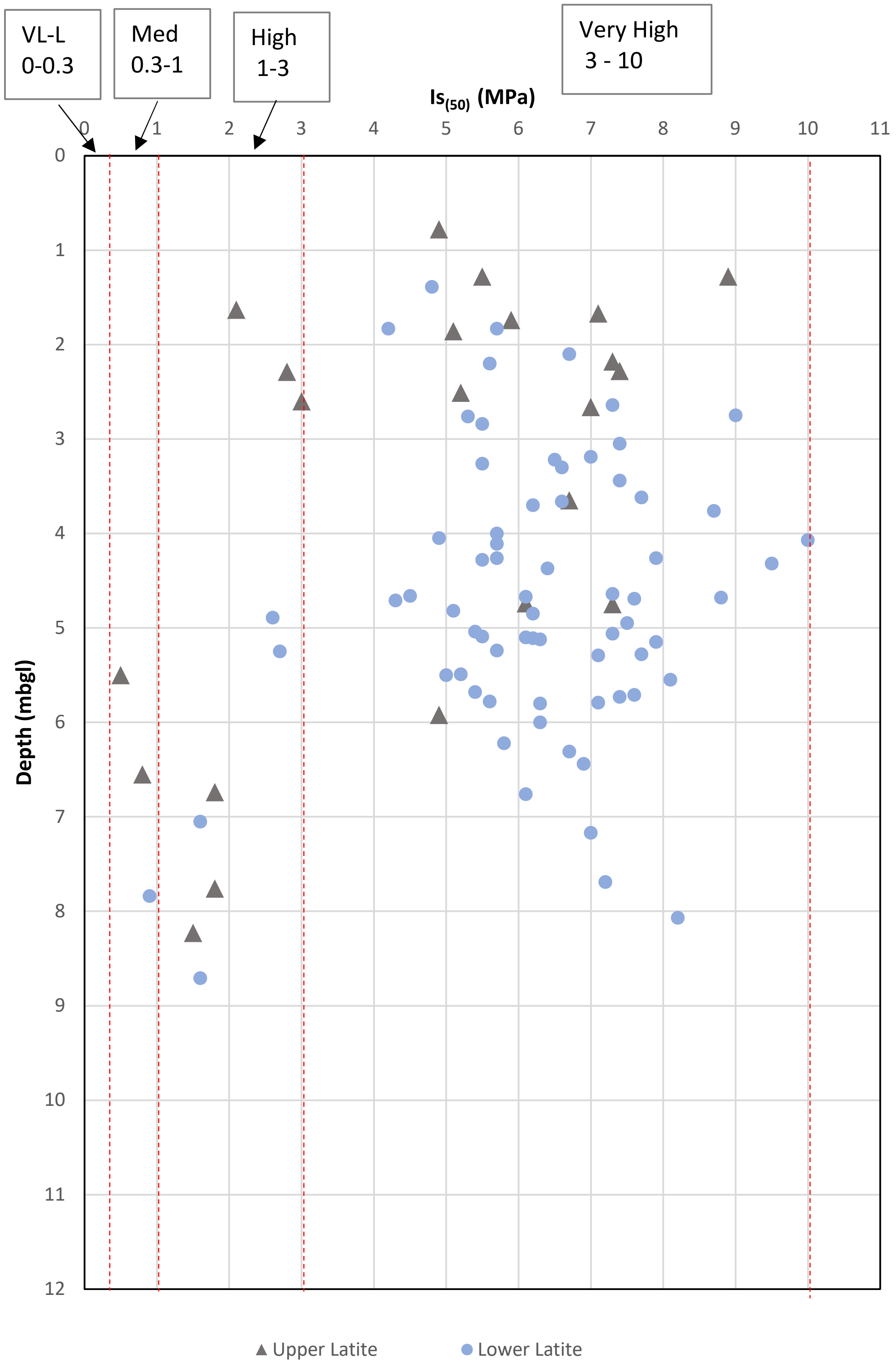
Title: **INFERRED GROUND PROFILE SECTION E-E AND SECTION F-F**

Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Report No: 34300LX2

Figure No: 06

JKGeotechnics



Strength classification as per AS 1726:2017 shown on top border

Drawing Title	FIGURE 07		
Point Load Index Results vs Depth (WCHC Site)	Scale at A4: NTS		
	Drawn QV	Approved RR	Date 15/03/2023
	Revision 1		

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APPENDIX A

Borehole Logs and Core Box Photos

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BOREHOLE LOG

Borehole No.
1
1 / 2

EASTING: 305507.264
NORTHING: 6181866.095

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 27.31 m
Date: 1/11/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					N=SPT 7/ 100mm REFUSAL	27				FILL: Silty clay, low to medium plasticity, dark grey and dark brown, with fine to medium grained sand, and fine to medium grained igneous and ironstone gravel, trace of root fibres.	w>PL			GRASS COVER MODERATE 'TC' BIT RESISTANCE
						26	1			REFER TO CORED BOREHOLE LOG				
						25	2							
						24	3							
						23	4							
						22	5							
						21	6							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34300LX2 WARRAWONG.GPJ <<DrawingFile>> 21/11/2022 11:07 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CORED BOREHOLE LOG

EASTING: 305507.264
NORTHING: 6181866.095

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 27.31 m
Date: 1/11/22 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK309 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		27			START CORING AT 0.68m					
90% RETURN		26	1		LATITE: fine to medium grained, dark grey speckled light grey and red brown. Non-intact from 0.68m to 1.65m, from 1.95m to 2.70m. Joint spacing generally >0.3m below 2.70m.	SW	VH	5.9	(0.76m) J, 60°, P, R, Clay FILLED (0.89m) J, 90°, C, R, Fe Sn (1.05m) J, 40°, C, R, Clay FILLED (1.15m) J, 60°, C, R, Fe Ct (1.34m) J, 40°, P, R, Fe Ct (1.42m) J, 40°, P, R, Fe Ct (1.57m) J, 40°, P, R, Fe Ct (1.76m) J, 90°, P, R, Fe Ct (1.95m) J, 90°, P, R, Fe Ct (2.20m) J, 90°, P, R, Fe Ct (2.58m) J, 90°, P, R, Fe Ct	Dapto Latite
100% RETURN		25, 24, 23, 22	2, 3, 4, 5					5.5 7.4 7.7 5.7 7.3	(2.68m) Cr, 0°, 35 mm.t (2.78m) J, 90°, C, R, Fe Sn (3.13m) J, 70°, C, R, Fe Sn (3.20m) Jh, 80°, Un, R, Fe Sn (3.40m) J, 10°, P, R, Fe Sn (4.95m) J, 70°, P, R, Ca FILLED, 2 mm.t (5.34m) J, 80°, C, R, Fe Ct (5.46m) J, 80°, C, R, Ca FILLED, 2 mm.t (5.60m) J, 70°, C, R, Cn	
		21	6		END OF BOREHOLE AT 5.73 m					

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH. Spacing (mm): 600, 200, 60, 20.

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	15/11/2022		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH01	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 2

EASTING: 305483.227
NORTHING: 6181898.159

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 32.08 m
Date: 2/11/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	N = 7 1,3,4	32 31	0 1		FILL: Silty clay, medium plasticity, dark brown, brown and dark grey, with fine to medium grained sand, trace of fine to coarse grained igneous and latite gravel, ash and root fibres.	w>PL		150 170 150	GRASS COVER APPEARS POORLY TO MODERATELY COMPACTED
	N=SPT 8/ 100mm REFUSAL			-	Extremely Weathered latite: sandy CLAY, low plasticity, brown and dark grey, fine to medium grained sand, trace of root fibres.	XW	Hd		DAPTO LATITE
		30 29	2 3		LATITE: fine grained, dark grey brown, light grey and red brown, with occasional extremely weathered bands.	HW	VL - L		VERY LOW 'TC' BIT RESISTANCE
			4		as above, but with occasional low to medium strength bands.		L		LOW RESISTANCE
		28 27 26	5 6		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 7.17m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.0m TO 7.17m. CASING 3.0m TO 0.05m. 2mm SAND FILTER PACK 3.0m TO 7.17m. BENTONITE SEAL 2.5m TO 3.0m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 2

