Our Ref: 190996 BMY/LF

6th February 2025

Port Stephens Council
PO Box 42
RAYMOND TERRACE NSW 2324

Dear Sir/Madam,

RE RESIDENTIAL APARTMENT BUILDING
LOTS 781 AND 782 DP 802108
38 STOCKTON STREET AND 8A TOMAREE STREET, NELSON BAY
STORMWATER MANAGEMENT PLAN

1.0 INTRODUCTION

With respect to the above-mentioned project, COHO Property has engaged ADW Johnson to prepare a Stormwater Management Plan to address the stormwater management requirements for the proposed residential development.

This report is to form part of the Development Application for the proposed development and details the constraints on the site, Council's stormwater management requirements and the proposed stormwater infrastructure to meet Council's requirements.

2.0 SITE INFORMATION

The site, being Lots 781 and 782 DP 802108, is approximately 0.23ha, zoned Local Centre (E1) and is bounded by Stockton Street to the east, Tomaree Street to the south and existing development to the east and north. The site locality can be seen in Figure **1**.

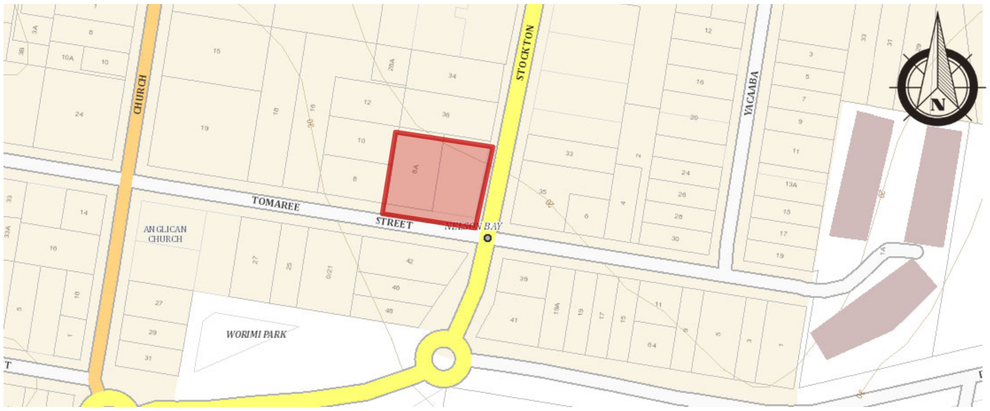


Figure 1: Site Locality
Source: SIX Maps

ADW JOHNSON PTY LIMITED
ABN 62 129 445 398

Sydney
Level 35 One International Towers
100 Barangaroo Avenue
Sydney NSW 2000
02 8046 7411
sydney@adwjohnson.com.au

Central Coast
5 Pioneer Avenue, Tuggerah NSW 2259
PO Box 3717, Tuggerah NSW 2259
02 4305 4300
coast@adwjohnson.com.au

Hunter
7/335 Hillsborough Road,
Warners Bay NSW 2282
02 4978 5100
hunter@adwjohnson.com.au

www.adwjohnson.com.au

2.1 Existing Site Drainage

The site generally falls from south-west to north-east towards Stockton Street with a total fall of approximately 4m. Majority of the runoff from the site currently discharges via sheet flow to Stockton Street, with no formal drainage infrastructure onsite with the exception of a small grated surface inlet pit located in a low point along the northern boundary. This inlet pit captures flows and discharges them north through the adjoining property via a single 100mm pvc pipe.

A review of the site survey, and a site inspection, indicates that there is no formal drainage infrastructure located in Stockton Street or within the vicinity of the site. A number of lintel pits are located within Tomaree Street, on the high side of the development site.

A copy of the site survey can be seen in **Appendix A.**

2.2 Geotechnical Parameters

Given the lack of stormwater infrastructure available to connect into, it is intended to infiltrate stormwater onsite. To inform the modelling parameters for the infiltration system, Tetratech undertook a geotechnical investigation of the site on 16th and 17th September.

The Tetratech report, which can be seen in **Appendix B**, outlines a number of key geotechnical parameters for the development. Specifically, in relation to the infiltration modelling, it has provided tested infiltration rates and water table levels.

2.2.1 Infiltration Rates

Infiltration testing was undertaken in two soil profiles, being the upper and lower aeolian sands. A review of the proposed building levels indicates that the base of the infiltration tank will be located in the upper aeolian sand profile and therefore the tested infiltration rate adopted for this report is 4 x 10^{-6} m/sec. Refer to the geotechnical report for full details of the infiltration testing.

2.2.2 Water table

Three monitoring wells were installed onsite to determine the groundwater level at various locations within the site. The following water depths were found at each of the monitoring wells and are detailed in **Table 1** below.

Table 1: Groundwater Levels from Survey and Geotech Data

Piezometer ID	RL Ground Level (m)	Depth to Water (m bgl)	Water Table Level (RL) (m)
D-BH01 (MW01)	20.51	10.74	9.77
B-BH02 (MW02)	23.37	12.46	10.91
GW-Well (MW03)	20.61	11.55	9.06

*m bgl – m below ground level
*GW-Well was an existing well from a previous investigation
*RL measured at top of monitoring well cap

The tank is located in between all three wells and therefore, the worst-case water table level of RL 10.91 was adopted for the purposes of this model.

2.2.3 Factor of Safety

In accordance with Port Stephens Council's infiltration technical manual, an appropriate factor of safety is to be applied to the infiltration rate for the purposes of the modelling. The factor of safety to be utilised is dependent on the risk profile of the catchment. The proposed development has been adopted in a low risk profile due to:

- There is no upstream catchment draining to the infiltration tank;
- The system is a closed system internal to the building structure and therefore blockage of the system is highly unlikely;
- Flows will be collected in water quality treatment devices before being discharged into the tank, once again minimising any risk of blockages;
- The water table is approximately 3.5m from the base of the tank (greater than the minimum 1m required) (see tank parameters in **Table 3**);
- Emergency overflow from the tank is provided to a public road and the therefore the consequence of failure is low.

Based upon the above, a factor of safety of **five** has been adopted. For the purposes of modelling, an infiltration rate of 8 x 10^{-6} m/sec has been adopted.

3.0 STORMWATER MANAGEMENT

The stormwater management system for the developed site consists of a treatment train of:

- All roof water is to be directed to a 5kL rainwater tank;
- Overflows from the tank are directed to a water quality chamber;
- Overflows from the water quality chamber are directed to an infiltration tank;
- Runoff external to the buildings is to be captured in grated surface inlet pits fitted with litter baskets. Captured flows are conveyed directly to the infiltration tank;
- An emergency overflow riser is to be provided to ensure any overflows from the infiltration tank are safely discharged to a public road.

The following sections of this report outline the stormwater management devices required to achieve compliance with Port Stephens Councils DCP.

3.1 Stormwater Infiltration

As mentioned in Section 2.1 there is limited drainage infrastructure for the site to discharge to. It has therefore been decided that the proposed infiltration tank is to be sized to infiltrate all flows from design storm events up to the 1% AEP event.

The sizing of the infiltration tank was conducted using a DRAINS routing model in conjunction with the Intensity-Frequency-Duration (IFD) data from the Council's document "Stormwater and Water Efficiency for Development" (2017).

The DRAINS model utilised the following catchment parameters, detailed in **Table 2**.

Table 2: Catchment Parameters

Area Type	Description	Area (m²)	Percentage of Total Site (%)
Pervious Areas	Landscape Areas	35.64	1.48
	Deep Soil Areas	476.44	19.75
Impervious Areas	Hardstand & Footpath Areas	150.00	6.22
	Roof Areas	1750.58	72.56

The infiltration tank size adopted was determined to ensure a functional architectural / structural design along with maximising the tank footprint to limit the depth of the tank.

As mentioned in Section 2.2.3, an emergency overflow pipe will be provided to ensure water has a safe escape route should the system block up.

The tank location, including the emergency overflow, can be seen in the engineering sketches attached in **Appendix C**.

The DRAINS model was analysed to determine the required storage volume within the tank to successfully infiltrate all design storms up to the1% AEP event. The DRAINS modelling resulted in the following tank parameters:

Table 3: Drains Modelling Results

Parameter	Value	Units
Top of Tank RL*	15.825	m
Infiltration Base RL	14.425	m
Tank Area	313	m^2
Top Water Level (1% AEP Event)	15.81	m

** Assumes a 250mm thick basement slab*

It can be seen from **Table 3** that the provision of a 1.4m deep by 313m^2 infiltration tank ensures that the 1% AEP event can be infiltrated.

Port Stephens Development Control Plan (PSDCP) require that post developed outflow from the subject site be demonstrated in all storm events. The proposed stormwater management system infiltrates the entire developed catchment into the treatment train provided in Section 3.0. This results in the post-developed flow rate and volume in all design storm events to be zero.

As highlighted in Appendix C, an Emergency Overflow Pipe is to be constructed at the top of the proposed infiltration tank that daylights at street level in the case of an emergency water level beyond the 1% AEP event.

The provision of an infiltration tank with the abovementioned parameters will ensure that the development has no impact on the surrounding stormwater network and therefore detention or external upgrades are not required.

3.2 Water Quality

To ensure PSC's water quality requirements are met, a treatment train consisting of the following devices are proposed:

- 5kL rainwater tank;
- Ocean Protect Storm Filter chamber (6.3m^2);
- 6 x 690mm Psorb Storm Filter cartridges within chamber.

The effectiveness of these treatment devices in achieving desired water quality outcomes was assessed using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC). This model simulates pollutant source elements and evaluates the treatment performance of the installed systems.

The stormwater management system is designed to efficiently direct and treat runoff from various catchment areas as follows:

Roof Catchment:

All rainwater collected from the roof of the residential apartment building is directed to a 5kL rainwater tank. This tank serves as the primary storage for rooftop runoff, allowing for reuse and reducing the volume of stormwater that requires treatment. The tank is equipped with an overflow mechanism to manage excess flow during heavy rainfall events.

Water Quality Chamber:

Overflow from the rainwater tank is directed to a water quality (WQ) chamber. This chamber is equipped with the Psorb Storm Filters, which effectively treat pollutants and fine particles before they can enter the downstream systems. This step is crucial for improving the quality of the water prior to infiltration.

Infiltration Basin:

After passing through the water quality chamber, the treated water or any overflow passes into the infiltration tank. Here, any remaining runoff is allowed to infiltrate into the surrounding soil, replenishing groundwater and minimising surface runoff, any overflow is discharged into Stockton Street. The design ensures that the infiltration capacity meets the needs of the site while complying with local regulations.

External Runoff:

Runoff from external areas, such as driveways and landscaped zones, is captured using grated surface inlet pits fitted with litter baskets. These pits collect debris and larger particles, preventing them from entering the stormwater system. The captured water is then conveyed directly to the infiltration tank for treatment and infiltration.

Port Stephens Development Control Plan 2014 (DCP) outlines relevant water quality objectives to be met by a development.

The relevant water quality targets to be met for our site as per B4.C of the DCP are as follows:

- Total Suspended Solids (TSS): 90% reduction in annual loading;
- Total Nitrogen (TN): 45% reduction in annual loading;
- Total Phosphorus (TP): 60% reduction in annual loading;
- Gross Pollutants (GP): 90% reduction in annual loading.

The results of the MUSIC modelling are summarised in **Table 4.**

Table 4: Treatment Train Effectiveness – Residential Apartment Building

Pollutant	Targets (%)	Reduction (%)
Total Suspended Solids TSS (kg/yr)	90.0	90.1
Total Phosphorus TP (kg/yr)	60.0	77.8
Total Nitrogen TN (kg/yr)	45.0	54.5
Gross Pollutants GP (kg/yr)	90.0	100

Table 4 demonstrates that the treatment train effectively reduced pollutants from the development, achieving efficiencies that exceed the established reduction targets given in Port Stephens DCP.

4.0 EROSION AND SEDIMENT CONTROL

Port Stephens Council requires the use of erosion and sediment controls to manage and contain pollutant runoff, both during construction and as long-term permanent treatments thus ensuring the minimisation of impact on the environment. All erosion and sediment controls and practices are to be in accordance with PSDCP.

During construction the treatment devices which will be utilised to contain the generated pollutants from the site may include, but are not limited to:

- Sediment basins;
- Silt fencing;
- Strawbale and geotextile fencing;
- Kerb inlet controls;
- Sandbag kerb inlet sediment traps;
- Shaker ramp;
- Diversion drains.

A detailed erosion and sediment control plan will be provided as part of the detailed design.

5.0 CONCLUSION

Through the provision of a water quality and conveyance treatment train, the proposed development complies with Port Stephens Council's DCP requirements for stormwater.

Should you have any questions or require any further advice please do not hesitate to contact the undersigned on 02 4305 4300 or benm@adwjohnson.com.au

Yours faithfully

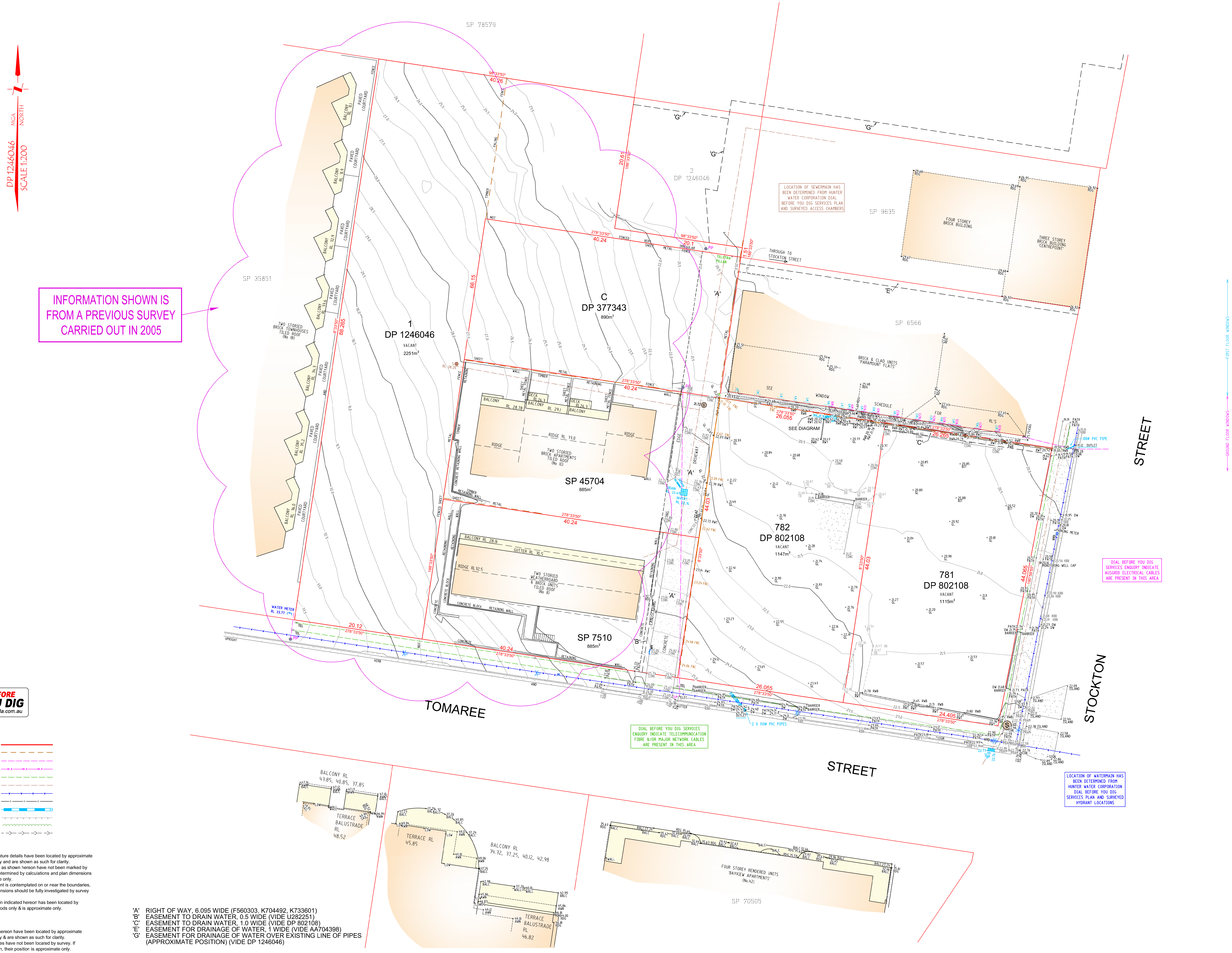
Ben Myles
SENIOR CIVIL ENGINEER
ADW JOHNSON PTY LTD
S:\190996\Design\Documents\190996 - Stormwater Management Plan.docx

Appendices:

Site Survey
Geotech Report
Concept Drawings

Appendix A

SITE SURVEY



LOCATION OF SEWERMAIN HAS BEEN DETERMINED FROM HUNTER WATER CORPORATION DIAL BEFORE YOU DIG SERVICES PLAN AND SURVEYED ACCESS CHAMBERS

DIAL BEFORE YOU DIG SERVICES ENQUIRY INDICATE TELECOMMUNICATION FIBRE &/OR MAJOR NETWORK CABLES ARE PRESENT IN THIS AREA

DIAL BEFORE YOU DIG SERVICES ENQUIRY INDICATE AUSGRID ELECTRICAL CABLES ARE PRESENT IN THIS AREA

LOCATION OF WATERMAIN HAS BEEN DETERMINED FROM HUNTER WATER CORPORATION DIAL BEFORE YOU DIG SERVICES PLAN AND SURVEYED HYDRANT LOCATIONS

'A' RIGHT OF WAY, 6.095 WIDE (F560303, K704492, K733601)
'B' EASEMENT TO DRAIN WATER, 0.5 WIDE (VIDE U282251)
'C' EASEMENT TO DRAIN WATER, 1.0 WIDE (VIDE DP 802108)
'E' EASEMENT FOR DRAINAGE OF WATER, 1 WIDE (VIDE AA704398)
'G' EASEMENT FOR DRAINAGE OF WATER OVER EXISTING LINE OF PIPES (APPROXIMATE POSITION) (VIDE DP 1246046)

LEGEND

× 41.23 GL	GROUND SURFACE LEVEL
DW	DRIVEWAY
WALL	GROUND LEVEL AT WALL EDGE
BB	BOTTOM OF BANK
TB	TOP OF BANK
FNC	FENCE
RWT	TOP OF RETAINING WALL
RWB	BOTTOM OF RETAINING WALL
KBB	BACK EDGE OF KERB
LIP	EDGE OF GUTTER/BITUMEN
BES	BACK OF EDGE STRIP
H	WATER HYDRANT
Xsv	WATER STOP VALVE
⊓	WATER METER
Ⓢ	SEWER CHAMBER/MAIN. SHAFT
S●	SEWER INSPECTION POINT
○	LIGHT POLE
●	POWER POLE
EP	ELECTRICITY PILLAR
▦	STORMWATER PIT
▭	LINTEL STORMWATER PIT
▢	LBO STORMWATER PIT
T	TELECOMMUNICATIONS PIT
▽	SURVEY STATION
—○—	ELECTRIC LIGHT POLE
—●—	ELECTRICITY POLE
○	ELECTRICITY PIT
▲	BENCH MARK
GE	EDGE OF GARDEN

WINDOW SCHEDULE

WINDOW	HEAD RL	SILL RL
W1	24.37	23.14
W2	24.37	23.14
W3	24.43	23.21
W4	24.31	23.68
W5	24.20	23.56
W6	24.26	23.12
W7	24.47	23.78
W8	24.46	23.76
W9	24.98	24.38
W10	24.94	24.34
W11	24.94	24.31
W12	25.48	24.24
W13	25.53	25.00
W14	25.52	25.00
W15	25.49	24.25
W16	25.48	24.23
W17	25.56	25.01
W18	21.85	20.85
W19	21.85	20.85
W20	22.35	21.75
W21	22.35	21.75
W22	22.35	21.75
W23	22.35	21.75
W24	22.60	21.20
W25	22.60	22.00
W26	22.60	21.20
W27	22.60	21.20
W28	22.62	22.02

FIRST FLOOR WINDOWS (W1–W17)
GROUND FLOOR WINDOWS (W18–W28)

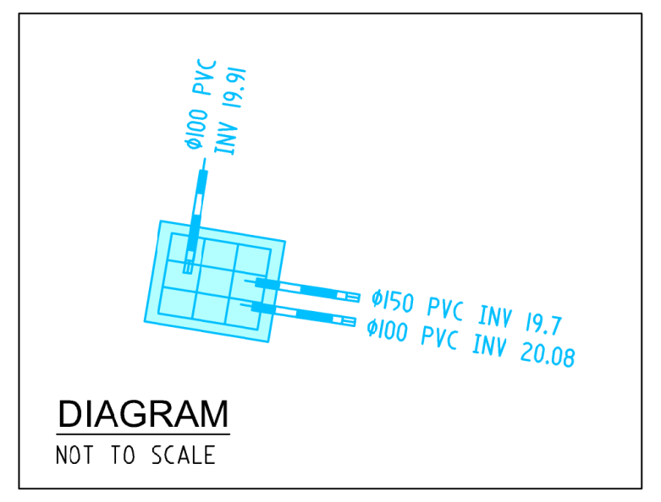


DIAGRAM
NOT TO SCALE

BEFORE YOU DIG
www.byda.com.au

LINETYPE LEGEND:

Boundary line	——
Fenceline	— / — / —
Overhead Electricity	—E—E—E—
Underground Electricity	—UG E—UG E—
Telephone Cable	—T—T—T—
Sewer main	—S—S—S—
Water main	—W—W—W—
Gas main	—G—G—G—
Stormwater pipe	▬▭▬▭
Top of bank	┬┬┬┬
Treeline	⌒⌒⌒⌒
Open Drain/Tabledrain	—>— —>—

GENERAL NOTE:

1. Topographical & feature details have been located by approximate survey methods only and are shown as such for clarity.
2. The title boundaries as shown hereon have not been marked by us but have been determined by calculations and plan dimensions and are approximate only.
3. If further development is contemplated on or near the boundaries, then boundary dimensions should be fully investigated by survey & marks placed.
4. Adjoining information indicated hereon has been located by remote survey methods only & is approximate only.