EASTING: 305483.227
NORTHING: 6181898.159

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2	**Core Size:** NMLC	**R.L. Surface:** 32.08 m
Date: 2/11/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK309	**Bearing:** N/A	**Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific	General	Formation
		29 – 28	– 4		START CORING AT 4.05m							
100% RETURN		28 – 25	4.05 – 7.26		Extremely Weathered latite: sandy CLAY, low plasticity, dark brown, dark grey and yellow brown, fine to medium grained sand, with occasional very low strength bands.	XW	Hd					Dapto Latite
					LATITE: fine to medium grained, dark grey speckled light gey and red brown. Joint spacing generally <0.2m.	SW	H - VH	7.3, 2.7, 6.7, 1.6	600 200 60 20	(4.52m) J, 90°, C, R, Fe Ct (4.77m) J, 25°, P, R, Fe Ct (4.95m) Cr, 0°, 15 mm.t (5.03m) Jh, 40°, P, R, Fe Ct (5.20m) J, 40°, C, R, Cn (5.36m) J, 45°, C, R, Fe Sn (5.45m) J, 80°, C, R, Fe Ct (5.65m) J, 90°, C, R, Fe Ct (5.79m) J, 30°, C, R, Fe Ct (5.91m) J, 25°, C, R, Fe Ct (5.98m) J, 80°, C, R, Fe Ct (6.15m) J, 90°, C, R, Fe Ct (6.50m) J, 90°, C, R, Fe Ct (6.68m) J, 90°, C, R, Fe Ct (6.86m) J, 40°, C, R, Clay FILLED (7.00m) J x 2, 70°, P, R, Fe Ct (7.18m) J, 70°, P, R, Fe Ct		
		25 – 23	7.26 – 9		END OF BOREHOLE AT 7.26 m				600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV	Client: Health Infrastructure NSW	
Approved:	RR	Project: Proposed WCHC Building – Port Kembla	
Date:	15/11/2022	Title: **ROCK CORE PHOTOGRAPH**	
Scale:	NTS		
Original Size:	A4	Project No: 34300LX2	BH02P	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
3
1 / 2

EASTING: 305519.247
NORTHING: 6181887.046

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 28.54 m
Date: 1/11/02 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				0			FILL: Silty clay, low to medium plasticity, dark brown, brown and dark grey, with fine to medium grained sand, and fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.	w~PL			GRASS COVER APPEARS WELL COMPACTED
		N = 10 2,5,5	28							210 200 250	
				1			as above, but brown and dark brown, trace of fine to coarse grained latite gravel.				
		N > 9 16,9/ 50mm REFUSAL	27								
				2							
			26			-	LATITE: fine grained, grey, dark grey, green grey and red brown.	HW	L - M		DAPTO LATITE
							REFER TO CORED BOREHOLE LOG				LOW TO MODERATE 'TC' BIT RESISTANCE
				3							
			25								
				4							
			24								
				5							
			23								
				6							
			22								

CORED BOREHOLE LOG

Borehole No. 3

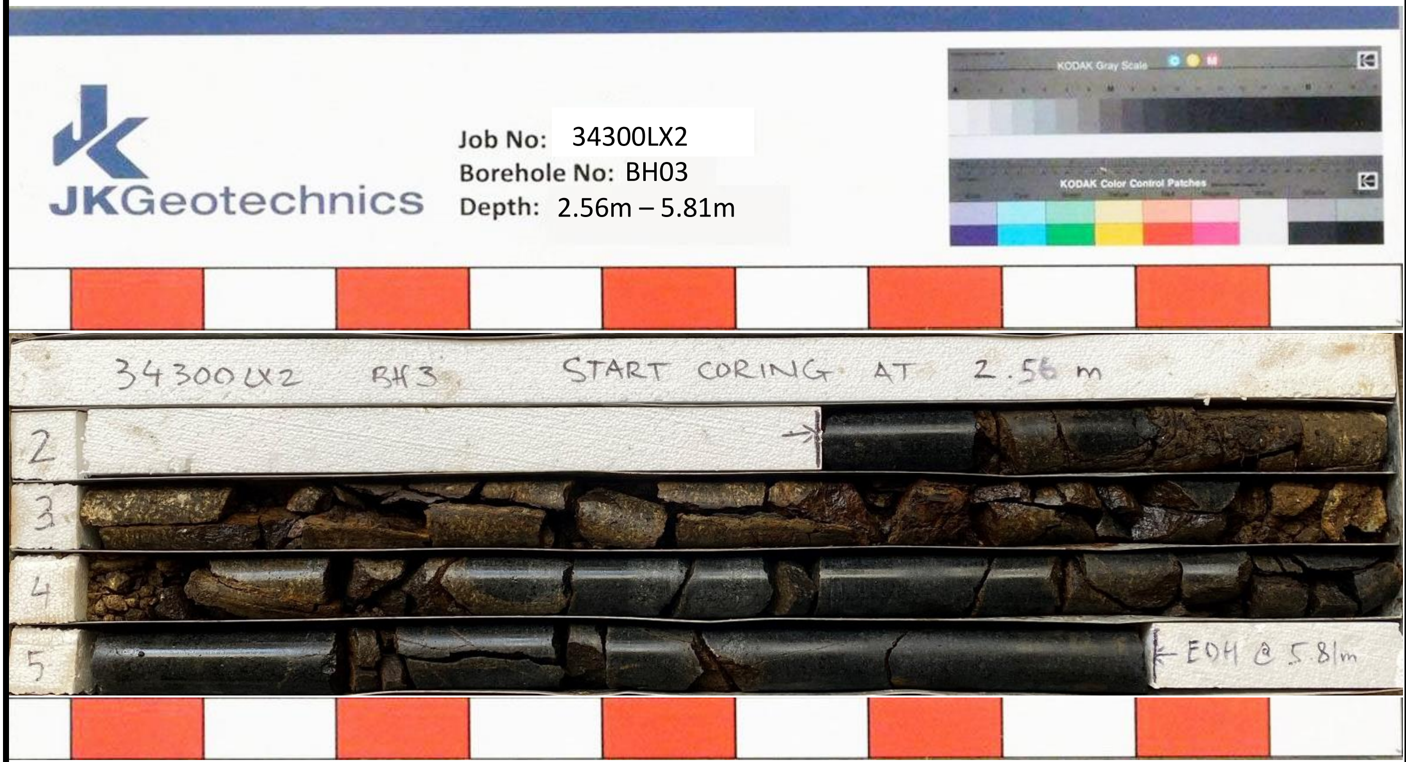
EASTING: 305519.247
NORTHING: 6181887.046

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 28.54 m
Date: 1/11/02 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK309 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	SPACING (mm)	DEFECT DETAILS – DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
		26			START CORING AT 2.56m						
80% RETURN			3		LATITE: fine grained, dark grey speckled light grey and red brown. Non-intact from 2.70m to 3.86m.	MW	VH	3.0		(2.70m) XWS, 0°, 25 mm.t (2.85m) J x 2, 70°, C, R, Fe Ct (3.30m) J, 90°, C, R, Fe Ct (3.74m) J, 90°, C, R, Fe Sn	Dapto Latite
		25	4		Extremely Weathered latite: sandy CLAY, low plasticity, dark brown, grey and light grey, with occasional low strength bands.	XW	Hd				
100% RETURN		24			LATITE: fine grained, dark grey speckled light grey and red brown. Joint spacing generally >0.3m.	SW	VH	6.4 8.8 6.1 7.6		(4.20m) J, 0°, C, R, Fe Sn (4.28m) J, 60°, C, R, Fe Sn (4.90m) J, 90°, C, R, Fe Ct (5.29m) J, 90°, C, R, Fe Sn	
		23	5								
		22	6		END OF BOREHOLE AT 5.81 m						
		21	7								
		20	8								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Drawn:	QV		Client: Health Infrastructure NSW
Approved:	RR		Project: Proposed WCHC Building – Port Kembla
Date:	15/11/2022		Title: **ROCK CORE PHOTOGRAPH**
Scale:	NTS		
Original Size:	A4		Project No: 34300LX2 | BH03 | Rev:

JKGeotechnics

BOREHOLE LOG

EASTING: 305534.879
NORTHING: 6181919.422

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 26.91 m
Date: 27/9/22 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING										FILL: Silty clay, medium to high plasticity, dark grey, with fine to medium grained sandstone and latite gravel, trace of root fibres.	w>PL			GRASS COVER
									-	LATITE: fine to medium grained, light grey, dark grey, green grey and orange brown.	MW	M		DAPTO LATITE MODERATE 'TC' BIT RESISTANCE HIGH RESISTANCE
						26	1			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 6.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.0m TO 6.1m. CASING 0.05m TO 1.0m. 2mm SAND FILTER PACK 1.2m TO 6.1m. BENTONITE SEAL 0.8m TO 1.2m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						25	2							
						24	3							
						23	4							
						22	5							
						21	6							
						20								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34300LX2 WARRAWONG.GPJ <<DrawingFile>> 21/11/2022 11:10 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
4
2 / 2

EASTING: 305534.879
NORTHING: 6181919.422

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2	**Core Size:** NMLC	**R.L. Surface:** 26.91 m
Date: 27/9/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
		26			START CORING AT 0.94m						
			1		NO CORE 0.33m						
70% RETURN		25	2		LATITE: fine grained, dark grey, speckled light grey and green grey. Joint spacing generally >0.6m below 2.5m.	SW	VH	5.5 7.1 7.4		(1.39m) J, 5°, P, S, Clay FILLED (1.45m) Cr, 0°, 75 mm.t (1.55m) Cr, 10°, 15 mm.t (1.61m) J, 25°, P, R, Cn (1.71m) J, 80°, C, R, Fe Ct (1.81m) Jh, 80°, C, R, Fe Ct (1.86m) Jx2, 50°, C, R, Fe Ct (2.13m) XWS, 10°, 25 mm.t (2.15m) J, 25°, P, R, Clay FILLED (2.23m) J, 30°, Un, R, Cn (2.41m) J, 80°, Un, R, Fe Sn	Dapto Latite
100% RETURN		24 23 22 21	3 4 5 6			FR		9.0 6.5 8.7 7.9 4.3 7.7 7.1		(2.50m) J, 60°, P, R, Cn (2.62m) J, 70°, C, R, Cn (2.86m) J, 5°, P, R, Cn (3.89m) J, 80°, C, R, Cn (4.51m) J, 25°, P, R, Clay FILLED (5.40m) J, 0°, P, R, Cn (5.44m) Jx2, 25°, Un, R, Cn (5.56m) Jh, 70°, C, R, Cn (6.05m) Jx2, 60°, P, R, Cn	
		20			END OF BOREHOLE AT 6.16 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV	Client: Health Infrastructure NSW
Approved:	RR	Project: Proposed WCHC Building – Port Kembla
Date:	27/09/2022	Title: **ROCK CORE PHOTOGRAPH**
Scale:	NTS	
Original Size:	A4	Project No: 34300LX2 / BH04P / Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
5
1 / 2

EASTING: 305507.874
NORTHING: 6181951.763

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 28.49 m
Date: 27/9/22 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	N = 4 2,2,2	28	0 – 1		FILL: Silty clay, medium plasticity, dark grey and dark brown, with fine to medium grained sand and fine to medium grained igneous and sandstone gravel, trace of brick fragments, ash and root fibres.	w~PL			GRASS COVER TOO FRIABLE FOR HP TESTING
	N = 8 2,3,5	27, 26	1 – 2		FILL: Silty clay, medium to high plasticity, dark brown, brown and dark grey, with fine to medium grained sand and fine to medium grained sandstone gravel, trace of root fibres.	w>PL		210 210 220	
		25	3	-	LATITE: fine to medium grained, dark grey, light grey, green grey and red brown, with occasional clay bands.	HW	L - M		DAPTO LATITE LOW TO MODERATE 'TC' BIT RESISTANCE HIGH RESISTANCE
		24, 23, 22	4, 5, 6		REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 8.15m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3m TO 8.15m. CASING 0.05m TO 3m. 2mm SAND FILTER PACK 3.2m TO 8.15m. BENTONITE SEAL 2.5m TO 3.2m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

SAMPLES: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 5

2 / 2

EASTING: 305507.874
NORTHING: 6181951.763

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2	**Core Size:** NMLC	**R.L. Surface:** 28.49 m
Date: 27/9/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		25		START CORING AT 3.60m					
70% RETURN		24	4, 5	LATITE: fine grained, dark grey, light grey and green grey. Non-intact below 4.4m.	SW	VH	5.7, 6.1	(3.64m) J, 35°, Un, R, Cn (3.83m) J, 40°, P, R, Cn (3.93m) Jhx2, 90°, C, R, Cn (4.11m) J, 90°, C, R, Cn (4.41m) XWS, 20°, 20 mm.t (4.50m) J, 70°, C, R, Fe Ct (4.67m) J, 80°, C, R, Cn (4.90m) J, 90°, C, R, Cn	Dapto Latite
70% RETURN		23	6	NO CORE 1.47m					
100% RETURN		22, 21	7, 8	LATITE: fine grained, dark grey, speckled light grey and green grey.	FR	VH	6.1, 7.0, 7.2, 8.2	(6.67m) J, 20°, P, R, Cn (6.87m) J, 0°, P, R, Fe Ct (6.91m) J, 5°, P, R, Fe Ct (6.95m) J, 20°, P, R, Cn (7.07m) J, 5°, P, R, Fe Sn (7.39m) J, 5°, P, R, Fe Sn	Dapto Latite
		20, 19	9	END OF BOREHOLE AT 8.15 m					

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW
Approved:	RR		Project: Proposed WCHC Building – Port Kembla
Date:	27/09/2022		Title: **ROCK CORE PHOTOGRAPH**
Scale:	NTS		
Original Size:	A4		Project No: 34300LX2 | BH05P | Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No. 6

EASTING: 305462.135
NORTHING: 6181845.521

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 31.07 m
Date: 31/10/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		31	0		ASPHALTIC CONCRETE: 70mm.t				
	N = 17 8,8,9	30	1	-	FILL: Silty clay, low to medium plasticity, dark brown and brown, with fine to medium grained sand, and fine to medium grained igneous and sandstone gravel, trace of ash and root fibres.	w<PL			APPEARS WELL COMPACTED TOO FRIABLE FOR HP TESTING
	N=SPT 12/ 150mm REFUSAL	29	2	-	LATITE: fine to medium grained, dark grey, light grey and red brown.	MW	M - H H		DAPTO LATITE MODERATE TO HIGH 'TC' BIT RESISTANCE HIGH RESISTANCE
		28	3		REFER TO CORED BOREHOLE LOG				
		27	4						
		26	5						
		25	6						

Samples: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. **6**

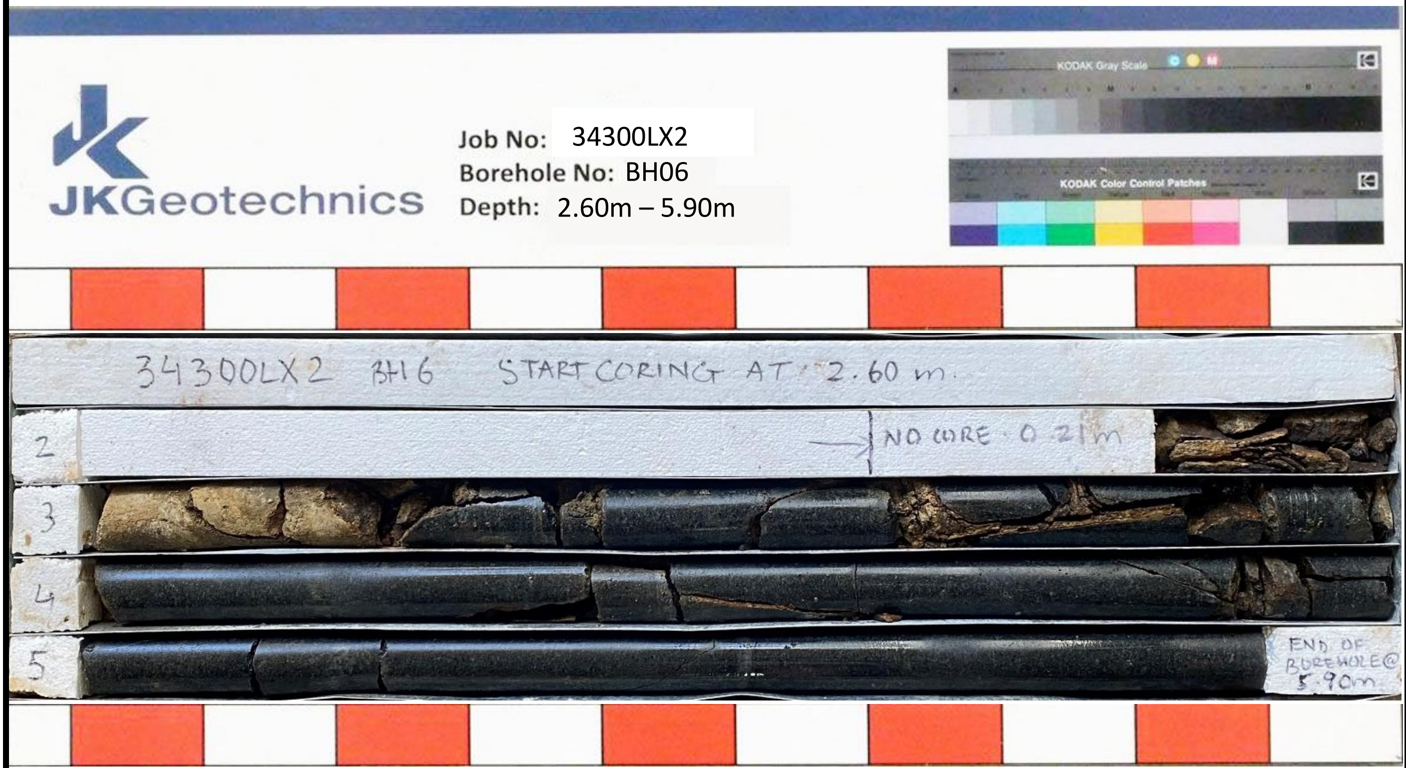
EASTING: 305462.135
NORTHING: 6181845.521

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 | **Core Size:** NMLC | **R.L. Surface:** 31.07 m
Date: 31/10/22 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK309 | **Bearing:** N/A | **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) 600 200 60 20	DEFECT DETAILS DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
		29			START CORING AT 2.60m						
100% RETURN					NO CORE 0.21m						
		28	3		LATITE: fine to medium grained, dark brown to yellow brown, non-intact from 2.81m to 3.0m.	HW	L			(2.89m) J, 90°, C, R, Fe Ct (3.22m) J, 50°, C, R, Fe Ct (3.29m) J, 80°, C, R, Fe Ct	
		27	4		LATITE: fine to medium grained, dark grey speckled light grey and red brown Joint spacing generally >0.6m below 4.0m.	SW	VH	7.4 10 7.6 7.1 6.3		(3.35m) J, 90°, C, R, Clay FILLED (3.62m) XWS, 0°, 28 mm.t (3.74m) J, 80°, P, R, Fe Ct (3.98m) J, 30°, C, R, Fe Sn (4.46m) J, 90°, C, R, Fe Ct (4.83m) J, 70°, C, R, Fe Ct	Dapto Latite
		26	5								
		25	6		END OF BOREHOLE AT 5.90 m						
		24	7								
		23	8								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	15/11/2022		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH06	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
7