SERVICES CAUTION

5. Services indicated hereon have been located by approximate survey methods only & are shown as such for clarity.
6. Underground services have not been located by survey. If indicated on the plan, their position is approximate only.
7. Verification of the service location should be sought with the relevant authority.
8. General caution is hereby given to any person excavating on site. Service enquiries to dial before you dig should be undertaken prior to such excavation. No liability to disruption or interference with any existing services due to the action of others exist through the making of this topographical survey.
9. **CONTACT DIAL BEFORE YOU DIG.**

0 4 8 12 16 20m
1:200 (A0)

AMENDMENTS
19/06/23 - INITIAL ISSUE
26/08/24 TS2 - BUILDINGS TO THE SOUTH
30/08/24 TS3 - WINDOWS AND RIDGES (SP 6566 & SP 9635)
02/09/24 TS3 - DRAINAGE AND EASEMENT DETAIL ADDED
31/01/25 TS4 - DRIVEWAY & RETAINING WALL INFO. ADDED (SP 7510 & SP 45704)

SCALE	DATUM	CONTOURS
1: 200 (A0)	AHD	0.5 m

SURVEY DATE: 31/05/2023
SURVEYOR: TD
PLOT DATE: 31/01/2025
DRAWN/CHKD BY: TD / AD
FILENAME: N:\38093 COHO\Drawings\38093 TS4.dwg

DRAWING TITLE
TOPOGRAPHICAL SURVEY

DEVELOPMENT
LOTS 781 & 782 DP 802108, LOT 1 DP 1246046,
LOT C DP 377343, SP 45704 AND SP 7510

CLIENT
COHO PROPERTY

NORTH POINT
SURVEYS
7B | 10 Yacaaba Street | Nelson Bay NSW 2315
PO Box 116 | Nelson Bay NSW 2315
Phone: 02 4984 2606
Email: admin@northpointsurveys.com.au

38093
TS4
TOPOGRAPHICAL SURVEY
1 OF 1

Appendix B

GEOTECH REPORT

Proposed Residential Development - 38 Stockton and 8A Tomaree Street, Nelson Bay

Geotechnical Investigation Report

COHO Property Pty Ltd

Reference: 754-NTLGE368007-AC.Rev1

14 October 2024

PROPOSED RESIDENTIAL DEVELOPMENT - 38 STOCKTON AND 8A TOMAREE STREET, NELSON BAY

Geotechnical Investigation Report

Report reference number: 754-NTLGE368007-AC.Rev1
14 October 2024

PREPARED FOR

COHO Property Pty Ltd
PO Box 1, Gladesville
NSW 1675

PREPARED BY

Tetra Tech Coffey
Unit 4, 60 Griffiths Road, Lambton NSW 2299 Australia
p: +61 2 4028 9700

ABN 55 139 460 521

This report presents our geotechnical investigation findings for 38 Stockton and Tomaree Street, Nelson Bay. This report supersedes any previous reports. Should you have questions regarding this report please contact the undersigned. We appreciate the opportunity to be of service.

For and on behalf of Tetra Tech Coffey

Merrick Jones
Senior Geotechnical Engineer

QUALITY INFORMATION

Revision history

Revision	Description	Date	Author	Reviewer	Approver
V0	Draft – initial release	3/10/2024	Merrick Jones	Jules Darras /Dr. Ching Dai	Merrick Jones
V1	Incorporate minor amendments	14/10/2024	Merrick Jones	Jules Darras	Merrick Jones

Distribution

Report Status	No. of copies	Format	Distributed to	Date
V0	1	PDF	Jared Buncombe	3/10/2024
V1	1	PDF	Jared Buncombe	14/10/2024

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APPENDICES

ACRONYMS/ABBREVIATIONS

Acronyms/Abbreviations	Definition
AHD	Australian Height Datum
BGL	Below Ground Level
BH	Borehole
CBR	California Bearing Ratio
CH	Chainage
CPT	Cone Penetrometer Testing
CPTu	Cone Penetrometer Testing with pore pressure measurement
COHO	COHO Property Pty Ltd
D	Dense
DA	Development Application
DBYD	Dial Before You Dig
DCP	Dynamic Cone Penetrometer
DMT	Flat Plate Dilatometer Testing
EC	Electrical Conductivity
GPS	Global Positioning System
HSSE	Health, Safety, Security & Environment
kPa	Kilopascals
L	Loose
MD	Medium Dense
mAHD	Metres Australian Height Datum
mBGL	Metres Below Ground Level
NATA	National Association of Testing Authorities
PSD	Particle Size Distribution
PSI	Preliminary Site Investigation
QA / QC	Quality Assurance/ Quality Control
RL	Reduced Level
SPT	Standard Penetration Test
SWMS	Safe Work Method Statement
Tetra Tech	Tetra Tech Coffey Pty Ltd
UPSS	Underground Petroleum Storage System
UST	Underground storage tank
VD	Very Dense
VL	Very Loose

1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Tetra Tech Coffey Pty Ltd (Tetra Tech) on behalf of COHO Property Pty Ltd (COHO), for the proposed development located at 38 Stockton and 8A Tomaree Street, Nelson Bay NSW, referred to herein as The Site. This revised report supersedes previous versions.

The work has been undertaken in general accordance with the scope, terms and conditions outlined in our proposal, reference 754-NTLGE368007-AA dated 6 September 2024. Environmental and contamination aspects nominated in our proposal are reported separately 754-NTLGE368007-AB.

Based on the drawings by Holdsworth Design Project No. 0159 the proposed development is to be eight storeys above ground and two basement levels. Historically the site was used as a petroleum station and mechanic's workshop. The site history and previous use of the site is covered in the contamination site suitability assessment report by Tetra Tech Coffey (2024) 754-NTLGE368007-AB dated 2 October 2024.

1.1 OBJECTIVES

Tetra Tech understands that the project is in the Development Application (DA) stage and that this report will inform the future engineering design. The objectives of this geotechnical report were to investigate the current ground conditions and to:

- Provide site classification for residential lots in accordance with AS2870-2011 to be used as a general guide for the development.
- Comment on founding conditions and provide geotechnical parameters for shallow (pad or strip) footing design including allowable bearing capacities for footings in accordance with AS2870-2011 (noting that AS2870 is applicable to residential structures of up to two levels).
- Provide preliminary deep foundation design parameters in accordance with AS2159-2009.
- Provide earthquake classification in accordance with AS1170.4.
- Provide preliminary retaining wall / shoring design parameters (K_0, K_a, K_p) noting these parameters are dependent on the type of retaining structure adopted.
- Provide general guidelines for earthworks, comment on excavatability and excavation stability.
- Provide preliminary pavement design with comments on construction methods, material specification and drainage.
- Comment on the basic geotechnical reduction factor for single and grouped piles with and without pile testing benefit.
- Comment on the inferred groundwater level and how it may affect the proposed development.
- Assess the approximate permeability (hydraulic conductivity) of the soils for the site using appropriate method(s) such as the Hvorslev method which has been undertaken for similar developments for PSC.
- Assess soil aggressivity to buried structural elements (durability of piling systems AS2159).
- Commentary on if the development will disturb Acid Sulfate Soils (ASS) based on field screening tests.
- Provide sufficient geotechnical investigation data to assist with the foundation engineering design undertaken by the nominated Structural Engineers.
- Provide discussion on the potential effects of the development on neighbouring properties.
- Provide a comment on the geotechnical risks and opportunities.

1.2 SCOPE OF WORK

To achieve these objectives the following scope of work was undertaken:

- Review previous information of the site from our records.
- Preparation of work health and safety documentation for the works.
- Engagement of subcontractors.
- Clearance of testing locations for underground utilities by an accredited service locator.
- Pre-drilling through asphalt surfacing to allow advancement of in-situ tests.
- Drilling of three (3) relatively deep boreholes.
- Drilling of two (2) relatively shallow boreholes.
- Advancement of three (3) in-situ piezocone penetrometer tests (CPTu).
- Advancement of two (2) in-situ flat-plate dilatometer tests (DMT).
- Eight (8) dynamic cone penetrometer (DCP) tests.
- In-situ permeability testing.
- Geotechnical laboratory testing consisting of:
  - Three (3) Particle size distribution tests
  - Four (4) Aggressivity tests
  - Four (4) acid sulfate screen tests
- Analysis and preparation of a geotechnical investigation report.

1.3 PREVIOUS REPORTS

Previous investigations have been completed at the Site, primarily focused on groundwater quality related to the underground petroleum storage system (UPSS) and possible presence of polyfluoroalkyl substances (PFAS) within the groundwater. These reports include:

- *Preliminary Environmental Assessment and Soil Gas Survey, Robert Carr & Associates (RCA) Australia Pty Ltd, April 2004* (RCA, 2004).
- *Groundwater Monitoring Well Report, AECOM, August 2010* (AECOM, 2010)
- *Phase 1 and Phase 2 Environmental Site Assessment, Caltex Service Station (Site ID 22347), 38 Stockton Street, Nelson Bay NSW 2315 dated 18 November 2010, URS Australia Pty Ltd* (URS, 2010)
- *Site Remediation and Validation Report Former Caltex Nelson Bay Service Station* (22347), *38 Stockton Street, Nelson Bay, NSW (Reference: ENAUWARA02022AA_R04, dated 8 July 2015. Coffey Environments)* (Coffey, 2015)
- *Site Audit Report, Ramboll 2015, Former Caltex Service Station, 38 Stockton Street, Nelson Bay (Audit Number GN 485, dated July 15, 2015, Ramboll Environ)* (Ramboll, 2015)
- *Demolition Works Factual Report, Former Caltex Nelson Bay Service Station (22347s) 38 Stockton Street, Nelson Bay NSW (Reference: ENAUWARA02022AA_R06, dated 26 April 2016, Coffey Environments)* (Coffey, 2016).

A site history, description and summary of previous work completed at The Site is addressed in the contamination site suitability assessment report 754-NTLGE368007-AB dated 2 October 2024.

1.4 ARCHITECTURAL DRAWINGS

The current drawings for the proposed development were developed by Holdsworth Design Project No. 0159. An elevation schematic of the proposed structure is shown below in Figure 1.

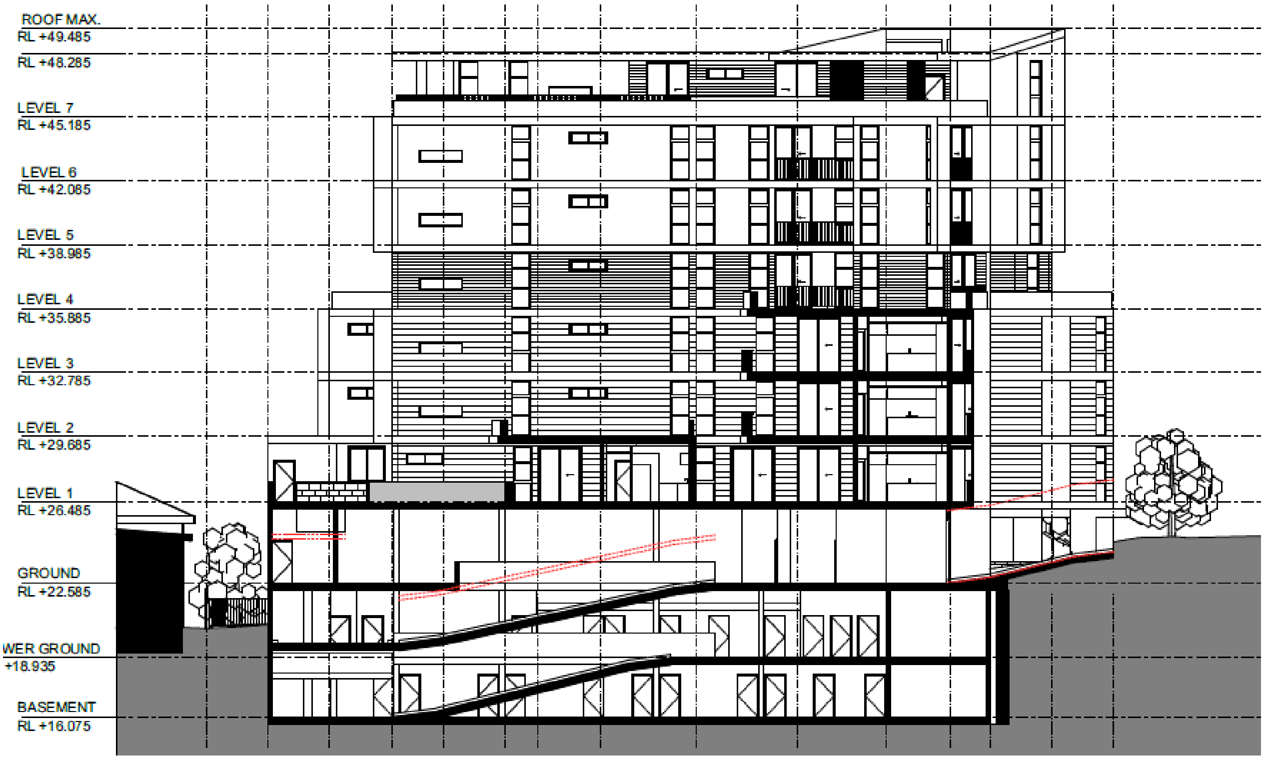


Figure 1. Elevation Schematic of the proposed superstructure showing the basement excavation (Extract from Holdsworth Design Section BB')

2. FIELDWORK

The geotechnical fieldwork was carried out between 16 and 17 of September 2024 and comprised of:

- Drilling three (3) deep boreholes (D-BH01, D-BH02 and D-BH03) to depths of 15.45m, 15.45m and 20.85m below the existing ground levels respectively with a track mounted drilling rig.
- Drilling two (2) boreholes (S-BH01, S-BH02) to depths of 3.0m below the existing ground level. Drilling during this event was undertaken with a track mounted drilling rig.
- Advancement of three (3) CPTu tests CPT01 to CPT03 to refusal in indurated sand at depths of 10.9m, 9.8m, and 10m below ground level respectively.
- Advancement of two (2) DMT (CPT01, CPT02) taken at every metre ranging between 1m to 6m.
- Conversion of two (2) boreholes to monitoring wells, D-BH01 (MW01) and D-BH02 (MW02).
- Eight (8) DCP tests to a depth of 3.0m except one location at DCP06 with refusal at 2.6m.

The boreholes and CPTu locations are shown in Figure 1 of the Drawings attached. In the field, boreholes, CPTu and DCP locations were set out and recorded using hand-held GPS to ± 5m accuracy. During the drilling, Standard Penetrometer Tests (SPTs) were conducted within boreholes D-BH01 and D-BH02 at approximately 1.5m depth intervals from about 1m depth to assess soil density/consistency and collect samples for logging and laboratory testing. Borehole D-BH03 was advanced without SPT testing to ascertain the depth to rock. The fieldwork was carried out in full-time presence of a Tetra Tech Engineering Geologist who observed the borehole drilling and CPTu testing, logged the encountered materials, recorded test results, noted groundwater levels, collected soil samples, and produced engineering logs of the boreholes. The engineering logs of the boreholes and CPT's are attached in Appendix A and B together with photos and explanation sheets defining the terms and symbols used in its preparation and summarised below in Table 1.

Table 1: Borehole, CPTu and DCP investigation locations

Test Identification	Depth (m BGL)	Easting (m MGA)(1)	Northing (m MGA)(1)
D-BH01 (MW01)	15.45	419658.3	6379077.7
D-BH02 (MW02)	15.45	419657.5	6379043.3
D-BH03	20.85	419677.1	6379058.2
S-BH01	3.0	419692.2	6379039.0
S-BH02	3.0	419680.0	6379072.1
GW-Well (MW03) (2)	15.1	419702.9	6379060.0
CPT1 (DMT01)	10.92	419662.2	6379061.8
CPT2 (DMT02)	9.78	419683.6	6379057.3
CPT3	10.02	419672.2	6379051.5
DCP 01	3.0	419696.8	6379067.3
DCP 02	3.0	419694.9	6379055.1
DCP 03	3.0	419692.4	6379040.0
DCP 04	3.0	419679.8	6379070.9
DCP 05	3.0	419677.6	6379042.2
DCP 06	2.6	419664.0	6379073.2
DCP 07	3.0	419664.7	6379061.0
DCP 08	3.0	419659.2	6379045.5

Notes: (1) Eastings and Northings were identified by Tetra Tech to ± 5m accuracy. (2) Existing monitoring well.

3. SITE CONDITIONS

3.1 REGIONAL GEOLOGY

The Site is located within gently undulating terrain, generally comprising windblown sand dunes. Based on the 1:250,000 Newcastle Geological Sheet and 1:100,000 Port Stephens Geological Sheet, the site is judged to be underlain by Quaternary aged alluvial deposits comprising gravel, sand, silt and clay, overlain by varying thicknesses of Aeolian (dune) sand deposits with variable indurated sands Waterloo Rock' Marine and freshwater deposits as described in the geological units below and shown in Figure 2.

- Qpbd (yellow) – Pleistocene dune: marine sand, indurated sands underlain by,
- Qpbdr (pale brown) – Pleistocene mantling dune: marine sand, indurated sands

Although not mapped within this site, there are nearby outcrops mapped as Neron Volcanics comprising Rhyodacitic Ignimbrite interbedded with tuffaceous sandstone and conglomerate which are inferred to underlie the surficial Pleistocene dune deposits.

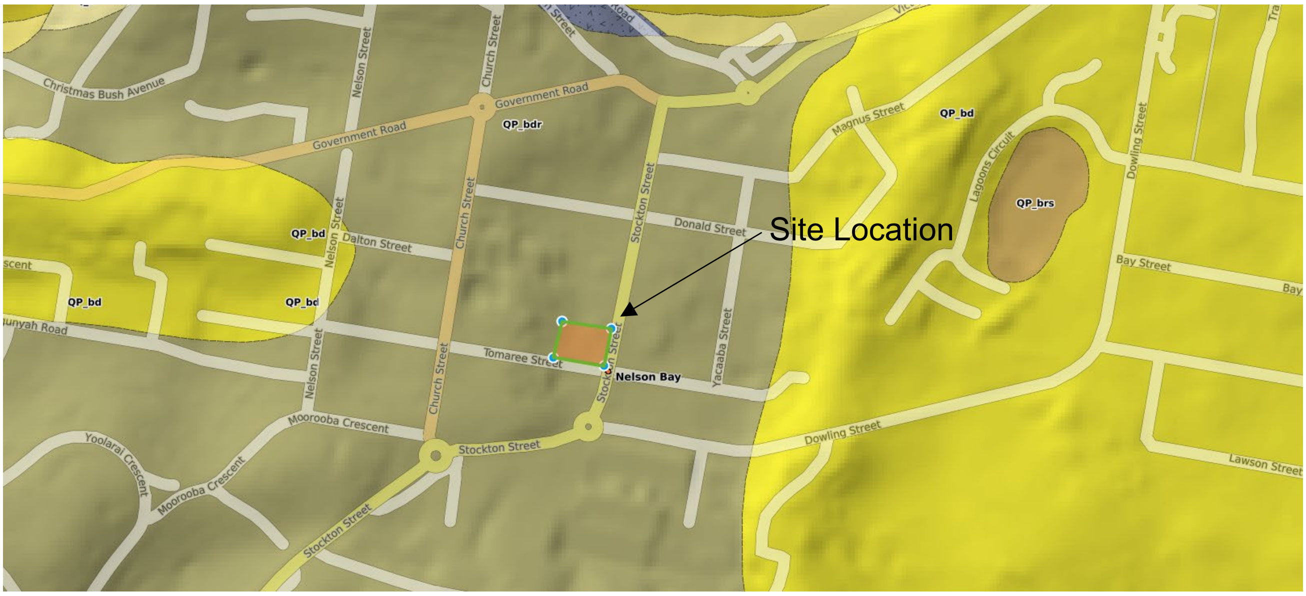


Figure 2. The site location in relation to the regional geology from MinView^{TM} 2020 (seamless geology)

3.2 SURFACE CONDITIONS

The Site is bounded by Stockton Street to the east and Tomaree Street to the south with a site area of approximately 2200m^{2}. Surrounding The Site are several medium density units and apartment developments generally less than about eight storeys in height. Across Stockton Street to the east is a two storey commercial building. While to the north are smaller two storey unit buildings.

The surface is currently covered in a gravel layer, with a few isolated shrubs. It is understood this gravel layer was placed in 2018 as part of the land being used as a car park. Some cut and fill across the site is evident. The general topography of the are falls at approximate 1 vertical to 7 horizontal to the northeast. Small retaining walls less than 1m in height are present on the north and southern boundaries as well as localised steepening of batters.

3.3 SUBSURFACE CONDITIONS

The subsurface conditions observed are provided in Table 2 with the distribution of geological units presented in Table 3. A geotechnical cross section has also been developed presented in the Drawings attached.

Table 2: Summary of geological units

Unit	Origin	Description
1a	Fill/ Road pavement	FILL: Sandy GRAVEL: fine to medium grained, grey subrounded to subangular with silt/clay.
1b	Fill	FILL: SAND: medium grained, mottled grey and dark grey, trace of rootlets, trace of fine to medium grained subangular gravel with silt/clay
1c	Fill/ reworked natural	FILL: SAND: medium grained, pale grey with silt/clay.
2a	Colluvium / former Topsoil	Silty SAND: fine to medium grained, dark brown to dark grey, trace of rootlets
3a	Aeolian	SAND: fine to medium grained, colours range from pale brown, pale orange and pale grey with silt.
3b	Indurated Sand	SAND: fine to medium grained, dark brown to red and orange
3c	Aeolian	SAND: fine to medium grained, pale brown to orange brown.
4a	Residual Soil	Not observed but anticipated to be SAND: fine to coarse grained

Table 3: Distribution of geological units

Unit		Depth to Base of geological unit (m)							
		D-BH01	**D-BH02**	**D-BH03**	**S-BH01**	**S-BH02**	**CPT1**	**CPT2**	**CPT3**
1a	Fill/ Road pavement	0.05	0.05	0.05	0.03	0.03	ND	ND	ND
1b	Fill	0.8	1.0	0.4	0.43	0.5	ND	ND	ND
1c	Fill/ reworked natural	2.75	2.0	2.0	>3.0	2.0	~2.0	~2.5	~3.0
2a	Colluvium / former topsoil	2.9	NE	NE	-	>3.0	NE	NE	NE
3a	Aeolian	7.2	10.3	ND	-	-	~9.0	~9.0	~8.5
3b	Indurated Sand	12.5	13.3	ND	-	-	>10.9	>9.8	>10.0
3c	Aeolian	>15.45	>15.45	19.5	-	-	-	-	-
4a	Residual Soil	-	-	20.85	-	-	-	-	-
5	Inferred Rock Level	-	-	>20.85	-	-	-	-	-
Notes	NE: Not encountered/ not observed ND: Not differentiated > : Limit of investigation								

3.4 GROUNDWATER

Groundwater was observed at a depth of 10.74m within D-BH01 (MW01), 12.46m within D-BH02 (MW02) and 11.55m within GW-Well (MW03) below the existing ground level. The wells were dipped approximately one week after the fieldwork was completed. Further discussion is provided in Section 9.

4. LABORATORY TESTING

4.1 MECHANICAL TESTING

The results from three particle size distribution tests taken at different depths are presented below in Figure 3.