EASTING: 305483.652
NORTHING: 6181819.833

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 28.17 m
Date: 3/11/22 **Datum:** AHD
Plant Type: JK309 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					N = 3 1,1,2	28 27	1			FILL: Silty clay, medium plasticity, dark brown and dark grey, with fine to medium grained sand, and fine to medium grained igneous and ironstone gravel, trace of ash and root fibres.	w>PL		160 160 150	GRASS COVER APPEARS POORLY COMPACTED
									-	LATITE: fine to medium grained, dark grey, light grey and red brown.	MW	M - H		DAPTO LATITE
						26 25 24 23 22	2 3 4 5 6			REFER TO CORED BOREHOLE LOG				MODERATE TO HIGH 'TC' BIT RESISTANCE

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 7

2 / 2

EASTING: 305483.652
NORTHING: 6181819.833

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2	**Core Size:** NMLC	**R.L. Surface:** 28.17 m
Date: 3/11/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK309	**Bearing:** N/A	**Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		27			START CORING AT 1.40m						
80% RETURN		26	2		LATITE: fine to medium grained, dark grey speckled light grey, red brown and green grey. Non-intact from 3.87m to 4.93m. Joint spacing generally >0.3m above 3.87m and below 4.93m. Sub-vertical infilled seams, rough, clay and extremely weathered latite filled up to 20mm.t between 3.87m and 4.93m.	SW	VH	5.7, 6.7, 7.3		(1.50m) J, 90°, C, R, Fe Ct (2.27m) J, 25°, P, R, Fe Ct (2.37m) J, 70°, C, R, Fe Ct (2.59m) J, 30°, C, R, Fe Ct (2.78m) XWS, 10°, 70 mm.t (2.92m) J, 20°, P, R, Clay FILLED	Dapto Latite
60% RETURN		25	3			MW		6.6		(3.17m) J, 40°, C, R, Fe Sn (3.23m) J, 20°, P, R, Fe Sn (3.76m) J, 90°, C, R, Fe Ct (3.81m) J, 90°, C, R, Fe Ct	
		24	4							(4.40m) Infilled Seam, 90°, C, R, Clay & XW LATITE FILLED, 20 mm.t	
		23	5			SW		7.5, 5.5, 7.4, 6.3		(5.49m) J, 90°, C, R, Fe Sn (5.62m) J, 45°, P, R, Fe Sn	
		22	6		END OF BOREHOLE AT 6.11 m						
		21	7								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV	Client: Health Infrastructure NSW	
Approved:	RR	Project: Proposed WCHC Building – Port Kembla	
Date:	15/11/2022	Title: **ROCK CORE PHOTOGRAPH**	
Scale:	NTS		
Original Size:	A4	Project No: 34300LX2	BH07	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
8
1 / 2

EASTING: 305464.625
NORTHING: 6181795.055

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 31.27 m
Date: 29/9/22 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		N = 10 2,9,1	31	0 – 1.4			FILL: Silty clay, medium plasticity, dark grey, with fine to coarse grained sandstone and latite gravel, trace of fine to medium grained sand and root fibres.	w>PL		210 180 180	GRASS COVER
			30	1.4 – 1.6		-	LATITE: fine to medium grained, dark grey, light grey, green grey and red brown.	MW	M - H		DAPTO LATITE MODERATE TO HIGH 'TC' BIT RESISTANCE 'TC' BIT REFUSAL
							REFER TO CORED BOREHOLE LOG				

RL scale: 31, 30, 29, 28, 27, 26, 25
Depth scale: 1, 2, 3, 4, 5, 6

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 8

EASTING: 305464.625
NORTHING: 6181795.055

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2	**Core Size:** NMLC	**R.L. Surface:** 31.27 m
Date: 29/9/22	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK308	**Bearing:** N/A	**Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS – SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		30			START CORING AT 1.60m						
80% RETURN		29	2		LATITE: fine grained, dark grey, speckled light grey, green grey and red brown. Non-intact from 1.9m to 3.1m. Joint spacing generally 0.3m to 0.6m below 3.1m.	SW	VH	5.1, 7.3		(1.84m) J, 70°, Un, R, Fe Ct (2.07m) J, 90°, C, R, Fe Ct (2.22-2.80m) J, 90°, C, R, Fe Ct (2.82m) Cr, 0°, 40 mm.t (2.92m) J, 80°, C, R, Fe Sn	Dapto Latite
		28	3					7.0, 6.6		(3.10m) J, 40°, P, R, Clay FILLED (3.40m) Cr, 20°, 20 mm.t (3.90m) J, 90°, C, R, Fe Ct	
90% RETURN		27	4					5.5		(4.12m) J, 90°, C, R, Fe Ct (4.46m) J, 90°, C, R, Fe Sn (4.59m) J, 20°, P, R, Fe Ct (4.74m) J, 70°, C, R, Clay FILLED	
		26	5					6.2, 5.4, 8.1		(5.01m) J, 80°, C, R, Fe Sn (5.15m) J, 90°, C, R, Fe Sn (5.43m) J, 75°, C, R, Fe Sn (5.75m) J, 90°, C, R, Fe Ct	
		25	6		END OF BOREHOLE AT 5.91 m						
		24	7								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	29/09/2022		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH08	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
9
1 / 2

EASTING: 305482.265
NORTHING: 6181787.714

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 29.53 m
Date: 26/9/22 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					N > 6 2,6/ 150mm REFUSAL	29				FILL: Silty clay, medium plasticity, dark grey, with fine to medium grained sand and fine to medium grained sandstone and igneous gravel, trace of root fibres.	w>PL		220 220 250	GRASS COVER
							1		-	LATITE: fine to medium grained, dark grey, light grey, green grey and red brown.	MW	M		DAPTO LATITE MODERATE 'TC' BIT RESISTANCE
						28				REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
							2							
						27								
							3							
						26								
							4							
						25								
							5							
						24								
							6							
						23								

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 9

EASTING: 305482.265
NORTHING: 6181787.714

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 29.53 m
Date: 26/9/22 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	Formation
		29			START CORING AT 0.93m					
70% RETURN		28	1 – 2		LATITE: fine to medium grained, dark grey, speckled light grey, green grey and red brown. Non-intact between 1.6m and 2m.	HW	H - VH	8.9 2.8 7.0	(0.95m) J, 90°, C, R, Clay FILLED (1.16m) CS, 0°, 30 mm.t (1.50m) J, 80°, C, R, Fe Ct (1.58m) Cr, 0°, 90 mm.t (1.70m) Jx2, 90°, C, Fe Ct (1.84m) CS, 40°, 30 mm.t (1.90m) J, 80°, C, R, Fe Ct (2.07m) J, 90°, C, R, Fe Ct (2.18m) J, 10°, C, R, Fe Ct (2.26m) J, 90°, C, R, Fe Ct (2.37m) Cr, 0°, 90 mm.t (2.47m) J, 90°, C, R, Fe Ct (2.59m) J, 70°, C, R, Fe Ct (2.79m) J, 90°, P, R, Fe Ct	Dapto Latite
80% RETURN		27	3		Extremely Weathered latite: sandy CLAY, low plasticity, yellow brown, orange brown and dark grey, fine to medium grained sand and gravel.	XW	Hd			
					NO CORE 0.33m					
		26	4		LATITE: fine to medium grained, dark grey, orange brown, green grey and orange brown. Non-intact.	HW	L		(3.45m) Jh, 90°, C, R, Fe Ct (3.54m) Jx2, 40°, P, R, Fe Ct (3.76m) J, 70°, P, R, Fe Ct (4.12m) J, 90°, C, R, Fe Ct	Dapto Latite
		25			Extremely Weathered latite: sandy CLAY, low plasticity, orange brown, yellow brown and dark grey, fine to medium grained sand.	XW	Hd			
					NO CORE 0.15m					
		24	5		LATITE: fine to medium grained, dark grey, speckled light grey and red brown. Generally non-intact.	MW - SW	H - VH	7.3 4.9	(4.74m) Cr, 0°, 30 mm.t (4.81m) J, 50°, C, R, Fe Ct (5.10m) J, 90°, C, R, Fe Ct (5.26m) J, 10°, P, R, Fe Ct (5.36m) Jhx2, 70°, P, R, Fe Ct (5.56m) J, 90°, C, R, Fe Ct (5.80m) J, 90°, C, R, Fe Ct	Dapto Latite
		23	6		END OF BOREHOLE AT 5.95 m					

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV	Client: Health Infrastructure NSW		
Approved:	RR	Project: Proposed WCHC Building – Port Kembla		
Date:	26/09/2022	Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS			
Original Size:	A4	Project No: 34300LX2	BH09	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
10