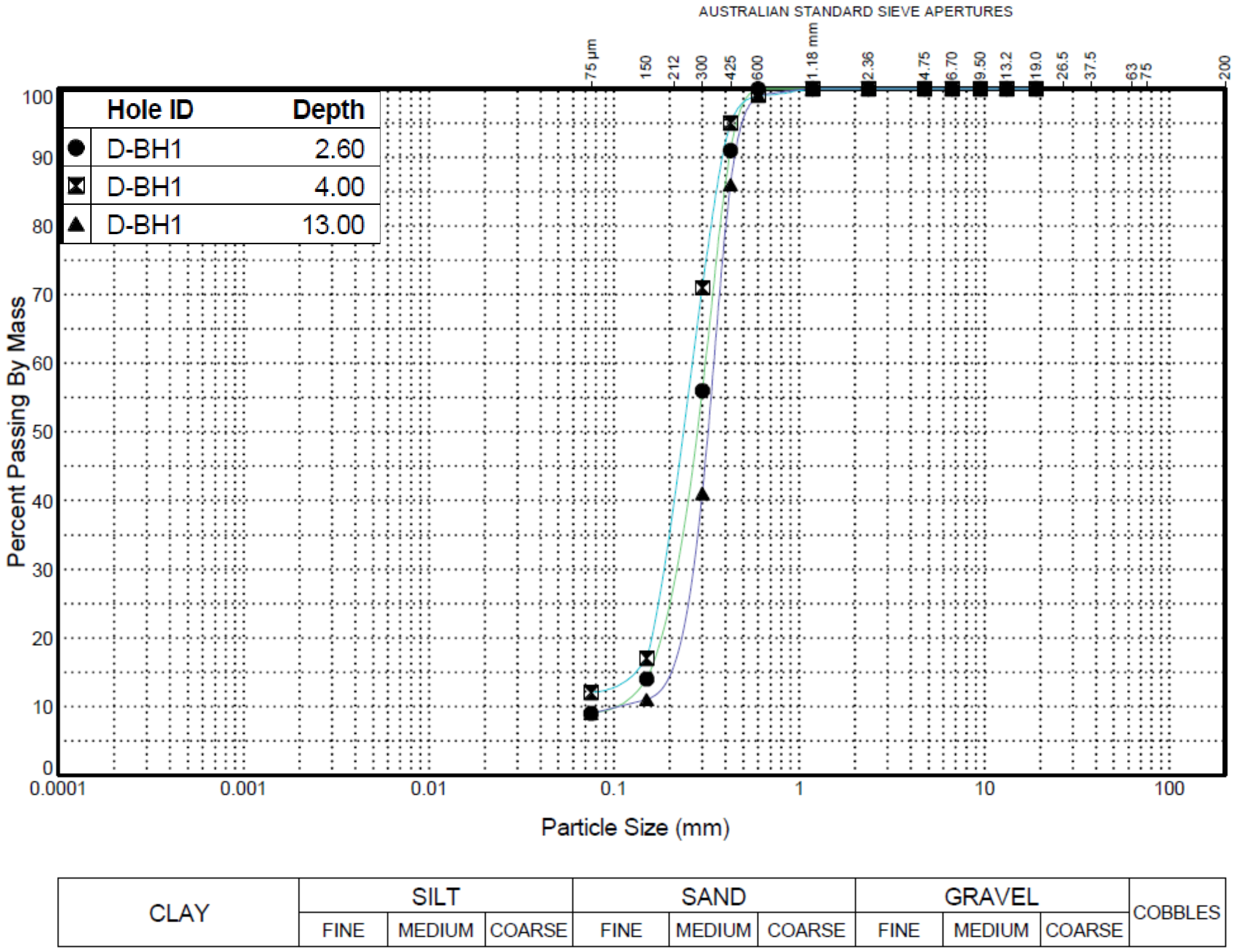


Figure 3. Summary of the particle size distribution testing

4.2 CHEMICAL TESTING

Four samples were collected for aggressivity testing, with results compared against the exposure classification in accordance with AS2159-2009 – Piling Design and Installation (Australian Standards, 2009). Results are provided in Table 4.

Table 4: Summary of aggressivity testing

ID	Depth and Unit (m)	Soil Condition	Chloride Cl (ppm)	Sulfate (SO_4) (ppm)	pH	Resistivity (ohm.cm)	Pile Type	Exposure Classification (1)
D-BH2	1.0 (Unit 1b)	B	19	87	5.4	12000	Concrete	Mild
							Steel	Non-Aggressive
D-BH2	2.5 (Unit 3a)	B	23	64	5.1	15000	Concrete	Mild
							Steel	Non-Aggressive
D-BH2	10.0 (Unit 3a)	A	<10	53	7.4	32000	Concrete	Non-Aggressive
							Steel	Non-Aggressive
D-BH2	15 (Unit 3c)	A	<10	<10	7.8	65000	Concrete	Non-Aggressive
							Steel	Non-Aggressive
Notes:	(1): Exposure classification in accordance with AS2159-2009 – Piling Design and Installation (Australian Standards, 2009)							

Acid Sulfate Soils field screen testing was undertaken with the results presented in Table 5.

Table 5. Summary of Acid Sulfate Soils field screen testing

ID	Depth and Unit (m)	pH-Field (pH)	pH-FOX (pH)	Reaction Rating
D-BH2	0.2 (Unit 1a)	10	10	4.0
D-BH2	1.0 (Unit 1b)	4.6	3.5	1.0
D-BH2	2.5 (Unit 1c)	4.9	4.4	1.0
D-BH2	7.0 (Unit 3a)	6.2	4.7	1.0
D-BH2	10.0 (Unit 3b)	5.5	5.0	1.0
D-BH2	15.0 (Unit 3c)	6.1	5.2	1.0

5. DISCUSSION

5.1 SIMPLIFIED GEOTECHNICAL MODEL

Based on the geotechnical investigation the following simplified geotechnical model with associated parameters has been developed and is presented in Table 6.

Table 6: Adopted Simplified Geotechnical Model and Design Parameters

Unit	Relative Density	Average Thickness (m)	γ (kN/m^3)	c' (kPa)	φ' (°)	E'_v (MPa)	υ	SPT Values (N)	Average q_c (MPa)
Compacted Controlled Fill	D	-	19	0	36	50	0.3	-	-
1 (a,b,c)	VL-L	2-2.75	18	0	25	5	0.3	1-10	1-2
2a	L	<0.5	18	0	30	10	0.3	8	4
3a	L-MD	7	19	0	33	30	0.3	8-20	5-20
3b	D-VD	3	20	0	38	80	0.3	40-R	30-40
3c	MD-D	7	19	0	34	40	0.3	19-22	-
4a	D	1.35	21	0	38	80	0.3	-	-

γ = Bulk unit weight
c' = Effective cohesion
φ' = Effective friction angle
E_v = Vertical Young's modulus
υ = Poisson's ratio
N = Standard penetrometer test
q_c = CPT cone resistance
R = Refusal

Notes: (1) Parameters have been summarised for the purpose of concept design. (2) The design will need to consider serviceability criteria using elastic parameters provided and reviewed as part of the detailed design (3) c' and φ' should be confirmed with triaxial testing for detailed design.

5.2 SITE CLASSIFICATION

A site classification was requested by the client, although this would not be applicable for the type of structure proposed, guidance can be taken from Residential Slabs and Footings AS2870 (Australian Standards, 2011) for indicative purposes only if a slab on ground with no connection to loaded columns is proposed. Based on the interpreted subsurface profile of a predominantly sand site, the site would be characterised as a Class A site provided that the building is founded below all fill which is anticipated due to the second basement level (lower ground being the first level). Elsewhere the site is classified Class P due to the presence of fill.

5.3 SITE PREPERATION

Site preparation suitable for structure or pavement support should consist of:

- Removing topsoil and/or deleterious material and transporting off site or to be used as landscaping.
- If site regrading is to occur (not anticipated), then approved clean sand fill should be compacted to a minimum density index of 70% (AS 1289 – 5.6.1) in maximum lifts of 300mm depth.
- The top 300mm of subgrade below general pavement areas should be compacted to a minimum density index of 75%.
- After excavation of the basement level in preparation for raft or basement floor slab, the upper 300mm of subgrade should be recompacted to minimum density index of 75% (AS 1289 – 5.6.1).

Earthworks should be planned, carried out and documented in accordance with recommendations outlined in AS3798-1996, '*Guidelines on Earthworks for Commercial and Residential Developments*'.

Based on the subsurface investigation, it is anticipated that the majority of excavated soils and spoil may be suitable for reuse as fill around the building area including reuse as backfill for retaining wall structures. The contamination site suitability assessment report by Tetra Tech Coffey (2024) 754-NTLGE368007-AB dated 2 October 2024 found The Site soils were suitable for reuse.

5.4 EXCAVATION CONDITIONS

It is anticipated that excavations for the proposed development would be achievable using a conventional excavator, which should be equipped with a smooth-walled (“gummy” bucket) to avoid over-disturbance of site soils below the required bulk excavation level. Based on the results of fieldwork, groundwater is not likely to be present on site within depths less than 7m below the existing ground surface. It is therefore expected bulk excavation to levels required for basement construction (varying from about 4m up to about 8m below existing ground level) are not likely to encounter inflows of water, although the risk of such inflow would increase with high rainfall.

Care must be taken not to cause relaxation of ground supporting nearby structures or infrastructure (e.g. roads and underground services) during excavations on The Site. Measures involving temporary earthworks batters or shoring systems should be employed while permanent retaining systems are constructed this can include using material on site to stabilise the toe of the unsupported excavation. Permanent and temporary earthworks batters may be formed at angles no steeper than 2.5H:1V and 2H:1V, respectively as shown in Table 7.

Table 7. Recommended unsupported excavation batter slopes for the geotechnical units

Unit / Material	Bulk Density (kN/m^3)	Temporary Batter	Permanent Batter
Unit 1 (a,b,c) (1)	18	2(H):1(V)	2.5(H):1(V)
Unit 2a/3a()	18/19	2(H):1(V)	2.5(H):1(V)

Notes: (1) Protection against erosion may be required.

5.5 AGGRESSIVITY TO BURIED STRUCTURAL ELEMENTS

Based on the exposure classification in accordance with AS2159-2009 – Piling Design and Installation (Australian Standards, 2009) the concrete elements would be classified as Mild and steel elements as Non-aggressive to Mild.

It is noted the above analysis does not consider acid sulfate soils. Should the water table be lowered additional design may be required as per AS2159-2009 – Piling Design and Installation (Australian Standards, 2009).

5.6 PRESENCE OF ACID SULFATE SOILS (ASS)

As discussed in the contamination site suitability assessment by Tetra Tech (Report 754-NTLGE368007-AB) a search of the NSW eSPADE V2.26, identifies the Site as not located within an area of Acid Sulfate Soils risk. Site elevation above 20mAHD precludes the area from coastal ASS risk.

The field screening test results presented in Table 5 indicated no presence of potential for ASS in the samples tested therefore an Acid Sulfate Management Plan should not be required for excavations on this site however Tetra Tech recommends the preparation of an Unexpected Finds Protocol (UFP) for inclusion in the Construction Environmental Management Plan.

5.7 EARTHQUAKE CLASSIFICATION AND ASSESSMENT

The Australian Standard for Earthquake loads AS 1170.4 (Australian Standards, 2024) provides guidance on the design of structures for earthquake loads. For Newcastle, AS 1170.4 quotes a Hazard Design Factor (Z) of 0.11. Based on the subsurface profile encountered during the geotechnical investigations and with reference to Section 3.1 of AS1170.4, the site sub-soil classification in accordance with AS 1170.4, Table 4.1 is considered Class C_e – Shallow soil site.

The site was also assessed for its liquefaction potential when subject to earthquake effects. The data from each CPT test was modelled through CPT liquefaction software (Cliq). The ground was modelled on the following assumptions:

- 6.0 Magnitude earthquake.
- Hazard Design Factor (Z) of 0.11 for the Newcastle region (AS 1170-2007, Table 3.2).
- Groundwater assumed to be at 8m below the existing ground level during the earthquake.
- Analysed for the top 20m of depth, as 20m below ground has no liquefication potential.

The outcome of the assessment showed the ground is almost certain to not liquify. Across the entire subsurface profile there is a low risk of liquefaction potential with details presented in Appendix D.

6. FOUNDATIONS

6.1.1 General

Results of the field investigation indicate the subsurface profile comprised of loose sands increasing density to dense to very dense at around 10m, before reducing to medium dense to dense from 13m. This profile is typical of the aeolian environment in which the site is situated, and subsurface profiles can vary significantly over short horizontal distances.

The architectural plans show that the finished floor level of the basement level is 16.075m, while the surrounding ground level ranges from 25.5m down to 20m. Therefore, the excavation may range from 4m at the northeastern corner up to approximately 8m at the southwestern corner.

Options for support of the proposed building are expected to include either a raft or piled raft footing, or deep footings (non-displacement or displacement piles) founded within medium dense sands or better.

6.1.2 Shallow Foundations

Shallow footings or mat foundations constructed over medium dense to dense sand material for Unit 3a are considered suitable. The allowable bearing capacity of shallow footings over this material may be proportioned for an estimated bearing capacity in the order of 200 kPa for small and isolated structures. Pad footings should be founded in the same unit of similar thickness if not differential settlement will need to be addressed. A combination of footing design such as pad and piles/pier foundations will also induce differential settlement, and this type of settlement will need to be assessed prior to construction. If a raft option is adopted this will need to be address as per section 6.1.3 and additional design considerations will be required.

6.1.3 Raft/Piled Raft Foundation

It has been our experience that at sites such as this one, where there are deep sand deposits, the use of a piled raft or raft footing system for buildings can result in reductions of cost and construction time, as compared to a conventional deep piled footing system.

A stiffened raft could be used to distribute loads more uniformly to the underlying soil. Whilst bearing capacity of a raft is not expected to be a problem, unacceptable total and differential settlement could be expected to occur under the raft footing. However further analysis provided at the detailed design stage could be undertaken to assess the feasibility of a raft foundation option. Differential settlements can be controlled by an adequately stiff raft; however, the additional thickness of concrete would need to be considered.

A piled raft foundation is a composite foundation system in which both the piles and the raft share the building loads. As compared to a conventional piled foundation, the number, diameter and/or length of piles can be reduced significantly by considering the contribution of the raft to the overall foundation capacity. The piles provide the majority of the foundation stiffness while the raft provides additional bearing capacity. Piles can be strategically located within the raft footing so that total and differential settlements are reduced to a tolerable limit. Piles in this footing system are mainly used to control the settlement.

Feasibility analysis and then a detailed analysis would be required if this option is to be further explored. These analyses would be carried out in close consultation with the Structural Engineer for the adopted parameters provided in Table 6. Tetra Tech Coffey would be pleased to assist with these analyses if required.

6.1.4 Deep Foundations

Design methodology

Pile foundations for the support of structures should be in accordance with AS2159-2009, a pile or pile group shall be proportioned so that the following inequality is satisfied:

$$R_{d,g} = \phi_g R_{d,ug} \geq E_d$$

where:

$R_{d,g}$ = Design geotechnical strength

ϕ_g = Geotechnical strength reduction factor

$R_{d,ug}$ = Design ultimate geotechnical strength

E_d = Design action effect

The use of limit state design requires that under the serviceability loading conditions performance of the foundation system be assessed, including pile group interaction effects, and that the resulting deflection does not exceed a tolerable limit. The tolerable limit may be for the purpose of meeting operational, durability, or aesthetic requirements. Working loads are considered in calculation utilising geotechnical strength parameters to which no reduction factor is applied.

For pier or pile support of the proposed apartment development, suitable options may include:

Non- Displacement

- Grout Injected Piles or CFA Piles founded in medium dense to dense sand.
- Cased bored piles. Uncased boreholes are not suitable due to the sandy soils.

Displacement

- Displacement cast-in-situ Screw Piles (e.g. 'Atlas' or 'Omega' piles) founded into medium dense to dense sand.
- Driven Precast Concrete piles into medium dense to dense sand.

Based on the investigation test results, loose to medium dense sand layers were encountered below the proposed basement level for approximately 3m in depth. Piles founded within this layer may achieve the adequate geotechnical strength for lightly loaded columns.

Piles should penetrate at least three pile diameters into the unit in order to adopt the bearing parameters associated with this unit, however would also be needed to be founded no less than 3 pile diameter above the base.

Based on the borehole and CPT results, it is recommended that piles are founded in medium dense sands or better. These may be proportioned for the Ultimate Geotechnical Strength Parameters presented in Table 8 in accordance with AS2159-1995, '*Piling Design and Installation*'.

Table 8. Preliminary Ultimate limit state parameters for pile design

Unit / Density	Pile Type	Ultimate End Bearing f_b (MPa)	Ultimate Side Adhesion f_s compression (6) (kPa)	Vertical Elastic Modulus (MPa)	Horizontal Elastic Modulus(5) (MPa)
Unit 3a(1) Loose to Medium Dense	Non Displacement(2)	1.5	15	30	20
	Displacement	3	35	40	30
Unit 3b(3)(4) Dense to Very Dense	Non Displacement(2)	6	50	80	60
	Displacement	10	100	100	75
Unit 3c Medium Dense to Dense	Non Displacement(2)	3	25	40	30
	Displacement	5	50	50	35

Notes:

(1) For Unit 3a the minimum embedment depth is approximately 3m from below the bulk excavation level.

(2) End bearing for Atlas piles and CFA would have a reduced end bearing capacity due to installation disturbance.

(3) Unit 3b has limited thickness and the potential for variability across the site.

(4) Due to the potential variability in the elevation and depth of Unit 3b and potential end bearing disturbance using CFA or similar methods it may be prudent to adopt the weaker underlying Unit 3c parameters. Also as it is necessary to found 3 pile diameters into the unit and 3 pile diameters above the base of unit, it may be impractical to found within this unit given it is only 3m in thickness.

(5) Horizontal elastic modulus taken as ~0.75 the vertical elastic modulus.

(6) Tension of the ultimate side adhesion is taken as ~0.7 of the value indicated for compression.

(7) The design should be checked for serviceability using limit state principals to control the settlement which should be reviewed once the working loads and settlement criteria are known as typically settlement will govern the design.

Please also note that the shaft friction along the pile is mobilised at a relatively small movement between the pile and soil i.e., at approximately, 0.5 to 1% of the diameter whereas the mobilisation of the ultimate base resistance requires movements in the order 5 to 10% of the pile diameter, which may be excessive for the structure to tolerate.

Therefore, when deciding on the pile lengths in sand (due to the possibility of soft pile toes) and in the absence of pile load tests, it would be prudent to attach more reliance to the shaft resistance as compared to the base or, in other words assess what proportion of the working load is carried by the shaft alone for a given / selected pile length. The bigger the proportion of the working load carried by the shaft the better the settlement performance is likely to be at service or working loads.

Should parameters for rock be required by the designer, additional investigation and testing would be required.

6.2 BASIC GEOTECHNICAL REDUCTION FACTOR

For limit state design, the design ultimate geotechnical pile capacity is derived by applying a basic geotechnical strength reduction factor (ϕ_{gb}) to the ultimate geotechnical pile capacity assessed using the ultimate shaft resistance and end bearing values shown in Table 8.

In accordance with AS2159-2009, ϕ_{gb} is dependent on an Average Risk Rating (ARR) which considers various geotechnical uncertainties, foundation system redundancy, construction supervision, quantity, and type of pile testing.

A preliminary assessment of ARR and ϕ_{gb} values has been conducted given the extent of geotechnical investigations performed and findings at this site, based on the following assumptions:

- Low redundancy foundation system.
- The design will be carried out by an experienced geotechnical professional using well-established and soundly based methods.
- Well established construction processes will be adopted, and detailed professional geotechnical supervision will be provided during pile construction.
- Performance of the supported structure is not monitored.

Based on our current understanding of the project and the above assumptions, the following preliminary values have been assessed:

- Average Risk Rating = 2.59
- Geotechnical strength reduction factor, ϕ_{gb}, 0.52 assuming a low redundancy system.

Testing may provide the additional confidence required to achieve a higher ϕ_{gb} value and more economical design. If no pile testing is conducted then the ϕ_{gb}, of 0.4 is to be adopted. Tetra Tech can review the final ϕ_{g} selection at the detailed design stage should we be involved at that stage.

6.3 GEOTECHNICAL REDUCTION FACTOR WITH THE BENEFIT OF PILE TESTING

Section 8.2.4 of AS 2159 provides guidance on the requirements for load testing of the basic geotechnical reduction factor is greater than 0.4. Table 8.2.4(A) provides pile testing requirements for serviceability for different ranges of risk rating. As per Table 8.2.4(A) for an ARR between 2.5 to 2.99 a minimum 1% of piles are to be tested for serviceability.

Increasing the percentage of pile load testing allows for a higher geotechnical strength reduction factor to be adopted in the design. The magnitude of the testing benefit depends, in addition to the percentage of load testing, on the type of piles and load testing carried out. For example, a static load test would provide a relatively greater testing benefit factored compared to dynamic pile load test. We have assessed and provided a range of geotechnical strength reduction factors for various percentages of load testing.

Table 9 provides an additional increase in the basic geotechnical strength reduction factor ϕ_{gb} based on the percentage of high-strain dynamic pile tests adopted. We suggest that a specialist and suitably experienced piling contractor should be consulted regarding appropriate pile load testing. Additional guidance for undertaking dynamic pile testing can be found in Appendix B of AS 2159-2009.

Table 9. Increase in geotechnical reduction factor based on percentage of pile high strain dynamic testing for ARR of 2.59

	Basic Geotechnical Strength reduction factor (ϕ_{gb})	Geotechnical Strength Reduction Factor – with testing benefit (ϕ_g)
No Testing	0.52	0.4 (no testing)
1 % pile testing	0.52	0.59
2 % pile testing	0.52	0.64
3 % pile testing	0.52	0.67
4 % pile testing	0.52	0.69
5 % pile testing	0.52	0.71

The range of geotechnical reduction factor with testing benefit has been assessed based on the following:

- We have assumed there are approximately 100 piles required for the project (i.e 1 dynamic pile test equates to 1%).
- CFA piling or other piling method technique will be adopted, observation and review of testing data to be conducted by Tetra Tech or suitably qualified Geotechnical Engineer.
- Specialist and suitably experienced piling contractors are consulted regarding appropriate pile load testing.

7. RETAINING WALLS AND EARTH RETAINING STRUCTURES

The design of retaining walls and shoring systems is geotechnically complex, and best carried out using soil-structure interaction analysis methods. For preliminary design, we have provided some preliminary retention design parameters. It is noted that the relative stiffness of the wall / shoring system will greatly influence the resulting earth pressure. The provided earth pressure coefficients are based off empirical methods and may not provide satisfactory solutions in some cases.

Limit state analyses in accordance with AS 4678 (Australian Standards, 2002) should be undertaken for the following failure mechanisms of soil support structures:

- Sliding within or at the base of the soil-support structures.
- Rotation of the soil-support structures.
- Rupture of structural elements such as nails, failure of connections between such elements.
- Global failure.
- Bearing failure.

The design of retaining walls should:

- Take into account loading from any proposed compaction of fill behind the wall.
- Provide adequate surface and subsurface drainage behind all retaining walls, possibly to incorporate free draining granular back fill to help prevent the build-up of hydrostatic pressures.
- Utilise materials that are not susceptible to deterioration.

Retaining walls should be founded in undisturbed natural soil below any fill, topsoil, slope wash or deleterious materials that is verifiably suited to support them in consideration of bearing capacity and settlement. The preliminary parameters for the retaining wall sections are provided Table 10.

Table 10. Preliminary design parameters for shoring and retaining walls

Unit	Relative Density	γ (kN/m^3)	φ' (°)	E'_v (MPa)	K_a	$K_o^{(1)}$	K_p
Compacted Controlled Fill	D	19	36	50	0.25	0.5	3.8
3a	L-MD	19	33	30	0.30	0.5	3.4
3b	D-VD	20	38	80	0.23	0.5	4.2
3c	MD-D	19	34	40	0.28	0.5	3.5
4a	D	21	38	80	0.23	0.5	4.2

γ = Bulk unit weight
φ' = Effective friction angle
E_v = Vertical Young’s modulus
K_a = Active earth pressure coefficient
K_o = At rest earth pressure coefficient
K_p = Passive earth pressure coefficient

Notes: (1) The K_o values are modified rather than in-situ values, assuming that at least a small amount of wall movement (0.1 to 0.3% of the wall height) is allowed to occur. If in-situ K_o values are required for detailed soil-structure analysis, specific testing will be required.

Active earth pressured coefficients should be adopted where wall movement of about 1% of the wall height can be tolerated. At rest pressure coefficients should be adopted where less movement can be tolerated. Retaining walls constructed as part of the building's basement will need to be designed for at rest (K_o) earth pressures due to their fixity. It should be noted that a well-constructed wall will still undergo movements of the order of 0.1% to 0.3% of the wall height where at rest pressures are adopted.

The global stability and deflection of the retaining system as well as the construction staging should be assessed by a Geotechnical Engineer once the proposed foundation system and construction methodology are identified. Notwithstanding the above comments on retention and stability, the contractor should comply with all statutory requirements for excavation and retention support.

7.1.1 Shallow retaining walls in sand

For shallow excavation, the design of cantilever retaining walls can assume a triangular earth pressure distribution to calculate earth pressures. The horizontal earth pressure profile for a triangular pressure distribution may be calculated using the following formula:

$$p= K (\gamma z + p_s)$$

where p = lateral earth pressure (kPa)

K = earth pressure coefficient, to be selected depending considering the amount of movement that can be tolerated.

γ = Bulk unit weight (kN/m^3) above water table.

z = depth below top of excavation (m)

p_s = design uniform surcharge pressure at ground level

7.1.2 Other retaining wall structures

Design of braced shoring or permanent retaining structure walls, which are constrained at several levels, can be based on a trapezoidal earth pressure distribution. Where retention of a multi-layered material profiles is required, modification of the distribution will be necessary.

Engineered retaining walls may be adopted and designed using the guidelines presented below. The design must include an assessment of global stability of the walls.

- For cantilever or gravity retaining walls, where movement is of little concern, a triangular lateral earth pressure distribution can be considered in the design of retaining wall, using the active effective lateral earth pressure coefficients of Table 10. This coefficient is proposed, assuming a horizontal backfill surface and no wall friction. If a sloping backfill is required, higher earth pressure parameters would apply.
- For lateral restraint the retaining walls must be embedded sufficiently into the Aeolian soils below bulk excavation level.
- If the top of retaining walls are to be restrained, such as by the floor slab of permanent structures, or if the wall are restraining areas which are sensitive to movement, the 'at rest' earth pressure coefficients (k_o), presented in Table 10 should be adopted for the above materials.
- Retaining walls must be designed and checked for both effective and total stress conditions (using the corresponding soil strength parameters)
- Any surcharge loads affecting the walls (such as inclined backfill surface, traffic loads, etc.) should be considered in the design.
- Drainage behind the wall should, as a minimum, comprise a geo-composite drain or geotextile wrapped gravel drain at the back of the wall that drains to a geotextile wrapped subsoil drain along the wall toe. The toe drain should discharge to the site storm water system to provide long term drainage behind retaining walls.

- Drainage measures as described below, if properly maintained, should reduce pore pressures at the back of the wall. However, pore pressures may still be generated at other points behind the wall. The design must incorporate an allowance for such pressures, and it would be prudent to assume hydrostatic pressure equals to one third of the wall height.
- Consideration should be given to the possibility of a hydrostatic pressure due to build-up of water behind the wall, unless permanent subsurface drainage can be provided.

The amount of movement that will be experienced by a retaining wall will depend on various factors including the earth pressures that exist, groundwater conditions and the excavation and construction sequence, including the tensioning sequence of anchors. Detailed soil structure interaction analysis should be carried out by Tetra Tech if movement-sensitive structures or neighbouring properties are located within close proximity to the retaining wall. In particular, if movement-sensitive services are located close to the excavation the design should consider the need to limit movements. In such situations the earth pressures calculated using coefficients in Table 10 may need to be modified to assess the impact on predicted movements.

Retaining walls not designed for full hydrostatic pressure should include free draining single size (10 mm single size gravel or coarser) aggregate backfill at the rear of the wall, with a slotted drainage pipe at the base of the backfill. The pipes should be designed to discharge water to a suitable drainage system. The backfill should be encapsulated within geotextile fabric.

Other retaining walls or shoring systems proposed will required detailed analysis from a Geotechnical Engineer. The ultimate lateral resistance of the piles should be factored in accordance with *AS2159-2009 Piling – Design and Installation* and geotechnical reduction factor applied. Tetra Tech would be pleased to assist with earth retaining structures or retaining wall design analyses if required.

8. PAVEMENTS

8.1 PRELIMINARY BASEMENT PAVEMENT SUBGRADE

The pavement thickness design option provided for a basement carpark is assumed to be a rigid, steel reinforced concrete structure pavement design with an unbound subbase of minimum 150mm thickness. It is understood that the basement will be utilised for general use and light vehicle parking with the occasional passing of service vehicles and delivery trucks. This option is only suitable if a piled foundation is adopted as a raft foundation will likely require a thicker concrete slab.

Tetra Tech adopted the following general assumptions for the purpose of the pavement design:

- Dimensions of the load wheels are 375mm width or less and are of pneumatic construction and 300kPa pressure.
- Width between the wheel loads is 1.7m
- The maximum axle load for a vehicle is 53kN.

A dynamic loading factor is applied to the axle loads for braking, cornering, and acceleration as per Section 7.2 of ‘*Austroads AGPT02’*. The ‘worst case’ combination loads of braking and cornering at the same time apply a dynamic factor, of 30%. The front axle load of the vehicle to be used for design is 70kN.

8.1.1 Design Subgrade CBR

Based on the findings from our geotechnical investigation, conditions at basement subgrade level are expected to be comprised of aeolian sand / indurated sand. The adopted CBR values is based on the conditions encountered in the boreholes and CPT testing.

The design subgrade CBR adopted for the assessment is 10%. The expected typical subgrade material is sand.

It should be noted that Tetra Tech assume appropriate drainage will form part of the construction works as the field moisture content of the subgrade in areas at the time of investigation will alter the effective CBR and modulus of the soil subgrade.

8.1.2 Traffic Loading

Tetra Tech have assumed average 80 passes or coverage per day for a vehicle / equipment over a given area of the rigid pavement area over a forty (40) year design life. The traffic loading for the basement carpark area is 1.2×10^6 No. passes of design vehicle / equipment.

The following inputs formed the analysis for the concrete pavement design:

- Design life = 40 years
- Daily passes or coverage (average over design life) = 80 Nos.
- Maximum front axle (single axle single tyre) load = 70kN
- Front wheel spacing = 1.7m
- Subgrade CBR in situ (minimum to 2m depth) = 10%
- Concrete Strength (f’c) = 40MPa
- Material factor k = 0.85

8.1.3 Preliminary Rigid Pavement Design Option

The rigid pavement design of the concrete basement area is to be of steel reinforced concrete construction of 40MPa f'c strength concrete and SL82 steel reinforcement based on a minimum steel percentage of 0.14% for jointed reinforced slabs from Cement Concrete & Aggregates Australia (CCAA) T48-2009 "Guide to industrial floors and pavements" (CCAA T48, 2009). The SL82 steel reinforcement and concrete strength should be assessed by the Structural Engineer. The slab centre thickness is to be minimum 150mm and an edge beam thickness or edge thickening of minimum 250mm (an additional 20mm construction tolerance for both centre slab and edge beam should be considered if future grinding is expected for the basement slab, based on RMS Supplement). The slab thicknesses (at centra and edge thickening) are minimum requirements at all locations. Design of joints, transition distance between slab centre and edge thickness and reinforcements is to be based on CCTA (2009) T48-2009. It is recommended to import some engineered fill (New Densely Graded Sub-base material or better) with a soaked CBR >30%, E > 200MPa and compact in a 300mm thick layer to 98% of standard compaction. A minimum 150mm thickness of granular subbase will be required below concrete slab. A debonding layer is to be placed between concrete base and granular subbase. It is permissible to excavate and replace existing subgrade to maintain desired finish level if needed.

8.1.4 Subgrade Preparation

It is recommended that subgrade preparation including verification of design subgrade CBR, fill placement and compaction be performed in the presence of a suitably experienced Geotechnical Engineer and the level of compaction checked by field density testing. It is expected that following excavation, some subgrade areas may need to be boxed out and replaced with engineered fill material (New Densely Graded Sub-base material (or better), soaked CBR >30%, E > 150MPa) to raise and / or replace the subgrade. Where engineered fill is required to raise or replace the subgrade, it should be placed and compacted. It is recommended that the following procedures be adopted for the preparation of subgrade for pavements, buildings and engineered fill:

- Earthworks should be planned, carried out and documented in accordance with the recommendations outlined in AS3798-2007 'Guidelines for Earthworks for Commercial and Residential Developments' and in accordance with local council Guidelines where applicable.
- Trafficking of the subgrade should be minimised or avoided (where possible) during construction to prevent the permanent deformation of the subgrade.

Specifically, the following recommendations are offered for the subgrade preparation of the proposed road:

- Excavation (where required) to subgrade formation level, with the spoiling of any deleterious material.
- Elimination of abrupt changes between subgrade conditions, by methods such as selective grading or mixing of material to provide a transition between material types, and moisture / density control of subgrade compaction.
- Proof rolling of the exposed subgrade with a heavy (minimum 10 tonne static) roller with any soft or weak areas detected to be excavated and replaced with a suitable compacted fill or subgrade replacement. Localised soft or weak areas detected during the proof rolling should be excavated and replaced with compacted fill/subgrade replacement comprising select subgrade filling having a soaked CBR > 10%. Proof rolling should be undertaken under the supervision of a suitably qualified engineer.
- Compaction of the subgrade filling or select should be to at least 100% of SMDD in layers of not greater than 250mm loose thickness and generally within ±2% of OMC.
- Protection of the subgrade to prevent any excessive wetting or drying.
- Subgrade preparation should be carried out during dry weather conditions, where possible. Provision should be made for effective diversion and removal of all surface water from the prepared subgrade from any source. The requirement for and extent of subgrade replacement should be confirmed by the Geotechnical Engineer at the time of construction.

9. GROUNDWATER ASSESSMENT

9.1 PIEZOMETER INSTALLATION

Three piezometers were installed on 17 September 2024. Piezometer completion details are presented in Table 12 and in the borehole logs attached in Appendix A. Piezometer locations are shown on the Drawings.

Table 11. Piezometer installation details

Piezometer ID	Eastings (m MGA)	Northings (m MGA)	Screen interval (m bgl)	Total Well depth (m bgl)	Screened lithology
D-BH01 (MW01)	419658.3	6379077.7	12.0-15.0	15.05	SAND
B-BH02 (MW02)	419657.5	6379043.3	12.0-15.0	15.05	SAND
GW-Well* (MW03)	419702.9	6379060.0	unknown	15.1	SAND

Note: m bgl – m below ground level   *GW-Well was an existing well from a previous investigation.

Groundwater levels were measured at the site piezometers on 26 September 2024 and are detailed in Table 13 below. Survey of the monitoring well would be required to ascertain the groundwater levels to AHD.

Table 12. Groundwater levels from gauging data

Piezometer ID	Date	Top of Casing (m bgl)	Total Well Depth (m bgl)	Depth to Water (m bgl)
MW01	26-09-24	0.5	15.05	10.74
MW02		0.5	15.05	12.46
MW03		0.5	15.1	11.55

Note: m bgl - m below ground level

It should be noted that fluctuations in groundwater levels can occur as a result of seasonal variations, temperature, rainfall and other similar factors, the influence of which may not be apparent at the time of this assessment. Data loggers may be installed in nominated wells to measure groundwater change over time.

9.2 ON-SITE STORMWATER BY INFILTRATION

Field assessment by means of boreholes encountered Aeolian sands. Infiltration testing comprised of 15 falling head permeability tests (three per well) to assess the estimated permeability of the existing site with a screened standpipe installed for each location. Typically for silty sands the hydraulic conductivity would range between 10^{-7} to 10^{-5} metres per second.

The hydraulic conductivity was estimated for the soils profile at the borehole locations using the Hvorslev method. This method was adopted over surface infiltration tests as a significant amount of material is expected to be removed during the bulk excavation. The test results are attached in Appendix E with a summary provided below in Table 14.

Table 13. Summary of estimated hydraulic conductivity test results

Test Location	Test Number	Screen Lithology	Hydraulic conductivity (K) (m/day)$^{(2)}$	Hydraulic conductivity (K) (m/sec)$^{(2)}$
S-BH01$^{(3)}$	1	Upper Aeolian	1.62	1.88×10^{-5}
S-BH01$^{(3)}$	2	Upper Aeolian	5.00	5.78×10^{-5}
S-BH01$^{(3)}$	3	Upper Aeolian	3.48	4.02×10^{-5}
S-BH02$^{(3)}$	4	Upper Aeolian	2.63	3.04×10^{-5}
S-BH02$^{(3)}$	5	Upper Aeolian	3.16	3.66×10^{-5}
S-BH02$^{(3)}$	6	Upper Aeolian	1.47	1.70×10^{-5}
D-BH01	1	Lower Aeolian	0.65	7.53×10^{-6}
D-BH01	2	Lower Aeolian	0.36	4.14×10^{-6}
D-BH01	3	Lower Aeolian	0.41	4.74×10^{-6}
D-BH02	4	Lower Aeolian	0.54	6.21×10^{-6}
D-BH02	5	Lower Aeolian	0.73	8.41×10^{-6}
D-BH02	6	Lower Aeolian	0.31	3.54×10^{-6}
GW Well$^{(1)}$	1	Unknown	2.22	2.57×10^{-5}
GW Well	2	Unknown	2.27	2.63×10^{-5}
GW Well	3	Unknown	1.71	1.98×10^{-5}

Notes: (1) GW Well is assumed to have a 3m screen at 12.0 to 15.0m below ground level however as the well is an existing well this screen is unknown. (2) hydraulic conductivity is a highly variable parameter with a coefficient of variation ranging between 100% to 300% and results within a single unit may vary by orders of magnitude depending on the subsurface conditions encountered. Permeability values may be higher or lower depending on the subsurface conditions encountered. (3) Permeability values for the shallow boreholes were taken from the initial gradient falling head curve.

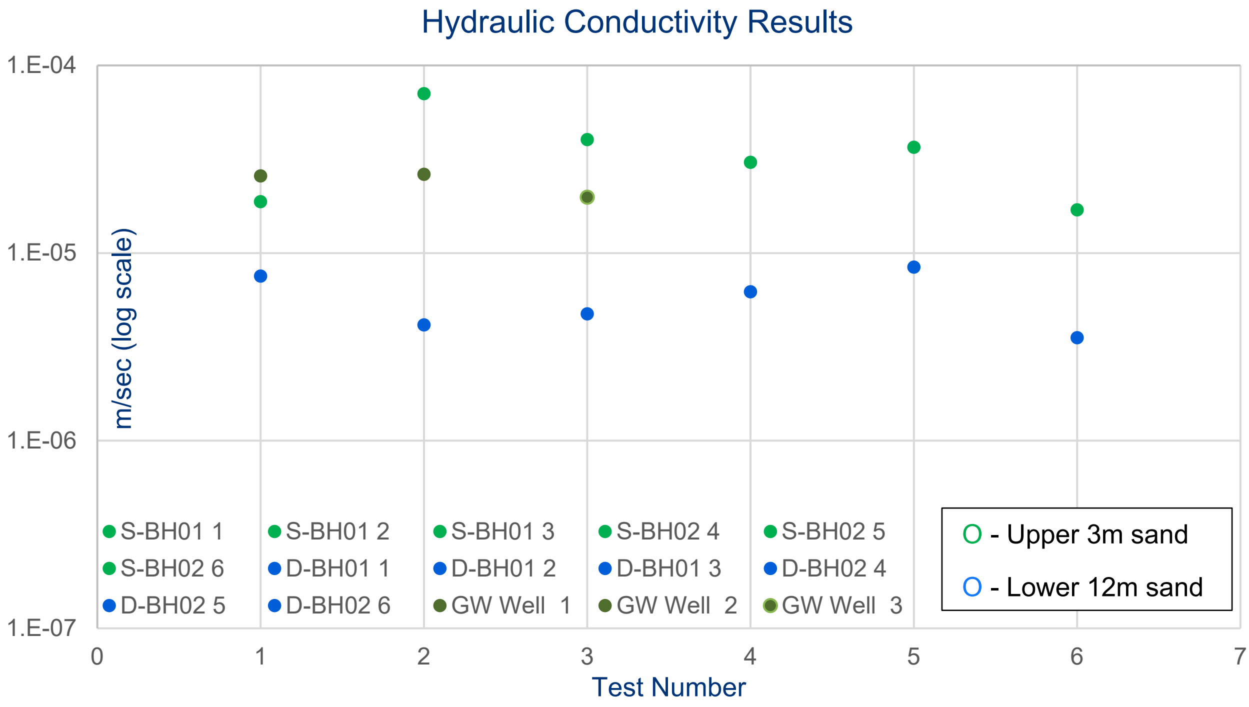


Figure 4. Hydraulic conductivity results

For design purposes, Port Stephens Council (PSC) usually requires that a reduction factor or factor of safety be applied to the nominated value(s) to obtain the long-term infiltration rate for design of on-site stormwater infiltration systems however this should be confirmed by PSC and the Civil Engineer.

As discussed in the contamination assessment by Tetra Tech (Report 754-NTLGE368007-AB), based on previous investigations, groundwater beneath the site flows in a northly direction.

10. RISK TO NEIGHBOURING PROPERTIES

10.1 GROUND DISTURBANCE

Careful examination should be made in the proposed building area for the presence of footings, service trenches and other subsurface structures associated with previous development of the lot. Where such structures are encountered, their removal and remediation should be documented by a Geotechnical Engineer at the time of bulk excavation.

10.2 VIBRATIONS DURING CONSTRUCTION

Care should be taken during site earthworks not to induce ground vibrations with the potential to cause damage to nearby structures. Excavation equipment should be selected to restrict such vibrations to levels that are within acceptable limits. Maximum tolerable vibration levels depend on the type of structure affected, its condition, and its proximity to the work area.

As a general guideline, a Peak Particle Velocity (PPV) of 5mm/sec is considered as the threshold at which a risk of ‘architectural’ damage exists to dwellings with plastered linings, and a PPV of 10-15mm/sec could present a risk of minor ‘structural’ damage to such dwellings. For the purposes of the above advice, ‘architectural’ damage is defined as damage that would not impair the function or use of a structure, and ‘structural’ damage as damage that would impair function or amenity. Tetra Tech are able to monitor ground vibration levels during construction work and to provide site-specific advice on levels of tolerable vibration, using equipment bolted to structures likely to be at risk.

10.3 EXCAVATION SUPPORT AND RETAINING WALLS

Excavation support and retaining walls should be designed using appropriate soil-structure interaction analysis methods and demonstrate that these elements provide the required restraint. Retaining walls or excavation support within the zone of influence to neighbouring properties can cause ground movement and settlements outside the development extents. Additionally appropriate instrumentation should be installed on retention structures to monitor any movements over time and reviewed as part of a monitoring plan.

10.4 DILAPIDATION REPORT

A dilapidation report / condition report documenting the condition of nearby residences / infrastructure that could conceivably be affected by construction activity is strongly recommended prior to the start of construction. A dilapidation study may also be completed during construction and once the construction has been completed to document to note changes, if any.

11. GEOTECHNICAL RISKS AND OPPORTUNITIES

11.1 RISKS

The following geotechnical risks have been identified:

- Deep retention, shoring or retaining walls for the basement excavation will require further detailed analysis.
- Very loose to loose aeolian sands present in the upper Units (Unit 1a,1b,1c and 2a).
- Temporary works including shoring of loose sands.
- Refusal of sheet piling works in the indurated sand layers and possible excess vibrations. Pre-drilling would likely be required to install sheet piles.
- Temporary retention may require ground anchors.
- Impacts on neighbouring properties including global stability and dilapidation assessments.
- Higher groundwater table than expected.

11.2 OPPORTUNITIES

Based on the site conditions the following opportunities have been identified:

- A raft foundation is likely possible within medium dense or better sands subject to detailed analysis
- Incorporation of the retaining walls into the raft foundation design
- Resue of on-site material for controlled fill if deemed suitable.
- Installation of data loggers to measure groundwater over time.

12. CLOSURE

The extent of testing associated with this assessment is limited to discrete test locations. Subsurface conditions away from the test locations may be different to those observed during testing and used as the basis of the site classifications contained in this report. If subsurface conditions encountered during excavation of the footing trenches differ from those given in this report further advice should be sought without delay.

Your attention is drawn to the attached document entitled “*Important Information about your Tetra Tech Coffey Report”*, which forms an integral part of this report.

13. REFERENCES

Australian Standards. (2002). AS 4678-2002. *Earth-retaining structures*.

Australian Standards. (2009). AS 2159-2009. *Piling - Design and installation*.

Australian Standards. (2011). AS 2870-2011. *Residential slabs and footings*.

Australian Standards. (2024). AS 1170.4-2024. *Structural design actions*.

CCAA T48. (2009). *Guide to Industrial Floors and Pavements - design, construction and specification.* Australia: Cement Concrete & Aggregates Australia.

Hvorslev, M. J. (1951). Time Lag and Soil Permeability in Ground-Water Observations. *Corps of Engineers, U.S. Army*, 1-57.

Kessler, J., & Ooserbaan, R. J. (1974). Determining hydraulic conductivity of soils: Drainage Principles and Applications Part III. *Internat. Inst. For Reclaimation & Imporvement, (ILRI).* Wageningen, The Netherlands.

LIMITATIONS

IMPORTANT INFORMATION ABOUT YOUR TETRA TECH COFFEY REPORT

As a client of Tetra Tech Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Tetra Tech Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Tetra Tech Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Tetra Tech Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Tetra Tech Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Tetra Tech Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Tetra Tech Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Tetra Tech Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Tetra Tech Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Tetra Tech Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Tetra Tech Coffey to work with other project design professionals who are affected by the report. Have Tetra Tech Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Tetra Tech Coffey for information relating to geoenvironmental issues.

Rely on Tetra Tech Coffey for additional assistance

Tetra Tech Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Tetra Tech Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Tetra Tech Coffey to other parties but are included to identify where Tetra Tech Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Tetra Tech Coffey closely and do not hesitate to ask any questions you may have.