EASTING: 305454.787
NORTHING: 6181777.116

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 32.24 m
Date: 28/9/22 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING	DS		32				FILL: Silty clay, medium plasticity, dark grey and dark brown, with fine to medium grained igneous gravel, trace of fine to medium grained sand, and root fibres.	w>PL			GRASS COVER
	DS	N > 20 2,8,12/ 50mm REFUSAL	31	1		-	Extremely Weathered latite: silty SAND, fine to medium grained, yellow brown, orange brown and light grey, trace of root fibres.	XW	Hd		DAPTO LATITE
	DS	N=SPT 8/ 150mm REFUSAL	30	2			LATITE: fine to medium grained, yellow brown, light grey and red brown.	HW	VL		VERY LOW 'TC' BIT RESISTANCE
									L - M		LOW TO MODERATE RESISTANCE
	DS		29	3					L		LOW RESISTANCE
	DS		28	4			LATITE: fine grained, light grey, dark grey, green grey, brown and yellow brown.	HW	L - M		LOW TO MODERATE RESISTANCE
			27, 26	5, 6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.5m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.5m TO 8.5m. CASING 0.05m TO 1.5m. 2mm SAND FILTER PACK 1.7m TO 8.5m. BENTONITE SEAL 1.0m TO 1.7m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 34300LX2 WARRAWONG.GPJ <<DrawingFile>> 21/11/2022 11:13 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
10
2 / 2

EASTING: 305454.787
NORTHING: 6181777.116

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 32.24 m
Date: 28/9/22 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ VL -0.1, L -0.3, M -1, H -3, VH -10, EH	DEFECT DETAILS: SPACING (mm) 600, 200, 60, 20	DEFECT DETAILS: DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness Specific / General	Formation
		28			START CORING AT 4.50m						
70% RETURN					NO CORE 0.46m						
		27	5		Extremely Weathered latite: sandy CLAY, low plasticity, dark brown and grey, fine to medium grained sand and gravel.	XW	Hd	0.50			Dapto Latite
90% RETURN			6		NO CORE 0.65m						
		26			LATITE: fine to medium grained, dark grey, light grey and red brown.	HW	M - H	0.80 1.8		(6.25m) J, 5°, P, R, Fe Sn (6.35m) J, 35°, P, R, Fe Sn (6.43m) Cr, 20°, 20 mm.t (6.53m) J, 20°, Un, R, Fe Sn (6.61m) J, 90°, C, R, Fe Sn (6.73m) J, 80°, C, R, Fe Sn (6.87m) J, 90°, C, R, Fe Sn	Dapto Latite
		25	7		NO CORE 0.45m						
		24	8		LATITE: fine to medium grained, dark grey, light grey and red brown.	HW	M - H	1.8 0.50		(7.46m) CS, 10°, 15 mm.t (7.57m) Jh, 70°, P, R, Fe Ct (7.73m) J, 25°, P, R, Fe Sn (7.81m) J, 0°, P, R, Fe Sn (7.87m) J, 0°, P, R, Fe Sn (7.88m) Jh, 90°, C, R, Fe Ct (8.08m) Jh, 40°, C, R, Fe Ct (8.16m) J, 90°, C, R, Fe Ct (8.31m) J, 90°, C, R, Fe Ct (8.52m) Cr, 0°, 110 mm.t	Dapto Latite
		23 22	9 10		END OF BOREHOLE AT 8.59 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	28/09/2022		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH10P	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No. **101P**

1 / 2

EASTING: 305347.59
NORTHING: 6181972.641

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 39.20 m
Date: 8/2/23 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			39	0		-	ASPHALTIC CONCRETE: 20mm.t	D			SCREEN: 3.62kg 0.1-1.0m, NO FCF
		N = 14 7,9,5					FILL: Silty sand, fine to medium grained, dark brown and dark grey, with fine to medium grained igneous and latite gravel, trace of clay nodules.				APPEARS MODERATELY COMPACTED
			38	1			FILL: Sandy clay, low plasticity, dark brown and brown and dark grey, fine to medium grained sand, with fine to medium grained igneous, ironstone and latite gravel.	w<PL			'TC' BIT REFUSAL
							REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 5.6m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.1m TO 5.6m. CASING 1.1m TO 0.05m. 2mm SAND FILTER PACK 1.4m TO 5.6m. BENTONITE SEAL 0.8m TO 1.4m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
			37	2							
			36	3							
			35	4							
			34	5							
			33	6							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
101P
2 / 2

EASTING: 305347.59
NORTHING: 6181972.641

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 39.20 m
Date: 8/2/23 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
					START CORING AT 1.20m						
		38			NO CORE 0.12m						
80% RETURN; 1 DAY AFTER COMPLETION					LATITE: fine grained, green grey, dark grey and red brown.	SW	VH	4.8, 4.2		(1.35m) J, 60°, C, R, Fe Ct (1.49m) J, 40°, C, R, Fe Ct (1.57m) J, 65°, C, R, Fe Ct (1.86m) Be x 2, 0°, P, R, Fe Ct (2.00m) Bex 2, 0°, P, R, Fe Ct	Dapto Latite
90% RETURN		37, 36, 35, 34	2, 3, 4, 5		LATITE: fine grained, dark grey, speckled green grey and light grey, no defects below 3.35m depth.	FR		5.6, 5.3, 5.5, 6.2, 5.7, 5.1, 6.3, 5.0		(2.50m) Jh, 80°, P, R, Ca FILLED, 2 mm.t (3.35m) Be, 0°, P, R, Fe Vn	
		33, 32	6, 7		END OF BOREHOLE AT 5.66 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	15/03/2023		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH101P	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
102
1 / 2

EASTING: 305386.177
NORTHING: 6182021.597

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 37.25 m
Date: 8/2/23 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				DS	N=SPT 5/ 50mm REFUSAL	37				FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained igneous and latite gravel, trace of root fibres.	M			GRASS COVER
						36	1			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL
						35	2							
						34	3							
						33	4							
						32	5							
						31	6							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. **102**

EASTING: 305386.177
NORTHING: 6182021.597

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 | **Core Size:** NMLC | **R.L. Surface:** 37.25 m
Date: 8/2/23 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		37			START CORING AT 0.60m						
70% RETURN		36	1		LATITE: fine to medium grained dark grey, green grey and red brown.	SW	VH	4.9		(0.68m) J, 70°, P, R, Cn (0.87m) J, 25°, C, R, Fe Ct	Dapto Latite
					as above, but light brown, brown and light grey, non-intact.	MW	M			(1.13m) J, 90°, C, R, Fe Ct	
		35	2		NO CORE 1.55m						
80% RETURN		34	3		NO CORE 0.71m						
					LATITE: fine to medium grained dark grey, green grey and red brown, non-intact.	MW	M			(3.66m) J, 90°, C, R, Fe Ct	Dapto Latite
		33	4		LATITE: fine to medium grained, dark grey speckled green grey and light grey, joint spacing generally >3.0m below 4.2m.	SW	VH	4.9 6.1 6.2 5.2		(3.85m) Be, 0°, P, R, Clay Ct (3.90m) J, 0°, P, R, Fe Ct (3.95m) J, 25°, P, R, Fe Ct (4.16m) J, 0°, P, R, Fe Vn (4.39m) J, 60°, P, R, Fe Ct (4.83m) J x 2, 40°, P, R, Fe Ct (5.55m) J, 40°, P, R, Fe Ct (5.61m) J x 2, 5°, P, R, Fe Ct	
		32	5								
		31	6		END OF BOREHOLE AT 5.62 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW
Approved:	RR		Project: Proposed WCHC Building – Port Kembla
Date:	15/03/2023		Title: **ROCK CORE PHOTOGRAPH**
Scale:	NTS		
Original Size:	A4		Project No: 34300LX2 | BH102 | Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
103

EASTING: 305349.963
NORTHING: 6182017.319

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 37.47 m
Date: 9/2/23 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES ES / U50 / DB / DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING				0		-	ASPHALTIC CONCRETE: 20mm.t				APPEARS POORLY COMPACTED
		N = 4 9,2,2	37				FILL: Silty clay, low to medium plasticity, dark grey and dark brown, with fine to medium grained igneous and ironstone gravel, trace of fine to medium grained sand.	w>PL		160 160 150	SCREEN: 4.2kg 0.1-1.0m, NO FCF
			36	1.5			REFER TO CORED BOREHOLE LOG				'TC' BIT REFUSAL

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
103

EASTING: 305349.963
NORTHING: 6182017.319

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 37.47 m
Date: 9/2/23 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness – Specific	General	Formation
		36			START CORING AT 1.50m							
40% RETURN					LATITE: fine to medium grained, grey, light grey and red brown.	MW	H	2.1		(1.52m) Cr, 10°, 52 mm.t (1.62m) J, 40°, Ir, R, Fe Ct (1.80m) XWS, 0°, 140 mm.t		Dapto Latite
			2		NO CORE 0.56m							
		35			LATITE: fine to medium grained, dark grey and red brown, speckled light grey and green grey.	HW	VH	5.2		(2.46m) J, 60°, Ir, R, Fe Sn (2.62m) J, 50°, P, S, Cn		Dapto Latite
30% RETURN			3		NO CORE 1.27m							
		34										
			4		LATITE: fine to medium grained, dark grey, green grey, light grey and red brown, joint spacing generally >0.2m below 4.3m.	MW	H			(3.95m) Cr, 0°, 90 mm.t (4.05m) J, 90°, C, R, Fe Ct (4.16m) J, 90°, C, R, Fe Ct (4.28m) J, 90°, C, R, Fe Ct		Dapto Latite
		33				SW	VH	2.6 5.7 5.4		(4.63m) J, 10°, Ir, R, Fe Ct (4.80m) Ji, 90°, C, R, Fe Ct (5.08m) J, 50°, P, R, Fe Sn (5.15m) J, 0°, P, R, Fe Ct (5.42m) J, 15°, P, R, Fe Ct		
		32	5									
					END OF BOREHOLE AT 5.63 m							
			6									
		31										
			7									
		30										

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	15/03/2023		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH103	Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
104
1 / 2

EASTING: 305333.881
NORTHING: 6182009.85

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 37.21 m
Date: 10/2/23 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		N = 4 1,2,2	37				FILL: Silty clay, medium plasticity, dark brown and dark grey, with fine to coarse grained sand, and fine to coarse grained igneous and latite gravel, trace of latite cobbles, metal fragments and root fibres.	w>PL		170 180 150	GRASS COVER APPEARS POORLY COMPACTED SCREEN: 10.12kg 0-0.1m, NO FCF
		N = 8 5,4,4	36	1			FILL: Sandy clay, low to medium plasticity, dark brown, dark grey and light brown, fine to coarse grained sand, and fine to coarse grained igneous, latite and sandstone gravel, trace of root fibres.			210 180 220	
			35	2			FILL: Silty clay, medium plasticity, dark grey and dark brown, with fine to medium grained sand, and fine to medium grained igneous and latite gravel, trace of timber fragments and ash.				
		N > 18/ 50mm REFUSAL	34	3		CH	Silty CLAY: high plasticity, green grey, grey and red brown, with fine to medium grained latite gravel, trace of root fibres.	w>PL	St	150 160 170	RESIDUAL
							LATITE: fine to medium grained, green grey, dark grey and red brown.	MW	H		DAPTO LATITE HIGH 'TC' BIT RESISTANCE 'TC' BIT REFUSAL
			33 32 31	4 5 6			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

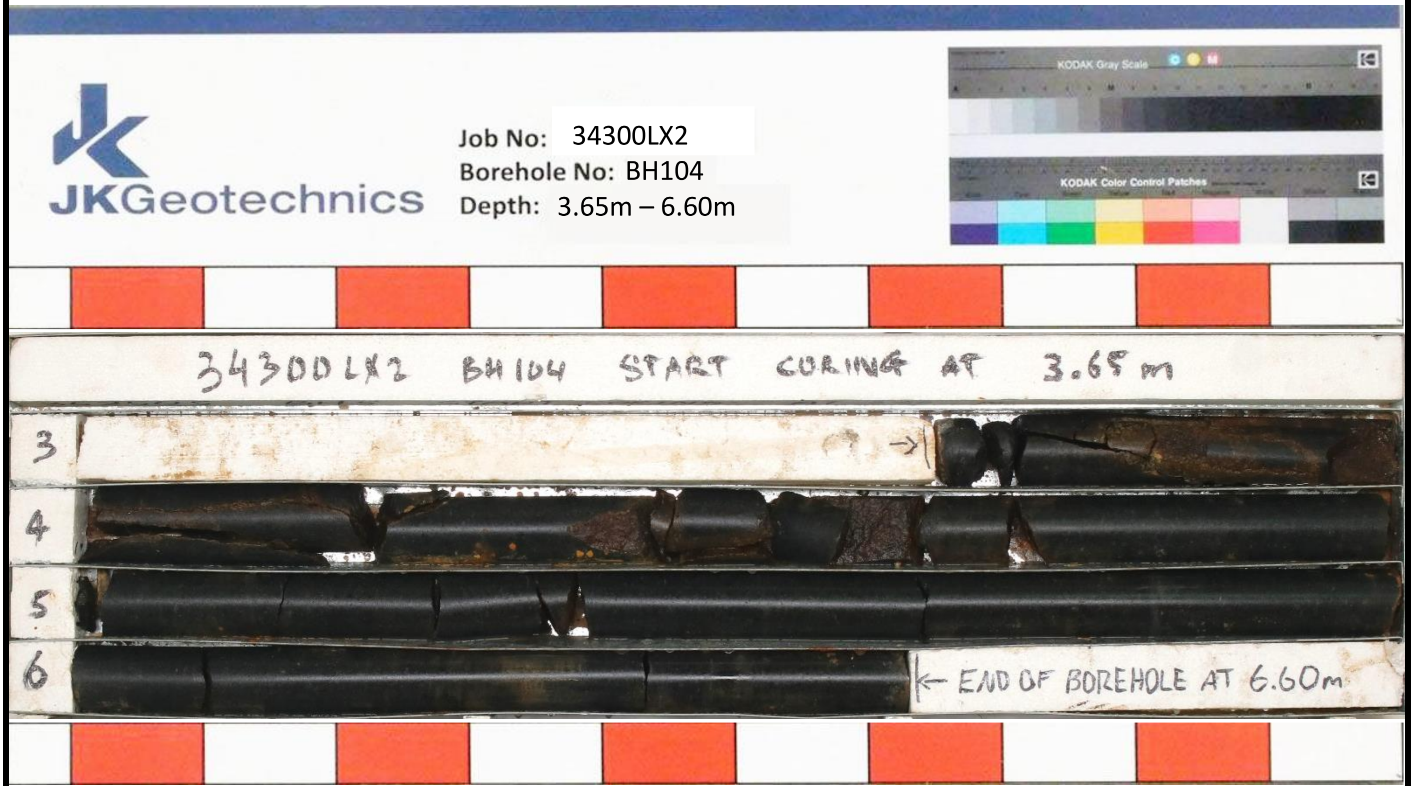
EASTING: 305333.881
NORTHING: 6182009.85

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Core Size:** NMLC **R.L. Surface:** 37.21 m
Date: 10/2/23 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK308 **Bearing:** N/A **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific	General	Formation
		34			START CORING AT 3.65m							
50% RETURN		33	4		LATITE: fine grained, dark grey and red brown speckled green grey and light grey, joint spacing generally >0.6m below 4.7m.	MW	VH	6.7		(3.68m) J, 20°, C, R, Fe Ct (3.71m) J, 10°, C, R, Fe Vn (3.82m) J, 70°, C, R, Fe Ct (4.05m) J, 90°, C, R, Fe Ct		Dapto Latite
		32	5			SW		9.5 4.5 7.9		(4.28m) J, 70°, C, R, Fe Ct (4.45m) J x 2, 20°, P, R, Fe Ct (4.53m) J, 80°, C, R, Fe Ct (4.64m) J, 20°, Ir, R, Fe Vn (4.71m) J, 20°, C, R, Fe Ct (5.36m) Jx 2, 20°, Ir, R, Fe Ct		
		31	6					5.6 5.8 6.9	600 200 60 20	(6.08m) J, 25°, P, R, Fe Ct		
		30	7		END OF BOREHOLE AT 6.61 m							
		29	8									
		28	9						600 200 60 20			

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Drawn:	QV		Client: Health Infrastructure NSW
Approved:	RR		Project: Proposed WCHC Building – Port Kembla
Date:	15/03/2023		Title: **ROCK CORE PHOTOGRAPH**
Scale:	NTS		
Original Size:	A4		Project No: 34300LX2 | BH104 | Rev:

JKGeotechnics

BOREHOLE LOG

Borehole No.
105P

EASTING: 305361.973
NORTHING: 6182043.73

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 **Method:** SPIRAL AUGER **R.L. Surface:** 36.70 m
Date: 7/2/23 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** Q.V./R.R.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			0		FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained igneous and latite gravel, trace of clay fines and root fibres.	D			GRASS COVER APPEARS POORLY COMPACTED
	N = 6 3,2,4	36			FILL: Silty clay, medium plasticity, dark brown and dark grey, with fine to medium grained sand, and fine to medium grained igneous and latite gravel, trace of root fibres.	w>PL		120 150 170	
			1						
	N = 4 2,2,2	35			as above, but dark brown, light brown and dark grey, with fine to medium grained ironstone gravel.			150 180 200	
2 DAYS AFTER COMPLETION		34	2		FILL: Silty clay, medium plasticity, dark grey and dark brown, with fine to medium grained igneous, ironstone and latite gravel.				
	N = 5 3,2,3	33	3 4					220 250 210	GROUNDWATER MONITORING WELL INSTALLED TO 9.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.1m TO 9.1m. CASING 6.1m TO 0.05m. 2mm SAND FILTER PACK 6.5m TO 9.1m. BENTONITE SEAL 5.5m TO 6.5m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
	N = 14 3,4,10	32 31	5 6	CI	Silty CLAY: medium plasticity, green grey and orange brown, with fine to medium grained sand, trace of ironstone gravel.	w>PL	VSt	350 310 280	RESIDUAL
	N=SPT 5/ 10mm REFUSAL	30			REFER TO CORED BOREHOLE LOG				