DRAWINGS

PROJECT PROJECTION: EPSG:7856 - GDA2020 / MGA zone 56

0 5 10 15 20

Aerial imagery sourced from SIXMAPS, Department of Customer Service

drawn	KF
approved	MJ
date	03/10/2024
scale	1:250
page size	A3

TETRA TECH COFFEY

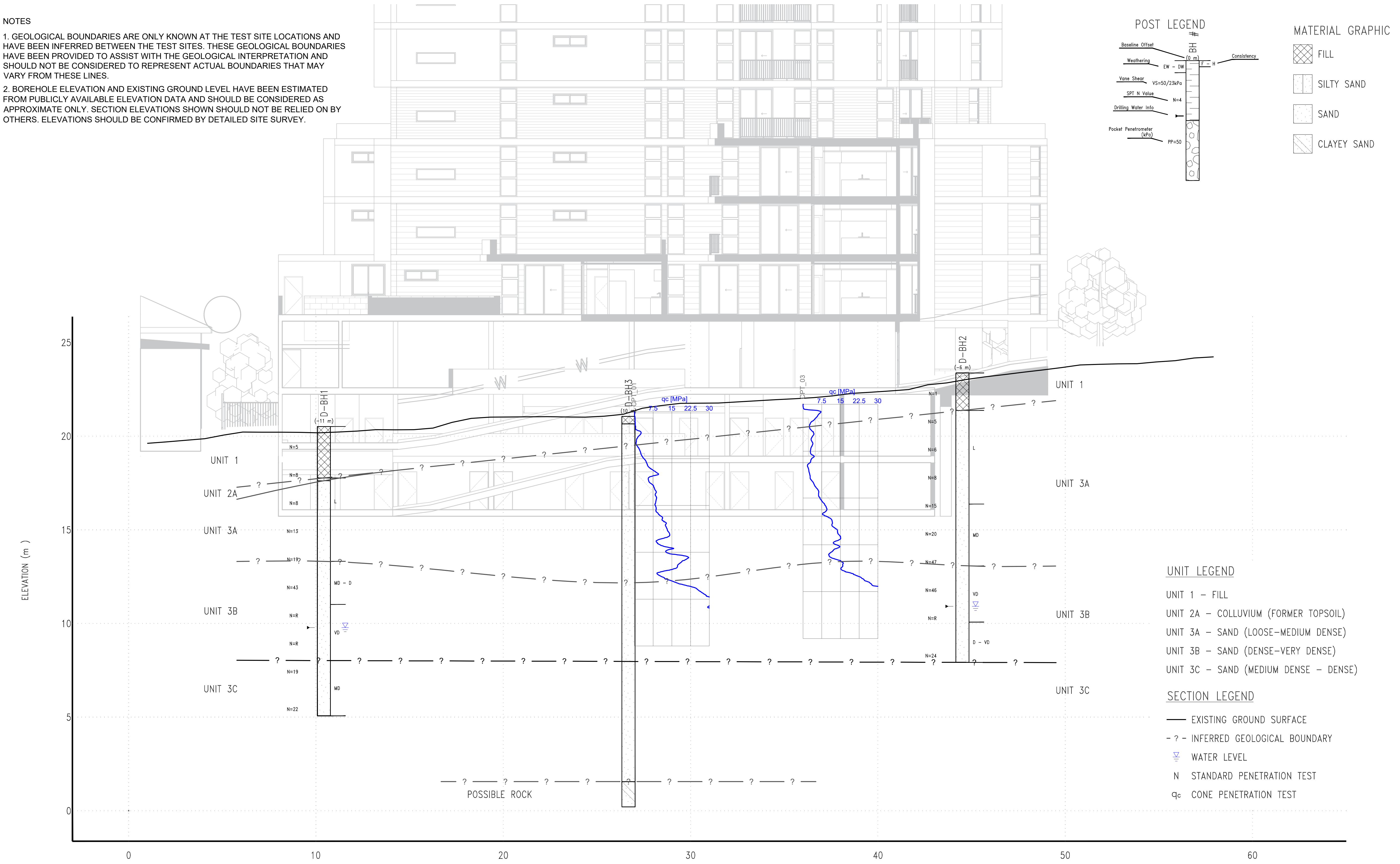
client:	ABC COMPANY
project:	Geotechnical Investigation Proposed Development 38 Stockton and 8A Tomaree Street, Nelson Bay
title:	GEOTECHNICAL INVESTIGATION SITE PLAN
project no:	754-NTLGE368007-AB
drawing no:	Drawing 1
rev:	A

PLOT DATE: 03/10/24 09:10 FILE: F:\F\GEOTECHNICS\1. JOBS\NTLGE\36\754-NTLGE368007 - 38 Stockton and 8A Tomaree Street, COHO\9. GIS\754-NTLGE368007.qgz

NOTES

1. GEOLOGICAL BOUNDARIES ARE ONLY KNOWN AT THE TEST SITE LOCATIONS AND HAVE BEEN INFERRED BETWEEN THE TEST SITES. THESE GEOLOGICAL BOUNDARIES HAVE BEEN PROVIDED TO ASSIST WITH THE GEOLOGICAL INTERPRETATION AND SHOULD NOT BE CONSIDERED TO REPRESENT ACTUAL BOUNDARIES THAT MAY VARY FROM THESE LINES.

2. BOREHOLE ELEVATION AND EXISTING GROUND LEVEL HAVE BEEN ESTIMATED FROM PUBLICLY AVAILABLE ELEVATION DATA AND SHOULD BE CONSIDERED AS APPROXIMATE ONLY. SECTION ELEVATIONS SHOWN SHOULD NOT BE RELIED ON BY OTHERS. ELEVATIONS SHOULD BE CONFIRMED BY DETAILED SITE SURVEY.



UNIT LEGEND

UNIT 1 – FILL

UNIT 2A – COLLUVIUM (FORMER TOPSOIL)

UNIT 3A – SAND (LOOSE–MEDIUM DENSE)

UNIT 3B – SAND (DENSE–VERY DENSE)

UNIT 3C – SAND (MEDIUM DENSE – DENSE)

SECTION LEGEND

— EXISTING GROUND SURFACE

– ? – INFERRED GEOLOGICAL BOUNDARY

▽ WATER LEVEL

N STANDARD PENETRATION TEST

qc CONE PENETRATION TEST

DRAFT

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revision

no.	description	drawn	approved	date
A	ORIGINAL ISSUE	-	-	-

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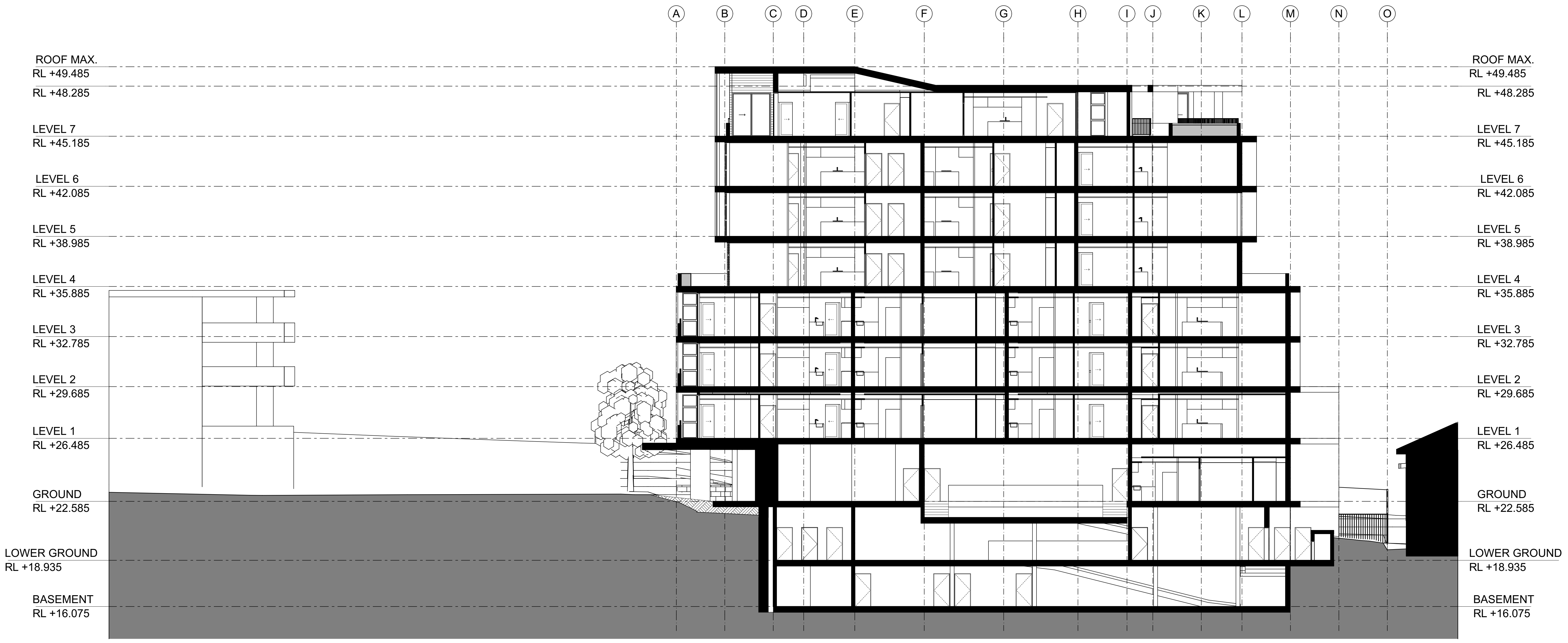
Scale (metres) 1:200

drawn	MJ
approved	-
date	03-10-2024
scale	AS SHOWN
original size	A3

TETRA TECH
COFFEY

client:	COHO PROPERTY PTY LTD
project:	PROPOSED RESIDENTIAL DEVELOPMENT 38 STOCKTON & 8A TOMAREE ST NELSON BAY
title:	SECTION BB
project no:	NTLGE368007
figure no:	2
rev:	A



SECTION ZZ

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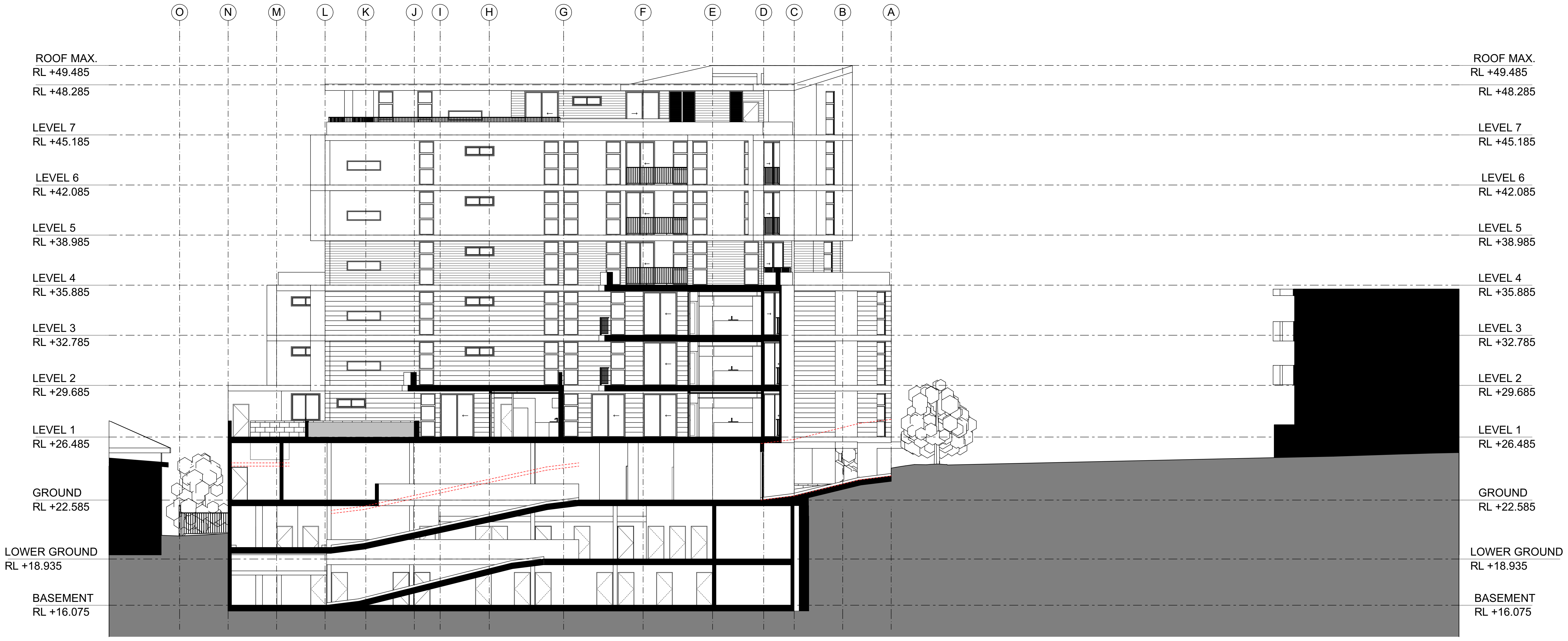

SECTION BB

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HOLDSWORTH DESIGN

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Nominated Architect: Brooke Holdsworth Registered Architect NSW 7453

NOT FOR CONSTRUCTION CERTIFICATE UNLESS ISSUED 'CONSTRUCTION CERTIFICATE' OR SIMILAR.
NOT FOR CONSTRUCTION UNLESS ISSUED 'FOR CONSTRUCTION' OR SIMILAR.
COMPLETION OF THE QUALITY ASSURANCE CHECKS IS VERIFICATION THAT THE DOCUMENT CONFORMS WITH THE REQUIREMENTS OF THE QUALITY PROJECT PLAN. WHERE THE QUALITY ASSURANCE CHECK IS INCOMPLETE THIS DOCUMENT IS PRELIMINARY FOR INFORMATION PURPOSES ONLY, OR SUCH PURPOSES AS STATED IN THE REVISION COLUMN.

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REV	DATE	COMMENTS	DRN	CHKD	VRFD
A	14.06.2024	CLIENT ISSUE	BH	BH	BH
B	03.07.2024	CLIENT & CONSULTANT ISSUE	BH	BH	BH
C	05.07.2024	DESIGN REVIEW PANEL - PRELIMINARY DISCUSSION	BH	BH	BH
D	16.08.2024	CLIENT ISSUE	BH	BH	BH
E	23.08.2024	CLIENT ISSUE	BH	BH	BH
F	29.08.2024	CLIENT & CONSULTANT ISSUE	BH	BH	BH
G	06.09.2024	CLIENT & CONSULTANT ISSUE	BH	BH	BH
H	15.09.2024	CLIENT & CONSULTANT ISSUE	BH	BH	BH

PROJECT : RESIDENTIAL APARTMENTS

CLIENT : COHO PROPERTY PTY. LTD.
COHO
PROPERTY

AUTHORITY : PORT STEPHENS COUNCIL

SITE : LOTS 781 & 782 DP 802108
CNR. STOCKTON & TOMAREE
STREETS
NELSON BAY NSW 2315

DRAWING : SECTIONS - SHEET 2

WORK IN FIGURED DIMENSIONS IN PREFERENCE TO SCALE. CHECK DIMENSIONS AND LEVELS ON SITE PRIOR TO THE ORDERING OF MATERIALS OR THE COMPLETION OF WORKSHOP DRAWINGS. IF IN DOUBT ASK. REPORT ALL ERRORS AND OMISSIONS.

FILENAME : 0159.240911_DA.pln
DATE PRINTED : 9/16/2024

DRAWN :	DATE :	SCALES :
BH	**SEP 24**	**1 : 200 @ A1**

PROJECT No :	PHASE :	DRAWING No :	REV :
0159	**DA**	**A-A504**	**H**

HOLDSWORTH DESIGN
ARCHITECTURE URBAN DESIGN INTERIOR DESIGN

APPENDIX A: ENGINEERING BOREHOLE LOGS

Soil Description Explanation Sheet (1 of 2)

DEFINITION:
In engineering terms soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

CLASSIFICATION SYMBOL & SOIL NAME

Soils are described in accordance with the Unified Soil Classification (UCS) as shown in the table on Sheet 2.

PARTICLE SIZE DESCRIPTIVE TERMS

NAME	SUBDIVISION	SIZE
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 µm to 2.36 mm
	medium	200 µm to 600 µm
	fine	75 µm to 200 µm

MOISTURE CONDITION

Dry	Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.
Moist	Soil feels cool and darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.
Wet	As for moist but with free water forming on hands when handled.

CONSISTENCY OF COHESIVE SOILS

TERM	UNDRAINED STRENGTH s_u (kPa)	FIELD GUIDE
Very Soft	<12	A finger can be pushed well into the soil with little effort.
Soft	12 – 25	A finger can be pushed into the soil to about 25mm depth.
Firm	25 – 50	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 – 100	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 – 200	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	>200	The surface of the soil can be marked only with the thumbnail.
Friable	–	Crumbles or powders when scraped by thumbnail.

DENSITY OF GRANULAR SOILS

TERM	DENSITY INDEX (%)
Very loose	Less than 15
Loose	15 – 35
Medium Dense	35 – 65
Dense	65 – 85
Very Dense	Greater than 85

MINOR COMPONENTS

TERM	ASSESSMENT GUIDE	PROPORTION OF MINOR COMPONENT IN:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: <5% Fine grained soils: <15%
With some	Presence easily detected by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 - 12% Fine grained soils: 15 - 30%

SOIL STRUCTURE

ZONING		CEMENTING	
Layers	Continuous across exposure or sample.	Weakly cemented	Easily broken up by hand in air or water.
Lenses	Discontinuous shape.	Moderately cemented	Effort is required to break up the soil by hand in air or water.
Pockets	Irregular inclusions of different material.		

GEOLOGICAL ORIGIN WEATHERED IN PLACE SOILS

Extremely weathered material	Structure and fabric of parent rock visible.
Residual soil	Structure and fabric of parent rock not visible.

TRANSPORTED SOILS

Aeolian soil	Deposited by wind.
Alluvial soil	Deposited by streams and rivers.
Colluvial soil	Deposited on slopes (transported downslope by gravity).
Fill	Man-made deposit. Fill may be significantly more variable between tested locations than naturally occurring soils.
Lacustrine soil	Deposited by lakes.
Marine soil	Deposited in ocean basins, bays, beaches and estuaries.

Soil Description Explanation Sheet (2 of 2)

SOIL CLASSIFICATION INCLUDING IDENTIFICATION AND DESCRIPTION

<table>
<tr><th colspan="7">FIELD IDENTIFICATION PROCEDURES USC
(Excluding particles larger than 60 mm and basing fractions on estimated mass)</th><th>USC</th><th>PRIMARY NAME</th></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS More than 50% of materials less than 63 mm is larger than 0.075 mm</td><td rowspan="16">(A 0.075 mm particle is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than 2.36 mm</td><td rowspan="2">CLEAN GRAVELS (Little or no fines)</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>GW</td><td>GRAVEL</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with more intermediate sizes missing.</td><td>GP</td><td>GRAVEL</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>SILTY GRAVEL</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>CLAYEY GRAVEL</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than 2.36 mm</td><td rowspan="2">CLEAN SANDS (Little or no fines)</td><td colspan="3">Wide range in grain sizes and substantial amounts of all intermediate sizes</td><td>SW</td><td>SAND</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing.</td><td>SP</td><td>SAND</td></tr>
<tr><td rowspan="2">SANDS WITH FINES (Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below).</td><td>SM</td><td>SILTY SAND</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below).</td><td>SC</td><td>CLAYEY SAND</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS More than 50% of material less than 63 mm is smaller than 0.075 mm</td><td></td><td colspan="4">IDENTIFICATION PROCEDURES ON FRACTIONS <0.2 mm</td><td></td><td></td></tr>
<tr><td rowspan="4">SILTS & CLAYS Liquid limit less than 50</td><th colspan="2">DRY STRENGTH</th><th>DILATANCY</th><th>TOUGHNESS</th><td></td><td></td></tr>
<tr><td colspan="2">None to Low</td><td>Quick to slow</td><td>None</td><td>ML</td><td>SILT</td></tr>
<tr><td colspan="2">Medium to High</td><td>None</td><td>Medium</td><td>CL</td><td>CLAY</td></tr>
<tr><td colspan="2">Low to medium</td><td>Slow to very slow</td><td>Low</td><td>CL</td><td>ORGANIC SILT</td></tr>
<tr><td rowspan="3">SILTS & CLAYS Liquid limit greater than 50</td><td colspan="2">Low to medium</td><td>Slow to very slow</td><td>Low to medium</td><td>MH</td><td>SILT</td></tr>
<tr><td colspan="2">High</td><td>None</td><td>High</td><td>CH</td><td>CLAY</td></tr>
<tr><td colspan="2">Medium to High</td><td>None</td><td>Low to medium</td><td>OH</td><td>ORGANIC CLAY</td></tr>
<tr><td colspan="3">HIGHLY ORGANIC SOILS</td><td colspan="4">Readily identified by colour, odour, spongy feel and frequently by fibrous texture.</td><td>PT</td><td>PEAT</td></tr>
<tr><td colspan="9">● Low plasticity – Liquid Limit w_L less than 35%. ● Medium plasticity – w_L between 35% and 50%. ● High plasticity – w_L greater than 50%.</td></tr>
</table>

COMMON DEFECTS IN SOIL

TERM	DEFINITION	DIAGRAM
PARTING	A surface or crack across which the soil has little or no tensile strength. Parallel or sub parallel to layering (eg bedding). May be open or closed.	
JOINT	A surface or crack across which the soil has little or no tensile strength but which is not parallel or sub parallel to layering. May be open or closed. The term 'fissure' may be used for irregular joints <0.2 m in length	
SHEARED ZONE	Zone in clayey soil with roughly parallel near planar, curved or undulating boundaries containing closely spaced, smooth or slickensided, curved intersecting joints which divide the mass into lenticular or wedge shaped blocks.	
SHEARED SURFACE	A near planar curved or undulating, smooth, polished or slickensided surface in clayey soil. The polished or slickensided surface indicates that movement (in many cases very little) has occurred along the defect.	

TERM	DEFINITION	DIAGRAM
SOFTENED ZONE	A zone in clayey soil, usually adjacent to a defect in which the soil has a higher moisture content than elsewhere.	
TUBE	Tubular cavity. May occur singly or as one of a large number of separate or inter-connected tubes. Walls often coated with clay or strengthened by denser packing of grains. May contain organic matter.	
TUBE CAST	Roughly cylindrical elongated body of soil different from the soil mass in which it occurs. In some cases the soil which makes up the tube cast is cemented.	
INFILLED SEAM	Sheet or wall like body of soil substance or mass with roughly planar to irregular near parallel boundaries which cuts through a soil mass. Formed by infilling of open joints.	

Rock Description Explanation Sheet (1 of 2)

The descriptive terms used by Coffey are given below. They are broadly consistent with Australian Standard AS1726-1993.

DEFINITIONS:	Rock substance, defect and mass are defined as follows:
Rock Substance	In engineering terms rock substance is any naturally occurring aggregate of minerals and organic material which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Effectively homogenous material, may be isotropic or anisotropic.
Defect	Discontinuity or break in the continuity of a substance or substances.
Mass	Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

SUBSTANCE DESCRIPTIVE TERMS:

ROCK NAME	Simple rock names are used rather than precise geological classification.
PARTICLE SIZE	Grain size terms for sandstone are:
Coarse grained	Mainly 0.6mm to 2mm
Medium grained	Mainly 0.2mm to 0.6mm
Fine grained	Mainly 0.06mm (just visible) to 0.2mm
FABRIC	Terms for layering of penetrative fabric (eg. bedding, cleavage etc.) are:
Massive	No layering or penetrative fabric.
Indistinct	Layering or fabric just visible. Little effect on properties.
Distinct	Layering or fabric is easily visible. Rock breaks more easily parallel to layering of fabric.

CLASSIFICATION OF WEATHERING PRODUCTS

Term	Abbreviation	Definition
Residual Soil	**RS**	Soil derived from the weathering of rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely Weathered Material	**XW**	Material is weathered to such an extent that it has soil properties, ie, it either disintegrates or can be remoulded in water. Original rock fabric still visible.
Highly Weathered Rock	**HW**	Rock strength is changed by weathering. The whole of the rock substance is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Some minerals are decomposed to clay minerals. Porosity may be increased by leaching or may be decreased due to the deposition of minerals in pores.
Moderately Weathered Rock	**MW**	The whole of the rock substance is discoloured, usually by iron staining or bleaching , to the extent that the colour of the fresh rock is no longer recognisable.
Slightly Weathered Rock	**SW**	Rock substance affected by weathering to the extent that partial staining or partial discolouration of the rock substance (usually by limonite) has taken place. The colour and texture of the fresh rock is recognisable; strength properties are essentially those of the fresh rock substance.
Fresh Rock	**FR**	Rock substance unaffected by weathering.

Notes on Weathering:
AS1726 suggests the term "Distinctly Weathered" (DW) to cover the range of substance weathering conditions between XW and SW. For projects where it is not practical to delineate between HW and MW or it is judged that there is no advantage in making such a distinction. DW may be used with the definition given in AS1726.
Where physical and chemical changes were caused by hot gasses and liquids associated with igneous rocks, the term "altered" may be substituted for "weathering" to give the abbreviations XA, HA, MA, SA and DA.

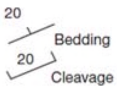
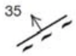
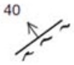
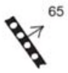
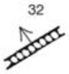
ROCK SUBSTANCE STRENGTH TERMS

Term	Abbre-viation	Point Load Index, $I_{s(50)}$ (MPa)	Field Guide
Very Low	**VL**	Less than 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; pieces up to 30mm thick can be broken by finger pressure.
Low	**L**	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show with firm bows of a pick point; has a dull sound under hammer. Pieces of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	**M**	0.3 to 1.0	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	**H**	1 to 3	A piece of core 150mm long by 50mm can not be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High	**VH**	3 to 10	Hand specimen breaks after more than one blow of a pick; rock rings under hammer.
Extremely High	**EH**	More than 10	Specimen requires many blows with geological pick to break; rock rings under hammer.

Notes on Rock Substance Strength:
In anisotropic rocks the field guide to strength applies to the strength perpendicular to the anisotropy. High strength anisotropic rocks may break readily parallel to the planar anisotropy.
The term "extremely low" is not used as a rock substance strength term. While the term is used in AS1726-1993, the field guide therein makes it clear that materials in that strength range are soils in engineering terms.
The unconfined compressive strength for isotropic rocks (and anisotropic rocks which fall across the planar anisotropy) is typically 10 to 25 times the point load index Is(50). The ratio may vary for different rock types. Lower strength rocks often have lower ratios than higher strength rocks.

Rock Description Explanation Sheet (2 of 2)

COMMON DEFECTS IN ROCK MASSES

Term	Definition	Diagram	Map Symbol	Graphic Log (Note 1)
Parting	A surface or crack across which the rock has little or no tensile strength. but which is not parallel or sub parallel to layering or planar anisotropy in the rock substance. May be open or closed.			
Joint	A surface or crack across which the rock has little or no tensile strength. but which is not parallel or sub parallel to layering or planar anisotropy in the rock substance. May be open or closed.			
Sheared Zone (Note 3)	Zone of rock substance with roughly parallel near planar, curved or undulating boundaries cut by closely spaced joints, sheared surfaces or other defects. Some of the defects are usually curved and intersect to divide the mass into lenticular or wedge shaped blocks.			
Sheared Surface (Note 3)	A near planar, curved or undulating surface which is usually smooth, polished or slickensided.			
Crushed Seam (Note 3)	Seam with roughly parallel almost planar boundaries, composed of disoriented, usually angular fragments of the host rock substance which may be more weathered than the host rock. The seam has soil properties			
Infilled Seam	Seam of soil substance usually with distinct roughly parallel boundaries formed by the migration of soil into an open cavity or joint, infilled seams less than 1mm thick may be described as veneer or coating on joint surface.			
Extremely Weathered Seam	Seam of soil substance, often with gradational boundaries. Formad by weathering of the rock substance in place.			

Notes on Defects:

1. Usually borehole logs show the true dip of defects and face sketches and sections the apparent dip.
2. Partings and joints are not usually shown on the graphic log unless considered significant.
3. Sheared zones, sheared surfaces and crushed seams are faults in geological terms.

DEFECT SHAPE TERMS

Planar	The defect does not vary in orientation
Curved	The defect has a gradual change in orientation
Undulating	The defect has a wavy surface
Stepped	The defect has one or more well defined steps
Irregular	The defect has many sharp changes of orientation

Note: The assessment of defect shape is partly influenced by the scale of the observation.

ROUGHNESS TERMS

Slickensided	Grooved or striated surface, usually polished
Polished	Shiny smooth surface
Smooth	Smooth to touch. Few or no surface irregularities
Rough	Many small surface irregularities (amplitude generally less than 1mm). Feels like fine to coarse sand paper.
Very Rough	Many large surface irregularities (amplitude generally more than 1mm). Feels like, or coarser than very coarse sand paper.

COATING TERMS

Clean	No visible coating
Stained	No visible coating but surfaces are discoloured
Veneer	A visible coating of soil or mineral, too thin to measure; may be patchy
Veneer	A visible coating up to 1mm thick. Thicker soil material is usually described using appropriate defect terms (eg, infilled seam). Thicker rock strength material is usually described as a vein.

BLOCK SHAPE TERMS

Blocky	Approximately equidimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section

72710 / 07-06

TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID.	***D-BH1***
sheet:	1 of 2
project no.	***754-NTLGE368007***
date started:	***16 Sep 2024***
date completed:	***16 Sep 2024***
logged by:	***KF***
checked by:	***MJ***

client: ***COHO Property Pty Ltd***

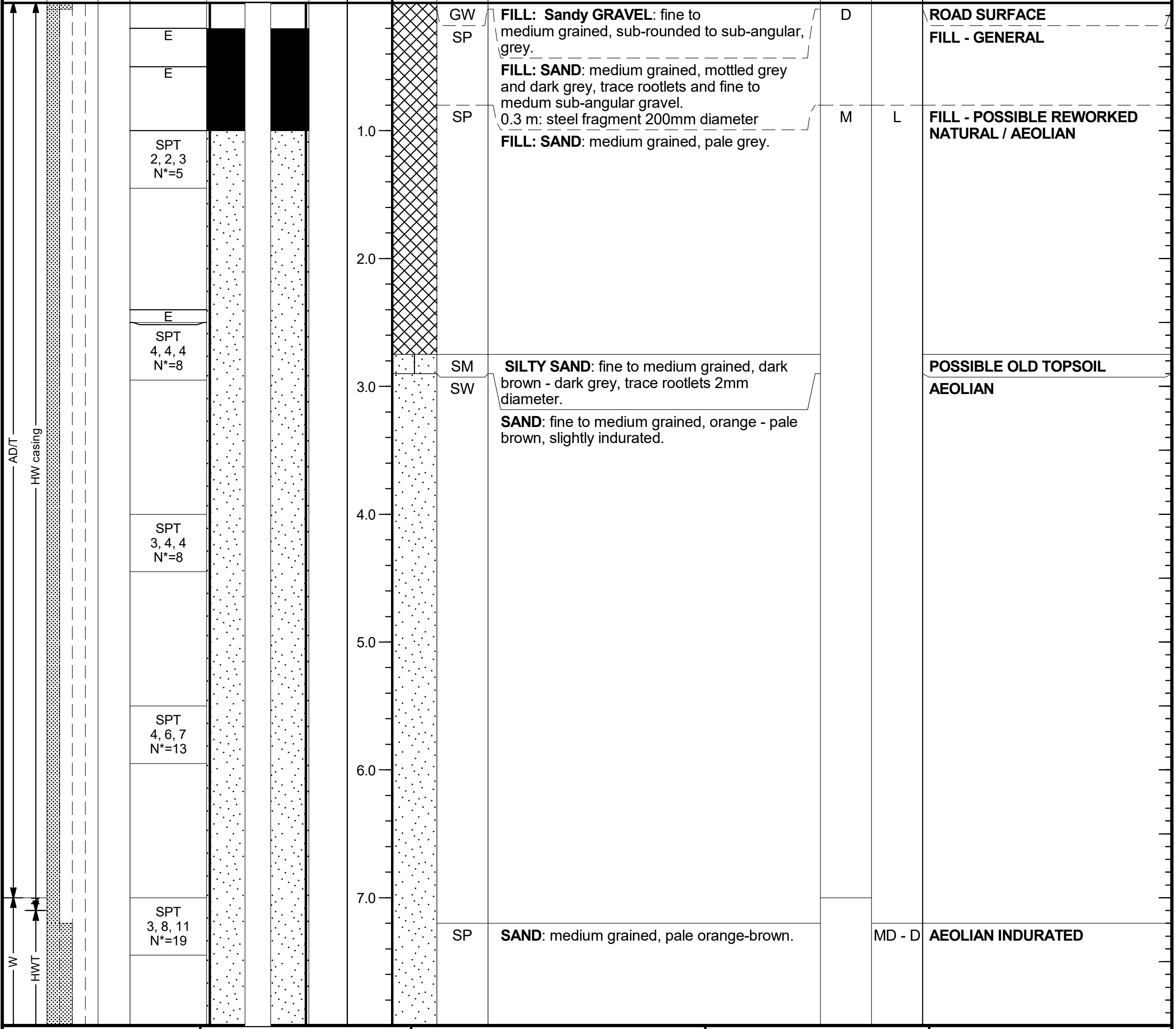
principal: ***COHO Property Pty Ltd***

project: ***Proposed Development Nelson Bay***

location: ***38 Stockton and 8A Tomaree Street, Nelson Bay***

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information | **well details** | **material substance**



method & support	samples & field tests	depth (m)
AD/T, HW casing	E	
	E	
	SPT 2, 2, 3 N*=5	1.0
		2.0
	E	
	SPT 4, 4, 4 N*=8	3.0
	SPT 3, 4, 4 N*=8	4.0
		5.0
	SPT 4, 6, 7 N*=13	6.0
		7.0
W, HWT	SPT 3, 8, 11 N*=19	

soil group symbol	material description **SOIL NAME**: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	structure and additional observations
GW	**FILL: Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, grey.	D		**ROAD SURFACE**
SP	**FILL: SAND**: medium grained, mottled grey and dark grey, trace rootlets and fine to medum sub-angular gravel. 0.3 m: steel fragment 200mm diameter			**FILL - GENERAL**
SP	**FILL: SAND**: medium grained, pale grey.	M	L	**FILL - POSSIBLE REWORKED NATURAL / AEOLIAN**
SM	**SILTY SAND**: fine to medium grained, dark brown - dark grey, trace rootlets 2mm diameter.			**POSSIBLE OLD TOPSOIL**
SW	**SAND**: fine to medium grained, orange - pale brown, slightly indurated.			**AEOLIAN**
SP	**SAND**: medium grained, pale orange-brown.		MD - D	**AEOLIAN INDURATED**

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud N nil
C casing

penetration
1 2 3 no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition
D dry
M moist
W wet
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

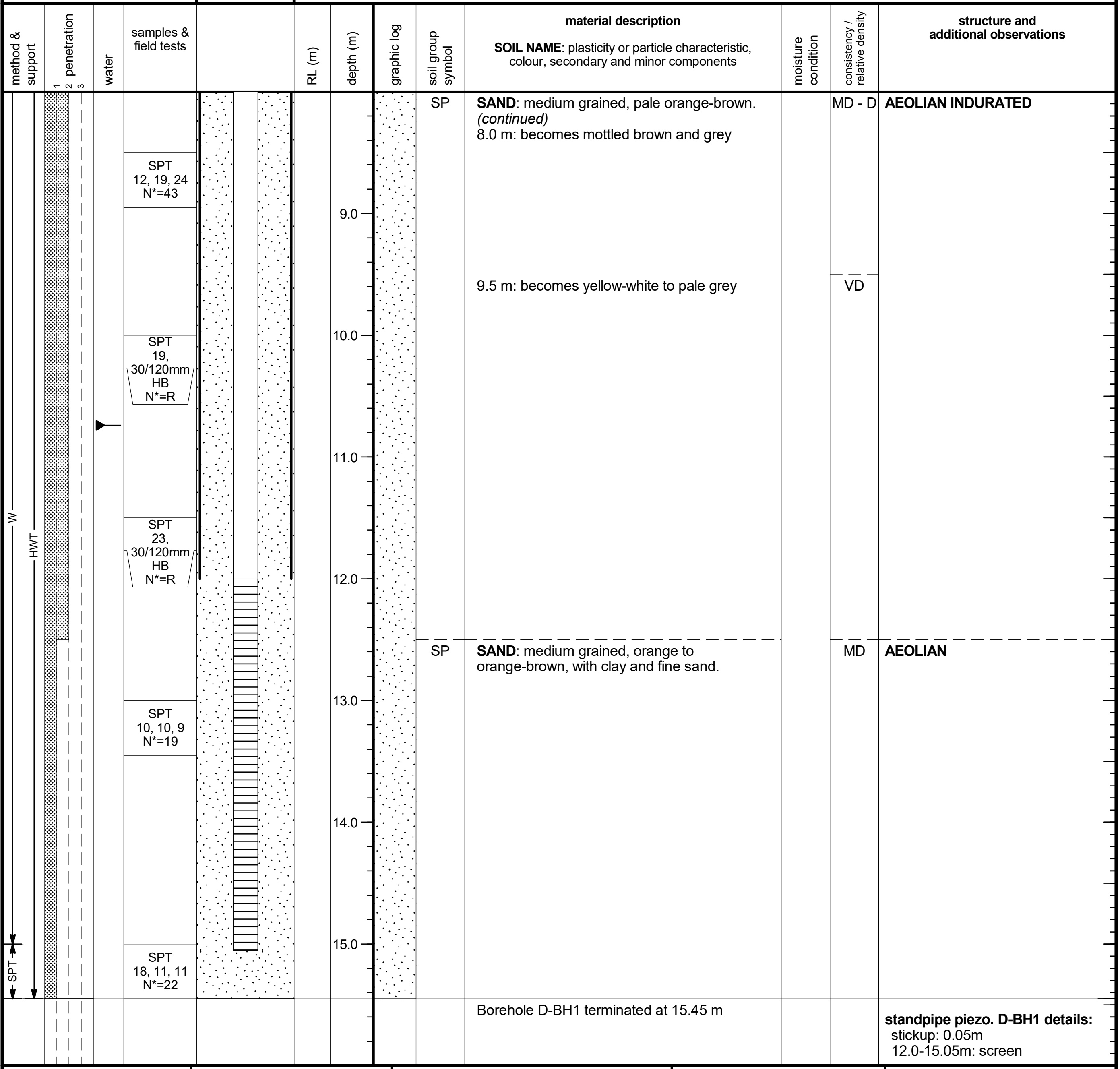
TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID.	**D-BH1**
sheet:	2 of 2
project no.	**754-NTLGE368007**
client:	**COHO Property Pty Ltd**
principal:	**COHO Property Pty Ltd**
project:	**Proposed Development Nelson Bay**
location:	**38 Stockton and 8A Tomaree Street, Nelson Bay**
date started:	**16 Sep 2024**
date completed:	**16 Sep 2024**
logged by:	**KF**
checked by:	**MJ**

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information | **well details** | **material substance**



samples & field tests	soil group symbol	material description (SOIL NAME: plasticity or particle characteristic, colour, secondary and minor components)	consistency / relative density	structure and additional observations
SPT 12, 19, 24 N*=43	SP	**SAND**: medium grained, pale orange-brown. *(continued)* 8.0 m: becomes mottled brown and grey	MD - D	**AEOLIAN INDURATED**
SPT 19, 30/120mm HB N*=R		9.5 m: becomes yellow-white to pale grey	VD	
SPT 23, 30/120mm HB N*=R				
SPT 10, 10, 9 N*=19	SP	**SAND**: medium grained, orange to orange-brown, with clay and fine sand.	MD	**AEOLIAN**
SPT 18, 11, 11 N*=22				

Borehole D-BH1 terminated at 15.45 m

standpipe piezo. D-BH1 details:
stickup: 0.05m
12.0-15.05m: screen

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
N nil
C casing

penetration
1 2 3
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition
D dry
M moist
W wet
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

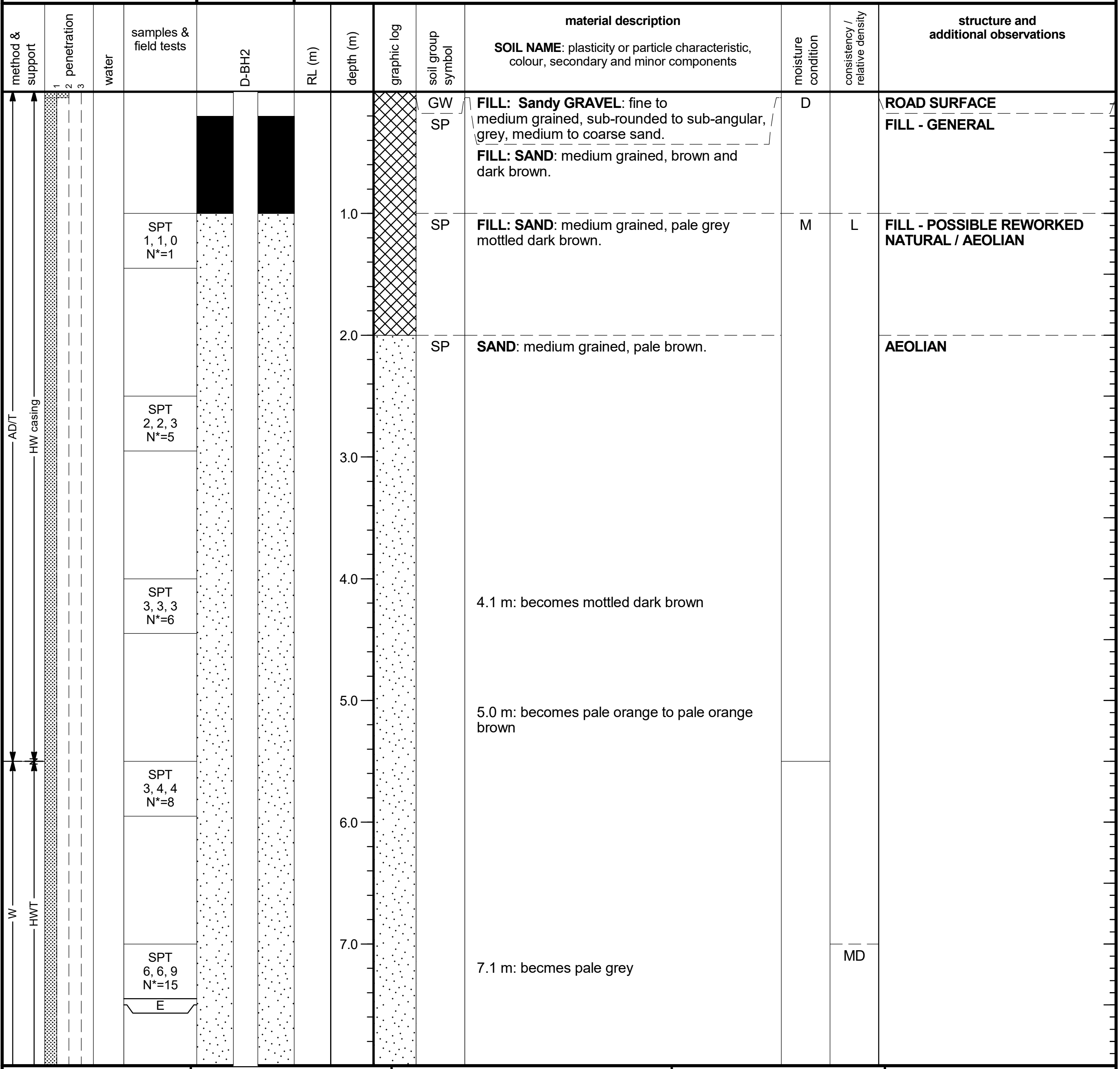
TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID.	***D-BH2***
sheet:	1 of 2
project no.	***754-NTLGE368007***
client:	***COHO Property Pty Ltd***
principal:	***COHO Property Pty Ltd***
project:	***Proposed Development Nelson Bay***
location:	***38 Stockton and 8A Tomaree Street, Nelson Bay***
date started:	***16 Sep 2024***
date completed:	***17 Sep 2024***
logged by:	***KF***
checked by:	***MJ***

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information | **well details** | **material substance**



depth (m)	method & support	samples & field tests	soil group symbol	material description	moisture condition	consistency / relative density	structure and additional observations
0.0	AD/T HW casing		GW	**FILL: Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, grey, medium to coarse sand.	D		**ROAD SURFACE**
			SP	**FILL: SAND**: medium grained, brown and dark brown.			**FILL - GENERAL**
1.0		SPT 1, 1, 0 N*=1	SP	**FILL: SAND**: medium grained, pale grey mottled dark brown.	M	L	**FILL - POSSIBLE REWORKED NATURAL / AEOLIAN**
2.0		SPT 2, 2, 3 N*=5	SP	**SAND**: medium grained, pale brown.			**AEOLIAN**
4.0		SPT 3, 3, 3 N*=6		4.1 m: becomes mottled dark brown			
5.0				5.0 m: becomes pale orange to pale orange brown			
5.5	W HWT	SPT 3, 4, 4 N*=8					
7.0		SPT 6, 6, 9 N*=15 E		7.1 m: becmes pale grey		MD	

method	
AD	auger drilling*
AS	auger screwing*
HA	hand auger
W	washbore
*	bit shown by suffix
e.g.	AD/T
B	blank bit
T	TC bit
V	V bit

support			
M	mud	N	nil
C	casing		

penetration

1 2 3 no resistance ranging to refusal

water

- 10-Oct-12 water level on date shown
- water inflow
- water outflow

samples & field tests	
B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition	
D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density	
VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