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
105P
2 / 2

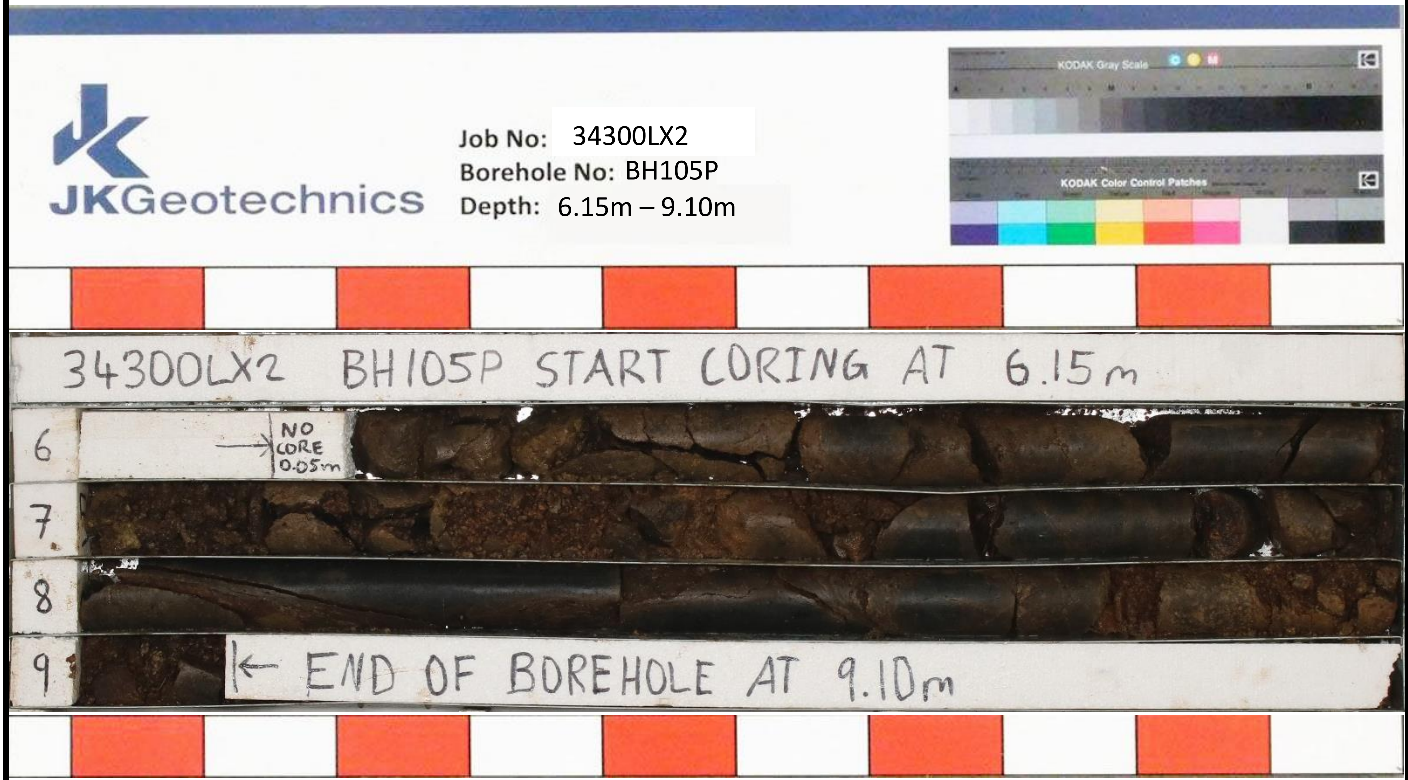
EASTING: 305361.973
NORTHING: 6182043.73

Client: HEALTH INFRASTRUCTURE NSW
Project: PROPOSED WCHC BUILDING - PORT KEMBLA
Location: PORT KEMBLA HOSPITAL, WARRAWONG, NSW

Job No.: 34300LX2 | **Core Size:** NMLC | **R.L. Surface:** 36.70 m
Date: 7/2/23 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** Q.V./R.R.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
					START CORING AT 6.15m						
80% RETURN		30, 29, 28	7, 8, 9		NO CORE 0.05m LATITE: fine grained, dark grey and red brown speckled green grey and light grey, generally non-intact to 7.6m.	HW	M - H			(6.28m) J x 2, 50 - 90°, C, S, Fe Ct (6.35m) J, 80°, C, R, Fe Ct (6.48m) J, 90°, C, R, Fe Ct (6.58m) Ji, 0°, P, R, Fe Sn (6.62m) Jx 2, 40°, P, R, Fe Ct (6.69m) XWS, 30°, 20 mm.t (6.82m) J, 35°, P, R, Fe Ct (6.90m) J, 50°, P, R, Fe Ct (7.06m) XWS, 150 mm.t (7.20m) J, C, R, Fe Ct (7.34m) XWS, 140 mm.t (7.43m) J, P, R, Fe Vn (7.47m) XWS, 30 mm.t (7.53m) J, Ir, Fe Ct (7.55m) XWS, 20 mm.t	Dapto Latite
						SW		0.90		(7.59m) J, P, R, Fe Ct (7.67m) J, P, R, Fe Vn (7.74m) J, C, R, Fe Ct (7.83m) J, P, R, Fe Ct (7.88m) J, P, R, Fe Ct (7.95m) J, C, R, Fe Ct	
							H	1.6		(8.05m) J, C, R, Fe Ct (8.17m) J, C, R, Fe Ct (8.39m) J, C, R, Fe Ct (8.60m) J, C, R, Fe Ct (8.79m) XWS, 45 mm.t	
					as above, but non-intact below 8.8m.	HW				(8.97m) XWS, 190 mm.t (9.09m) J, C, R, Fe Ct	
		27, 26, 25, 24	10, 11, 12		END OF BOREHOLE AT 9.10 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS



Drawn:	QV		Client: Health Infrastructure NSW		
Approved:	RR		Project: Proposed WCHC Building – Port Kembla		
Date:	15/03/2023		Title: **ROCK CORE PHOTOGRAPH**		
Scale:	NTS				
Original Size:	A4		Project No: 34300LX2	BH105P	Rev:

APPENDIX B

Laboratory Test Results Datasheets

115 Wicks Road
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PO Box 976
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Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Shellharbour Hospital
Location: Port Kembla Hospital, Warrawong, NSW

Report No.: 34300LX2 - A
Report Date: 25/10/2022

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
5	1.50 - 1.95	34.0	98	28	70	20.5*

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 11/10/2022.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage cracked/curled.

NATA

NATA Accredited Laboratory Number:1327

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25/10/2022
Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A1
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Report No.:**	34300LX2 - A1
Project:	Proposed WCHC Building - Port Kembla	**Report Date:**	24/11/2022
Location:	Port Kembla Hospital, Warrawong, NSW	**Page 1 of 1**	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
3	1.50 - 1.70	30.6	64	20	44	17.5*

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 16/11/2022.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage curled/cracked.

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24/11/2022
Authorised Signature / Date
(D. Treweek)

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Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A2
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client:	JK Geotechnics	**Report No.:**	34300LX2 - A2
Project:	Proposed WCHC Building - Port Kembla	**Report Date:**	15/03/2023
Location:	Port Kembla Hospital, Warrawong, NSW	**Page 1 of 1**	

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
101	0.50 - 0.95	16.0	-	-	-	-
103	0.50 - 0.95	20.7	-	-	-	-
104	1.50 - 1.95	10.2	29	16	13	6.5
105	3.00 - 3.45	38.6	63	20	43	15.0*
105	4.50 - 4.95	31.0	-	-	-	-

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 16/02/2023.
- * Denotes Linear Shrinkage curled.
- Sampled and supplied by client. Samples tested as received.

NATA

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15/03/2023
Authorised Signature / Date
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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B
ROCK UNIAXIAL COMPRESSIVE STRENGTH REPORT

Report No.: 34300LX2 - B
Date: 26/10/2022
Page No: 1 of 1

Client: JK Geotechnics
Project: Proposed Shellharbour Hospital
Location: Port Kembla Hospital, Warrawong, NSW

Borehole Number	4	8			
Depth (m)	3.15 - 3.23	3.21 - 3.37			
Date Tested	24/10/2022	24/10/2022			
Mass per Unit Volume (t/m³):	2.784	2.762			
Moisture Content of Sample (%)	0.7	0.7			
Dimensions:					
Average Diameter (mm)	51.7	51.6			
Length (mm)	149	129			
Core Strength (MPa)	92.4	44.4			
Time to Failure (minutes)	17	11			
Mode of Failure	Tensile Dominated	Tensile Dominated			

Notes: Sample supplied and sampled by client.