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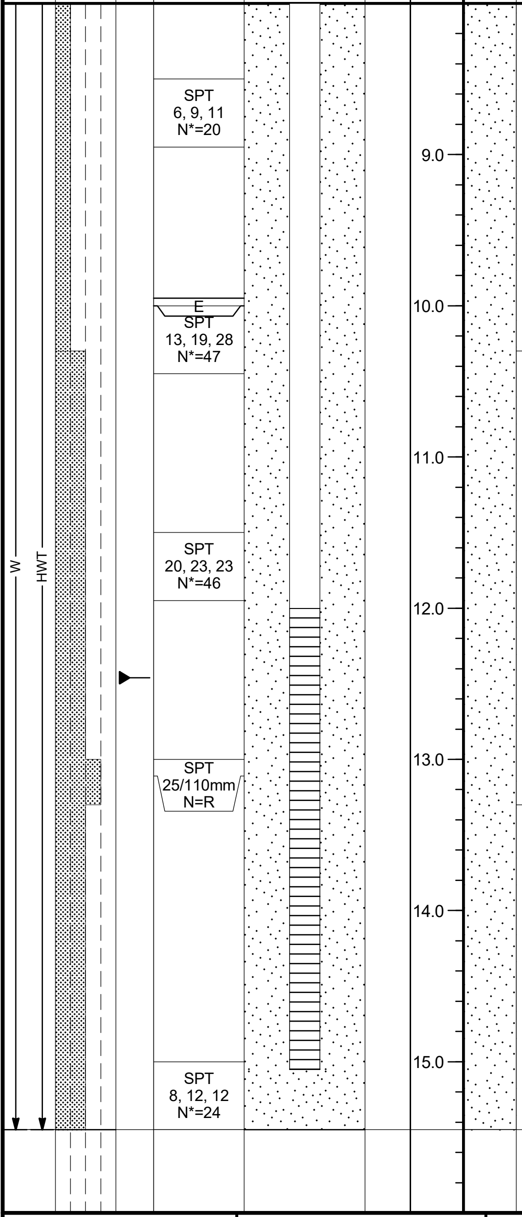
TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID.	**D-BH2**
sheet:	2 of 2
project no.	**754-NTLGE368007**
client:	**COHO Property Pty Ltd**
principal:	**COHO Property Pty Ltd**
project:	**Proposed Development Nelson Bay**
location:	**38 Stockton and 8A Tomaree Street, Nelson Bay**
date started:	**16 Sep 2024**
date completed:	**17 Sep 2024**
logged by:	**KF**
checked by:	**MJ**

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information | **well details** | **material substance**



samples & field tests
SPT 6, 9, 11 N*=20
E SPT 13, 19, 28 N*=47
SPT 20, 23, 23 N*=46
SPT 25/110mm N=R
SPT 8, 12, 12 N*=24

soil group symbol	material description	moisture condition	consistency / relative density	structure and additional observations
SP	**SAND**: medium grained, pale brown. *(continued)*		MD	**AEOLIAN**
SW	**SAND**: fine to medium grained, mottled pale brown and dark brown, with silt. 11.0 m: decreased silt, no mottle 11.7 m: becoming red-orange 13.11 to 13.26 m: hard bond, near refusal		VD	**AEOLIAN INDURATED**
SW	**SAND**: fine to medium grained, pale brown.		D - VD	**AEOLIAN**
	Borehole D-BH2 terminated at 15.45 m			**standpipe piezo. D-BH2 details:** stickup: -0.05m 12.0-15.05m: screen

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
N nil
C casing

penetration
1 2 3 no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B bulk disturbed sample
D disturbed sample
E environmental sample
SS split spoon sample
U## undisturbed sample ##mm diameter
HP hand penetrometer (kPa)
N standard penetration test (SPT)
N* SPT - sample recovered
Nc SPT with solid cone
VS vane shear; peak/remouded (kPa)
R refusal
HB hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition
D dry
M moist
W wet
Wp plastic limit
Wl liquid limit

consistency / relative density
VS very soft
S soft
F firm
St stiff
VSt very stiff
H hard
Fb friable
VL very loose
L loose
MD medium dense
D dense
VD very dense

TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID. **D-BH3**
sheet: 1 of 3
project no. **754-NTLGE368007**

client: **COHO Property Pty Ltd**
principal: **COHO Property Pty Ltd**
project: **Proposed Development Nelson Bay**
location: **38 Stockton and 8A Tomaree Street, Nelson Bay**

date started: **17 Sep 2024**
date completed: **17 Sep 2024**
logged by: **KF**
checked by: **MJ**

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information | **well details** | **material substance**

method & support	penetration 1 2 3	water	samples & field tests	well details	RL (m)	depth (m)	graphic log	soil group symbol	material description SOIL NAME: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	structure and additional observations
AD/T HW casing								GW	**FILL: Sandy GRAVEL**: fine to medium grained, sub-rounded to sub-angular, grey.	D M		**FILL - GENERAL**
								SP	**SAND**: medium grained, pale yellow-brown.			**POSSIBLE REWORKED NATURAL / AEOLIAN**
						1.0						
						2.0		SP	**SAND**: medium grained, pale yellow-brown.			**AEOLIAN**
W HWT						3.0						
						4.0						
						5.0						
						6.0						
						7.0						

method
AD	auger drilling*
AS	auger screwing*
HA	hand auger
W	washbore
*	bit shown by suffix
e.g.	AD/T
B	blank bit
T	TC bit
V	V bit

support
M mud
N nil
C casing

penetration
1 2 3 no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests
B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition
D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density
VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

Engineering Log - Borehole

Hole ID.	**D-BH3**
sheet:	2 of 3
project no.	**754-NTLGE368007**
date started:	**17 Sep 2024**
date completed:	**17 Sep 2024**
logged by:	**KF**
checked by:	**MJ**

client: ***COHO Property Pty Ltd***
principal: ***COHO Property Pty Ltd***
project: ***Proposed Development Nelson Bay***
location: ***38 Stockton and 8A Tomaree Street, Nelson Bay***

position: Not Specified | surface elevation: Not Specified | angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted | drilling fluid: Water | casing diameter : HW

drilling information | **well details** | **material substance**

method & support	penetration 1 2 3	water	samples & field tests	well details	RL (m)	depth (m)	graphic log	soil group symbol	material description **SOIL NAME**: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	structure and additional observations
W HWT								SP	**SAND**: medium grained, pale yellow-brown. *(continued)*			**AEOLIAN**
						9.0						
						10.0						
						11.0						
						12.0						
						13.0						
						14.0			14.0 m: becomes orange-red possible indurated sand			
						15.0			15.0 m: becomes off-white to pale grey			

method
AD auger drilling*
AS auger screwing*
HA hand auger
W washbore

* bit shown by suffix
e.g. AD/T
B blank bit
T TC bit
V V bit

support
M mud
N nil
C casing

penetration
1 2 3
no resistance ranging to refusal

water
10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests

B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

TETRA TECH
COFFEY

Engineering Log - Borehole

Hole ID. **D-BH3**
sheet: 3 of 3
project no. ***754-NTLGE368007***

client: ***COHO Property Pty Ltd***
principal: ***COHO Property Pty Ltd***
project: ***Proposed Development Nelson Bay***
location: ***38 Stockton and 8A Tomaree Street, Nelson Bay***

date started: ***17 Sep 2024***
date completed: ***17 Sep 2024***
logged by: ***KF***
checked by: ***MJ***

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid: Water
casing diameter : HW

drilling information				well details	material substance							
method & support	penetration 1 2 3	water	samples & field tests		RL (m)	depth (m)	graphic log	soil group symbol	material description **SOIL NAME**: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	structure and additional observations
W HWT								SP	**SAND**: medium grained, pale yellow-brown. *(continued)*			**AEOLIAN**
						17.0						
						18.0			18.0 m: becomes pale brown			
						19.0						
								SC	**CLAYEY SAND**: medium grained, pale red-brown.			**POSSIBLE RESIDUAL SOIL / INDURATED SAND**
						20.0						
						21.0			Borehole D-BH3 terminated at 20.85 m			**PROBABLE ROCK**
						22.0						
						23.0						

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method

AD	auger drilling*
AS	auger screwing*
HA	hand auger
W	washbore
*	bit shown by suffix
e.g.	AD/T
B	blank bit
T	TC bit
V	V bit

support

M	mud	N	nil
C	casing		

penetration

1 2 3 no resistance ranging to refusal

water

10-Oct-12 water level on date shown
◀ water inflow
▶ water outflow

samples & field tests

B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

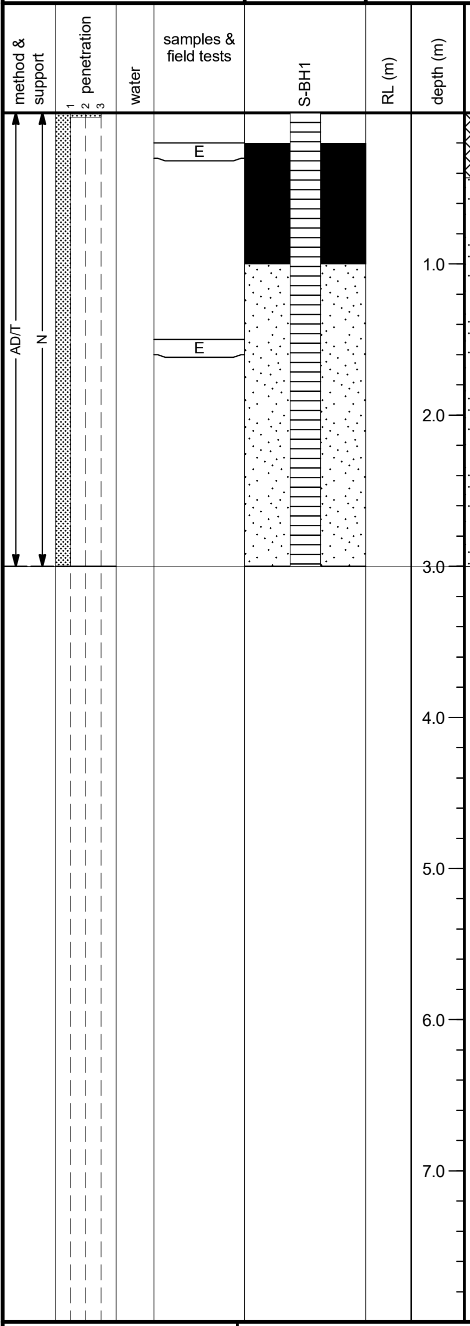
TETRA TECH COFFEY

Engineering Log - Borehole

Hole ID.	*S-BH1*
sheet:	1 of 1
project no.	*754-NTLGE368007*
client:	*COHO Property Pty Ltd*
principal:	*COHO Property Pty Ltd*
project:	*Proposed Development Nelson Bay*
location:	*38 Stockton and 8A Tomaree Street, Nelson Bay*
date started:	*16 Sep 2024*
date completed:	*16 Sep 2024*
logged by:	*KF*
checked by:	*MJ*

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid:
hole diameter : 125 mm

drilling information | **well details** | **material substance**



soil group symbol	material description (SOIL NAME: plasticity or particle characteristic, colour, secondary and minor components)	moisture condition	consistency / relative density	structure and additional observations
SW	**FILL: ASHPHALTIC CONCRETE**: black, 40% aggregate.	D		**ROAD SURFACE**
	FILL: Gravelly SAND: medium grained, dark grey, fine to medium sub-angular gravel.			**FILL - GENERAL**
SP	**SAND**: medium grained, yellow-brown to pale brown.			**FILL - POSSIBLE REWORKED NATURAL / AEOLIAN**
	Borehole S-BH1 terminated at 3.00 m			**standpipe piezo. S-BH1 details:** stickup: -0.05m 0.0-3.0m: screen

method

AD	auger drilling*
AS	auger screwing*
HA	hand auger
W	washbore
*	bit shown by suffix
e.g.	AD/T
B	blank bit
T	TC bit
V	V bit

support

M	mud	N	nil
C	casing		

penetration

1 2 3 no resistance ranging to refusal

water

10-Oct-12 water level on date shown
water inflow
water outflow

samples & field tests

B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density

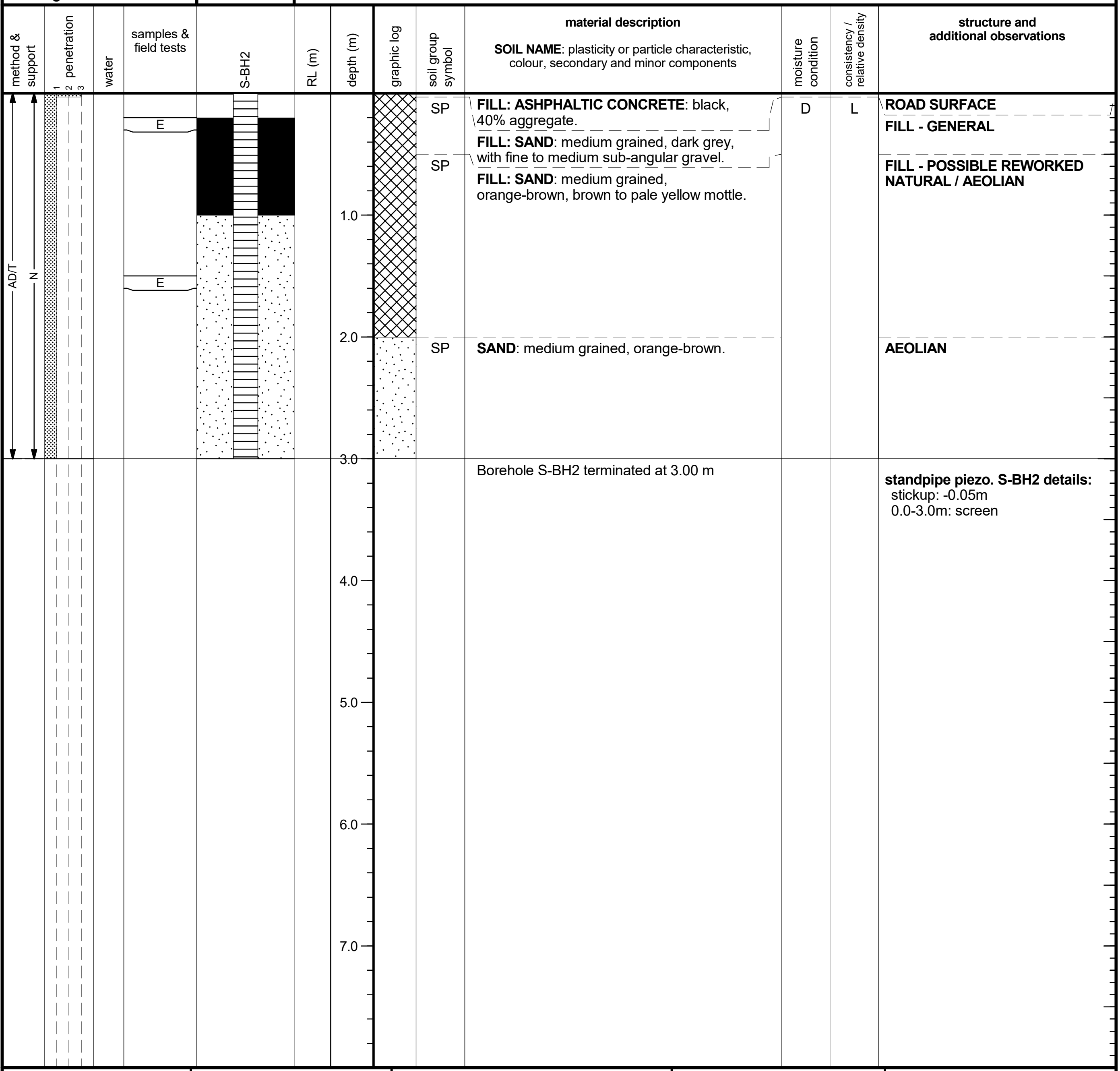
VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

Engineering Log - Borehole

Hole ID.	***S-BH2***
sheet:	1 of 1
project no.	***754-NTLGE368007***
client:	***COHO Property Pty Ltd***
principal:	***COHO Property Pty Ltd***
project:	***Proposed Development Nelson Bay***
location:	***38 Stockton and 8A Tomaree Street, Nelson Bay***
date started:	***16 Sep 2024***
date completed:	***16 Sep 2024***
logged by:	***KF***
checked by:	***MJ***

position: Not Specified
surface elevation: Not Specified
angle from horizontal: 90°
equipment type: Hanjin DB8, Track mounted
drilling fluid:
hole diameter : 125 mm

drilling information | **well details** | **material substance**



depth (m)	soil group symbol	material description **SOIL NAME**: plasticity or particle characteristic, colour, secondary and minor components	moisture condition	consistency / relative density	structure and additional observations
0.0	SP	**FILL: ASHPHALTIC CONCRETE**: black, 40% aggregate.	D	L	**ROAD SURFACE**
		FILL: SAND: medium grained, dark grey, with fine to medium sub-angular gravel.			**FILL - GENERAL**
	SP	**FILL: SAND**: medium grained, orange-brown, brown to pale yellow mottle.			**FILL - POSSIBLE REWORKED NATURAL / AEOLIAN**
2.0	SP	**SAND**: medium grained, orange-brown.			**AEOLIAN**
3.0		Borehole S-BH2 terminated at 3.00 m			**standpipe piezo. S-BH2 details:** stickup: -0.05m 0.0-3.0m: screen

method

AD	auger drilling*
AS	auger screwing*
HA	hand auger
W	washbore
*	bit shown by suffix
e.g.	AD/T
B	blank bit
T	TC bit
V	V bit

support

M	mud	N	nil
C	casing		

penetration

1 2 3 no resistance ranging to refusal

water

▼ 10-Oct-12 water level on date shown
▶ water inflow
◀ water outflow

samples & field tests

B	bulk disturbed sample
D	disturbed sample
E	environmental sample
SS	split spoon sample
U##	undisturbed sample ##mm diameter
HP	hand penetrometer (kPa)
N	standard penetration test (SPT)
N*	SPT - sample recovered
Nc	SPT with solid cone
VS	vane shear; peak/remouded (kPa)
R	refusal
HB	hammer bouncing

soil group symbol & soil description
based on AS 1726:2017

moisture condition

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

consistency / relative density

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable
VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

TETRA TECH COFFEY

Dynamic Cone Penetrometer Test Sheet

Client:	COHO Property	**Office:**	Newcastle	**Project Number:**	754-NTLGE368007
Principal:		**Date:**	16/09/2024		
Project Name:	36 Stockton and 8A Tomaree St, Nelsc	**Performed By:**	KF	**Sheet:**	1 of 2
Test Location:	36 Stockton and 8A Tomaree St, Nelsc	**Checked By:**	MJ		

Test Method AS 1289.6.3.2-1997(R2013) ☐ AS 1289.6.3.3-1997(R2013) ■ RTA Test Method T161 ☐ NZS 4402.6.5.2 (1988) ☐

DCP ID: GEOT02 Calibration due date: 16/9/2024

Test No:	**1**	**Test No:**	**2**	**Test No:**	**3**	**Test No:**	**4**
Test Location:	Refer Map	Test Location:	Refer Map	Test Location:	Refer Map	Test Location:	Refer Map
Starting Depth (m):	GL	Starting Depth (m):	0.4	Starting Depth (m):	0.3	Starting Depth (m):	0.3
Depth (m)	**Blows**	**Depth (m)**	**Blows**	**Depth (m)**	**Blows**	**Depth (m)**	**Blows**
0.10	41	0.10		0.10		0.10	
0.20	18	0.20		0.20		0.20	
0.30	13	0.30		0.30		0.30	5
0.40	15	0.40		0.40	9	0.40	10
0.50	13	0.50	7	0.50	14	0.50	11
0.60	11	0.60	10	0.60	12	0.60	12
0.70	12	0.70	12	0.70	12	0.70	9
0.80	10	0.80	11	0.80	12	0.80	8
0.90	9	0.90	9	0.90	10	0.90	8
1.00	8	1.00	8	1.00	11	1.00	6
1.10	8	1.10	8	1.10	15	1.10	7
1.20	10	1.20	8	1.20	9	1.20	6
1.30	9	1.30	5	1.30	9	1.30	8
1.40	9	1.40	5	1.40	9	1.40	8
1.50	9	1.50	5	1.50	7	1.50	6
1.60	7	1.60	4	1.60	7	1.60	6
1.70	7	1.70	4	1.70	5	1.70	6
1.80	7	1.80	6	1.80	4	1.80	4
1.90	7	1.90	7	1.90	4	1.90	4
2.00	6	2.00	5	2.00	5	2.00	3
2.10	7	2.10	5	2.10	3	2.10	3
2.20	7	2.20	4	2.20	5	2.20	4
2.30	8	2.30	5	2.30	4	2.30	4
2.40	8	2.40	5	2.40	4	2.40	5
2.50	7	2.50	5	2.50	5	2.50	6
2.60	6	2.60	4	2.60	5	2.60	5
2.70	6	2.70	5	2.70	4	2.70	5
2.80	5	2.80	5	2.80	4	2.80	4
2.90	4	2.90	4	2.90	4	2.90	4
3.00	5	3.00	4	3.00	5	3.00	5

Test Method		Drop Weight	Drop Height	Cone/Blunt tip	DCP Id
AS 1289.6.3.2-1997(R2013) - 9kg Dynamic Cone Penetrometer Test.		9 kg	510 mm	Cone	
AS 1289.6.3.3-1997 (R2013) - Perth penetrometer test		9 kg	600 mm	Blunt	
RTA Test Method T161 (October 2012)		9 kg	510 mm	Cone	
NZS 4402.6.5.2 (1998) Determinaton of the penetration resistance of a		9 kg	510 mm	Cone	

Notes: DCP testing is typically restricted to depths less than 3m. Testing should stop if the cone resistance exceeds 8 blows per 20mm to avoid tip damage. Perth penetrometer testing should stop if the blow count exceeds 30 blows per 300mm to avoid damage to equipment.