- Cores tested in "as received condition".
- Date of receipt of sample: 12/10/2022.
- Test method completed within general accordance with AS 4133.4.3.2.
- Refer to core logs for material description.
- Wykeham Farrance Test Apparatus - 200kN Load Cell.

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B1
ROCK UNIAXIAL COMPRESSIVE STRENGTH REPORT

Report No.: 34300LX2 - B1
Date: 22/11/2022
Page No: 1 of 1

Client: JK Geotechnics
Project: Proposed WCHC Building - Port Kembla
Location: Port Kembla Hospital, Warrawong, NSW

Borehole Number	2	7		
Depth (m)	5.05 - 5.20	1.55 - 1.77		
Date Tested	18/11/2022	18/11/2022		
Mass per Unit Volume (t/m³):	2.753	2.768		
Moisture Content of Sample (%)	1.4	1.2		
Dimensions:				
Average Diameter (mm)	51.7	51.8		
Length (mm)	108	170		
Core Strength (MPa)	101.5	117.4		
Time to Failure (minutes)	8.5	8		
Mode of Failure	Tensile Dominated	Tensile Dominated		

Notes: Sample supplied and sampled by client.
- Cores tested in "as received condition".
- Date of receipt of sample: 15/11/2021.
- Test method completed within general accordance with AS 4133.4.3.2.
- Refer to core logs for material description.
- Wykeham Farrance Test Apparatus - 200kN Load Cell.

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE B2
ROCK UNIAXIAL COMPRESSIVE STRENGTH REPORT

Report No.: 34300LX2 – B2
Date: 7/03/2023
Page No: 1 of 1

Client: JK Geotechnics
Project: Proposed WCHC Building - Port Kembla
Location: Port Kembla Hospital, Warrawong, NSW

Borehole Number	102	104			
Depth (m)	4.40 - 4.60	4.70 - 5.00			
Date Tested	3/03/2023	3/03/2023			
Mass per Unit Volume (t/m^3):	2.801	2.773			
Moisture Content of Sample (%)	2.6	0.5			
Dimensions:					
Average Diameter (mm)	51.5	51.6			
Length (mm)	153	147			
Core Strength (MPa)	58.4	96.1			
Time to Failure (minutes)	25 mins	N/A			
Mode of Failure	Tensile Dominated	N/A - No break			

Notes: Sample supplied and sampled by client.

- Cores tested in "as received condition".
- Date of receipt of sample: 16/02/2023.
- Test method completed within general accordance with AS 4133.4.3.2.
- Refer to core logs for material description.
- Wykeham Farrance Test Apparatus - 200kN Load Cell.

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure **Ref No:** 34300LX2

Project: Proposed WCHC Building - Port Kembla **Report:** C

Location: Port Kembla Hospital, Warrawong, NSW **Report Date:** 15/11/22

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH1	1.74 - 1.78	5.9	118	A
	2.84 - 2.88	5.5	110	A
	3.05 - 3.09	7.4	148	A
	3.62 - 3.65	7.7	154	A
	4.11 - 4.14	5.7	114	A
	5.06 - 5.09	7.3	146	A
BH2	4.64 - 4.67	7.3	146	A
	5.25 - 5.27	2.7	54	A
	6.31 - 6.34	6.7	134	A
	7.05 - 7.08	1.6	32	A
BH3	2.60 - 2.64	3	60	A
	4.37 - 4.40	6.4	128	A
	4.68 - 4.71	8.8	176	A
	5.10 - 5.14	6.1	122	A
	5.71 - 5.74	7.6	152	A
BH4	1.28 - 1.32	5.5	110	A
	1.67 - 1.71	7.1	142	A
	2.28 - 2.32	7.4	148	A
	2.75 - 2.78	9	180	A
	3.22 - 3.25	6.5	130	A
	3.76 - 3.79	8.7	174	A
	4.26 - 4.29	7.9	158	A
	4.71 - 4.75	4.3	86	A
	5.28 - 5.31	7.7	154	A
	5.79 - 5.82	7.1	142	A

NOTES

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure **Ref No:** 34300LX2

Project: Proposed WCHC Building - Port Kembla **Report:** C

Location: Port Kembla Hospital, Warrawong, NSW **Report Date:** 15/11/22

BOREHOLE NUMBER	DEPTH (m)	$I_{S\,(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH5	4.00 - 4.03	5.7	114	A
	4.74 - 4.76	6.1	122	A
	6.76 - 6.79	6.1	122	A
	7.17 - 7.21	7	140	A
	7.69 - 7.73	7.2	144	A
	8.07 - 8.11	8.2	164	A
BH6	3.44 - 3.47	7.4	148	A
	4.07 - 4.09	10	200	A
	4.69 - 4.72	7.6	152	A
	5.29 - 5.33	7.1	142	A
	5.80 - 5.83	6.3	126	A
BH7	1.83 - 1.87	5.7	114	A
	2.10 - 2.14	6.7	134	A
	2.64 - 2.66	7.3	146	A
	3.30 - 3.34	6.6	132	A
	4.95 - 4.99	7.5	150	A
	5.09 - 5.12	5.5	110	A
	5.73 - 5.77	7.4	148	A
	6.00 - 6.04	6.3	126	A
BH8	1.86 - 1.89	5.1	102	A
	2.18 - 2.21	7.3	146	A
	3.19 - 3.22	7	140	A
	3.66 - 3.69	6.6	132	A
	4.28 - 4.30	5.5	110	A
	4.85 - 4.88	6.2	124	A

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure **Ref No:** 34300LX2

Project: Proposed WCHC Building - Port Kembla **Report:** C

Location: Port Kembla Hospital, Warrawong, NSW **Report Date:** 15/11/22

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
BH8	5.04 - 5.06	5.4	108	A
	5.55 - 5.57	8.1	162	A
BH9	1.28 - 1.32	8.9	178	A
	2.29 - 2.32	2.8	56	A
	2.66 - 2.69	7	140	A
	4.75 - 4.78	7.3	146	A
	5.92 - 5.94	4.9	98	A
BH10	5.50 - 5.54	0.5	10	A
	6.55 - 6.59	0.8	16	A
	6.74 - 6.77	1.8	36	A
	7.76 - 7.80	1.8	36	A
	8.23 - 8.26	1.5	10	A

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

TABLE C1
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure NSW **Ref No:** 34300LX2

Project: Proposed WCHC Building - Port Kembla **Report:** C1

Location: Port Kembla Hospital, Warrawong,NSW **Report Date:** 17/02/23

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
101	1.39 - 1.40	4.8	96	A
	1.83 - 1.85	4.2	84	A
	2.20 - 2.24	5.6	112	A
	2.76 - 2.78	5.3	106	A
	3.26 - 3.29	5.5	110	A
	3.70 - 3.74	6.2	124	A
	4.26 - 4.29	5.7	114	A
	4.82 - 4.85	5.1	102	A
	5.12 - 5.15	6.3	126	A
	5.50 - 5.54	5	100	A
102	0.78 - 0.82	4.9	98	A
	4.05 - 4.08	4.9	98	A
	4.67 - 4.70	6.1	122	A
	5.11 - 5.15	6.2	124	A
	5.49 - 5.52	5.2	104	A
103	1.63 - 1.67	2.1	42	A
	2.51 - 2.55	5.2	104	A
	4.89 - 4.93	2.6	52	A
	5.24 - 5.27	5.7	114	A
	5.68 - 5.72	5.4	108	A
104	3.65 - 3.68	6.7	134	A
	4.32 - 4.36	9.5	190	A
	4.66 - 4.69	4.5	90	A
	5.15 - 5.18	7.9	158	A
	5.78 - 5.81	5.6	112	A

NOTE: SEE PAGE 2

TABLE C1
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Health Infrastructure NSW

Ref No: 34300LX2

Project: Proposed WCHC Building - Port Kembla

Report: C1

Location: Port Kembla Hospital, Warrawong,NSW

Report Date: 17/02/23

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
104	6.22 - 6.25	5.8	116	A
	6.44 - 6.48	6.9	138	A
105	7.84 - 7.88	0.9	18	A
	8.71 - 8.74	1.6	32	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 316819

Client Details	
Client	JK Geotechnics
Attention	Alexander James Moran
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details	
Your Reference	**34300LX2**
Number of Samples	2 Soil
Date samples received	17/02/2023
Date completed instructions received	17/02/2023

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details	
Date results requested by	24/02/2023
Date of Issue	24/02/2023

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		316819-1	316819-2
Your Reference	UNITS	104	105
Depth		1.5-1.95	3-3.45
Date Sampled		16/02/2023	16/02/2023
Type of sample		Soil	Soil
Date prepared	-	23/02/2023	23/02/2023
Date analysed	-	23/02/2023	23/02/2023
pH 1:5 soil:water	pH Units	6.6	7.0
Chloride, Cl 1:5 soil:water	mg/kg	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	260
Resistivity in soil*	ohm m	560	43

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			23/02/2023	[NT]	[NT]	[NT]	[NT]	23/02/2023	[NT]
Date analysed	-			23/02/2023	[NT]	[NT]	[NT]	[NT]	23/02/2023	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	98	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	99	[NT]

Result Definitions	
NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.