TETRA TECH COFFEY

Dynamic Cone Penetrometer Test Sheet

Client: COHO Property **Office:** Newcastle **Project Number:** 754-NTLGE368007

Principal: **Date:** 16/09/2024

Project Name: 36 Stockton and 8A Tomaree St, Nels(**Performed By:** KF **Sheet:** 1 of 2

Test Location: 36 Stockton and 8A Tomaree St, Nels(**Checked By:** MJ

Test Method AS 1289.6.3.2-1997(R2013) ☐ AS 1289.6.3.3-1997(R2013) ☒ RTA Test Method T161 ☐ NZS 4402.6.5.2 (1988) ☐

DCP ID: GEOT02 Calibration due date: 16/9/2024

Test No:	**5**	**Test No:**	**6**	**Test No:**	**7**	**Test No:**	**8**
Test Location:	Refer Map	Test Location:	Refer Map	Test Location:	Refer Map	Test Location:	Refer Map
Starting Depth (m):	0.2	Starting Depth (m):	GL	Starting Depth (m):	GL	Starting Depth (m):	0.2
Depth (m)	**Blows**	**Depth (m)**	**Blows**	**Depth (m)**	**Blows**	**Depth (m)**	**Blows**
0.10		0.10	12	0.10	23	0.10	
0.20		0.20	13	0.20	19	0.20	
0.30	5	0.30	18	0.30	18	0.30	15
0.40	7	0.40	19	0.40	20	0.40	16
0.50	7	0.50	19	0.50	20	0.50	12
0.60	8	0.60	24	0.60	19	0.60	11
0.70	8	0.70	30	0.70	16	0.70	11
0.80	9	0.80	28	0.80	15	0.80	10
0.90	6	0.90	21	0.90	12	0.90	9
1.00	3	1.00	17	1.00	11	1.00	8
1.10	3	1.10	16	1.10	11	1.10	6
1.20	4	1.20	12	1.20	8	1.20	8
1.30	3	1.30	13	1.30	10	1.30	8
1.40	4	1.40	13	1.40	10	1.40	7
1.50	4	1.50	15	1.50	12	1.50	7
1.60	3	1.60	15	1.60	11	1.60	7
1.70	3	1.70	16	1.70	10	1.70	6
1.80	3	1.80	15	1.80	10	1.80	6
1.90	4	1.90	15	1.90	9	1.90	5
2.00	4	2.00	15	2.00	10	2.00	6
2.10	3	2.10	15	2.10	9	2.10	5
2.20	4	2.20	14	2.20	11	2.20	7
2.30	4	2.30	15	2.30	11	2.30	6
2.40	4	2.40	14	2.40	12	2.40	4
2.50	5	2.50	15	2.50	13	2.50	5
2.60	5	2.60	20	2.60	12	2.60	5
2.70	5	2.70		2.70	11	2.70	4
2.80	6	2.80		2.80	11	2.80	3
2.90	5	2.90		2.90	10	2.90	4
3.00	6	3.00		3.00	9	3.00	3

Test Method		Drop Weight	Drop Height	Cone/Blunt tip	DCP Id
AS 1289.6.3.2-1997(R2013) - 9kg Dynamic Cone Penetrometer Test.		9 kg	510 mm	Cone	
AS 1289.6.3.3-1997 (R2013) - Perth penetrometer test		9 kg	600 mm	Blunt	
RTA Test Method T161 (October 2012)		9 kg	510 mm	Cone	
NZS 4402.6.5.2 (1998) Determinaton of the penetration resistance of a		9 kg	510 mm	Cone	

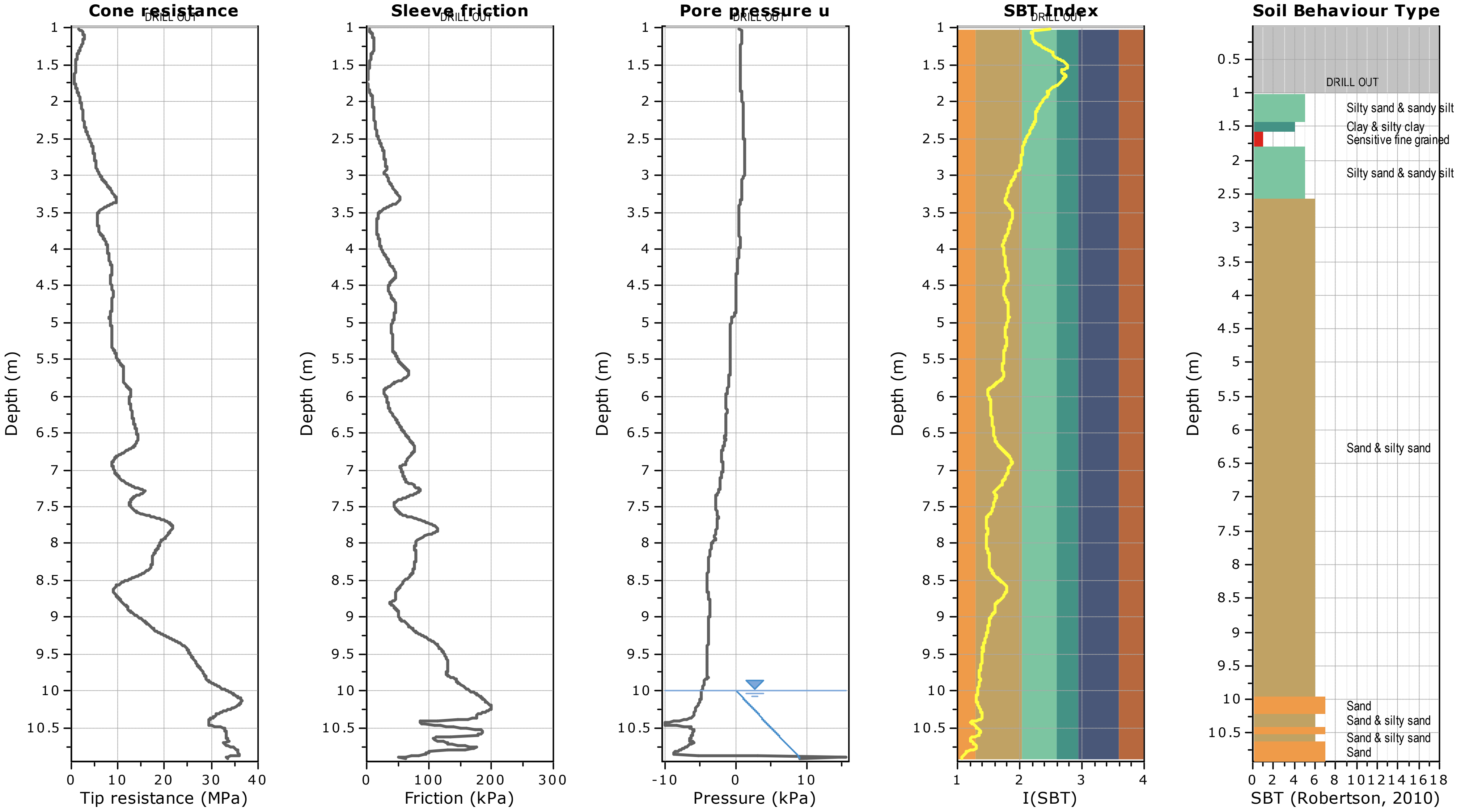
Notes: DCP testing is typically restricted to depths less than 3m. Testing should stop if the cone resistance exceeds 8 blows per 20mm to avoid tip damage. Perth penetrometer testing should stop if the blow count exceeds 30 blows per 300mm to avoid damage to equipment.

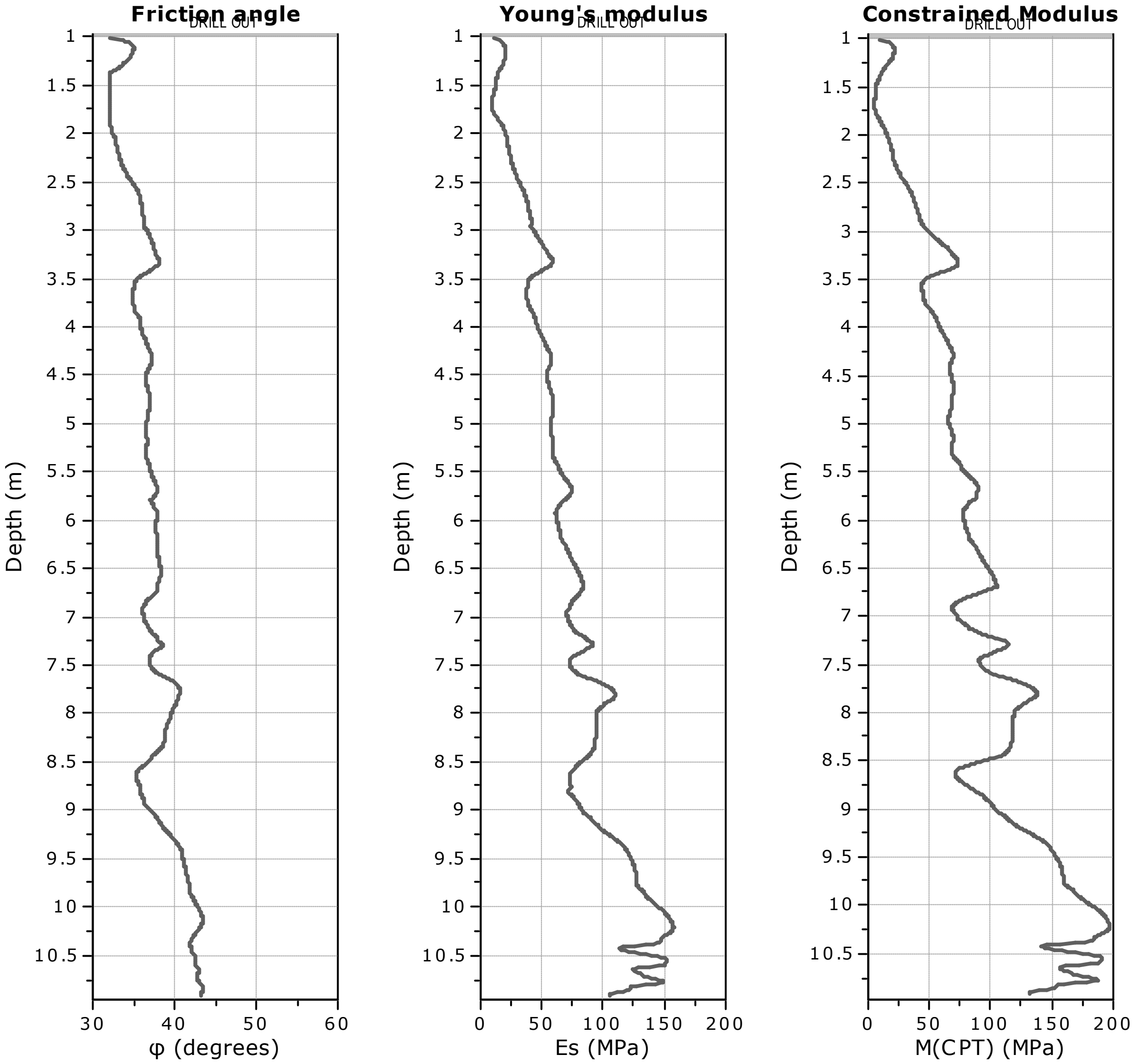
APPENDIX B: CPT LOGS

TETRA TECH
COFFEY

CPT: CPT-01
Total depth: 10.92 m, Date: 17/09/2024
Surface Elevation: 0.00 m, Est. GWL: 10.00 m
Coords: X:419662.20, Y:6379061.79
Cone Type: C10CFIIP.C22246
Cone Operator: DL

Project:
Location:

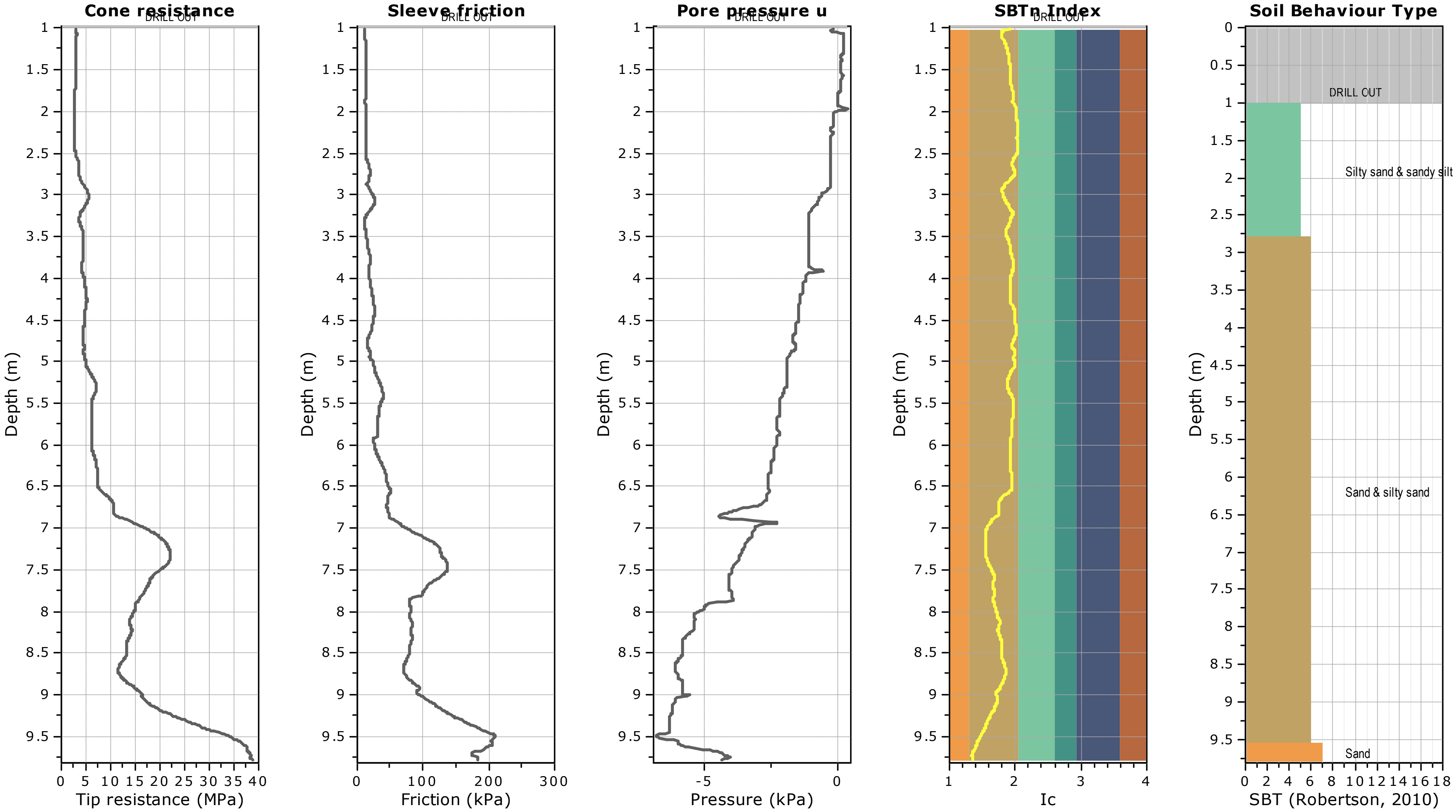



Friction angle
DRILL OUT
Depth (m)
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
9
9.5
10
10.5
30
40
50
60
φ (degrees)
Young's modulus
DRILL OUT
Depth (m)
0
50
100
150
200
Es (MPa)
Constrained Modulus
DRILL OUT
Depth (m)
0
50
100
150
200
M(CPT) (MPa)

CPT: CPT-02

Total depth: 9.78 m, Date: 17/09/2024
Surface Elevation: 0.00 m, Est. GWL: 10.00 m
Coords: X:419683.63, Y:6379057.31
Cone Type: C10CFIIP.C22246
Cone Operator: DL

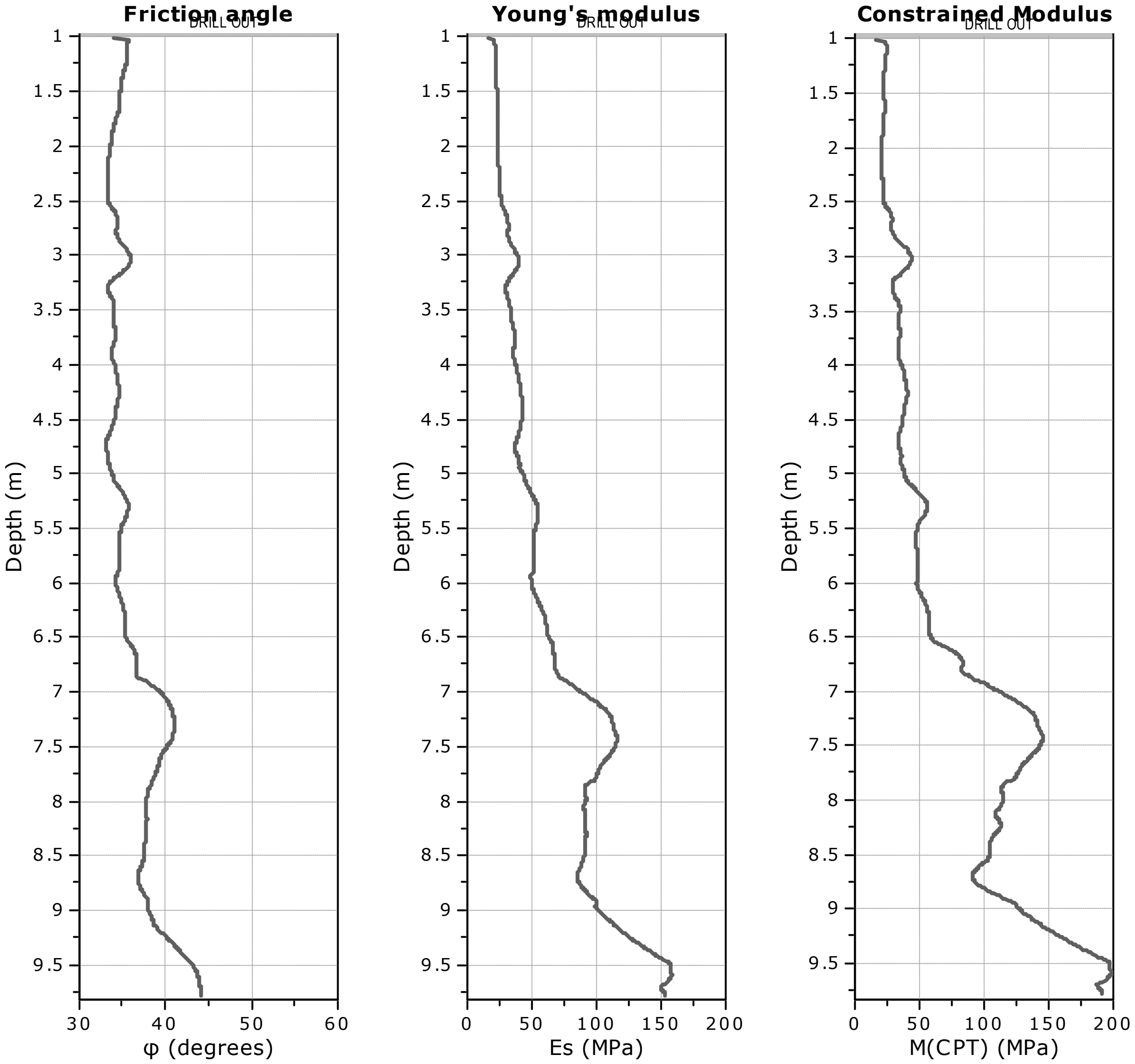
Project:
Location:



CPT: CPT-02

Total depth: 9.78 m, Date: 17/09/2024
Surface Elevation: 0.00 m, Est. GWL: 10.00 m
Coords: X:419683.63, Y:6379057.31
Cone Type: C10CFIIP.C22246
Cone Operator: DL

Project:
Location:



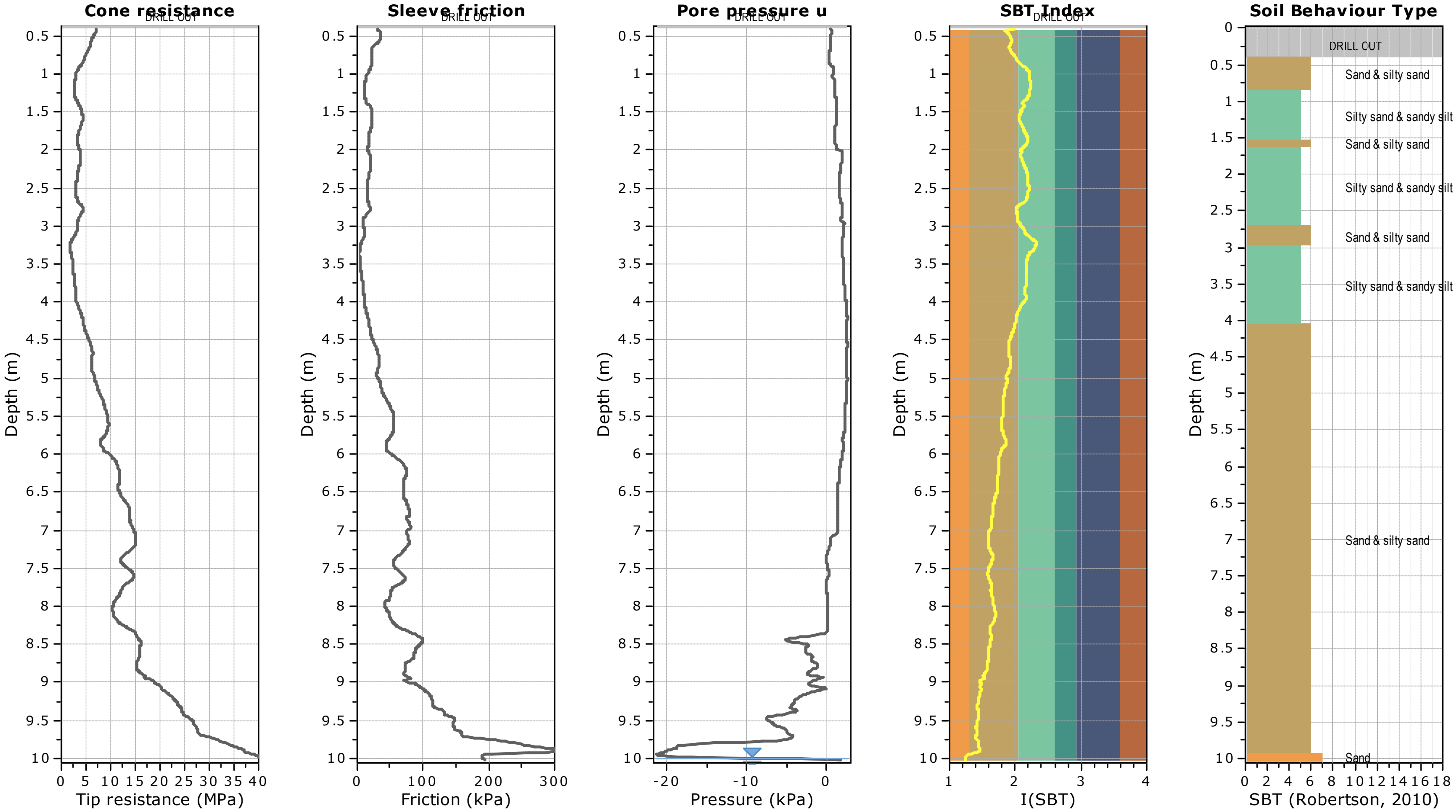
TETRA TECH
COFFEY

CPT: CPT-03

Total depth: 10.02 m, Date: 17/09/2024
Surface Elevation: 0.00 m, Est. GWL: 10.00 m
Coords: X:419672.25, Y:6379051.53
Cone Type: C10CFIIP.C22246
Cone Operator: DL

Project:

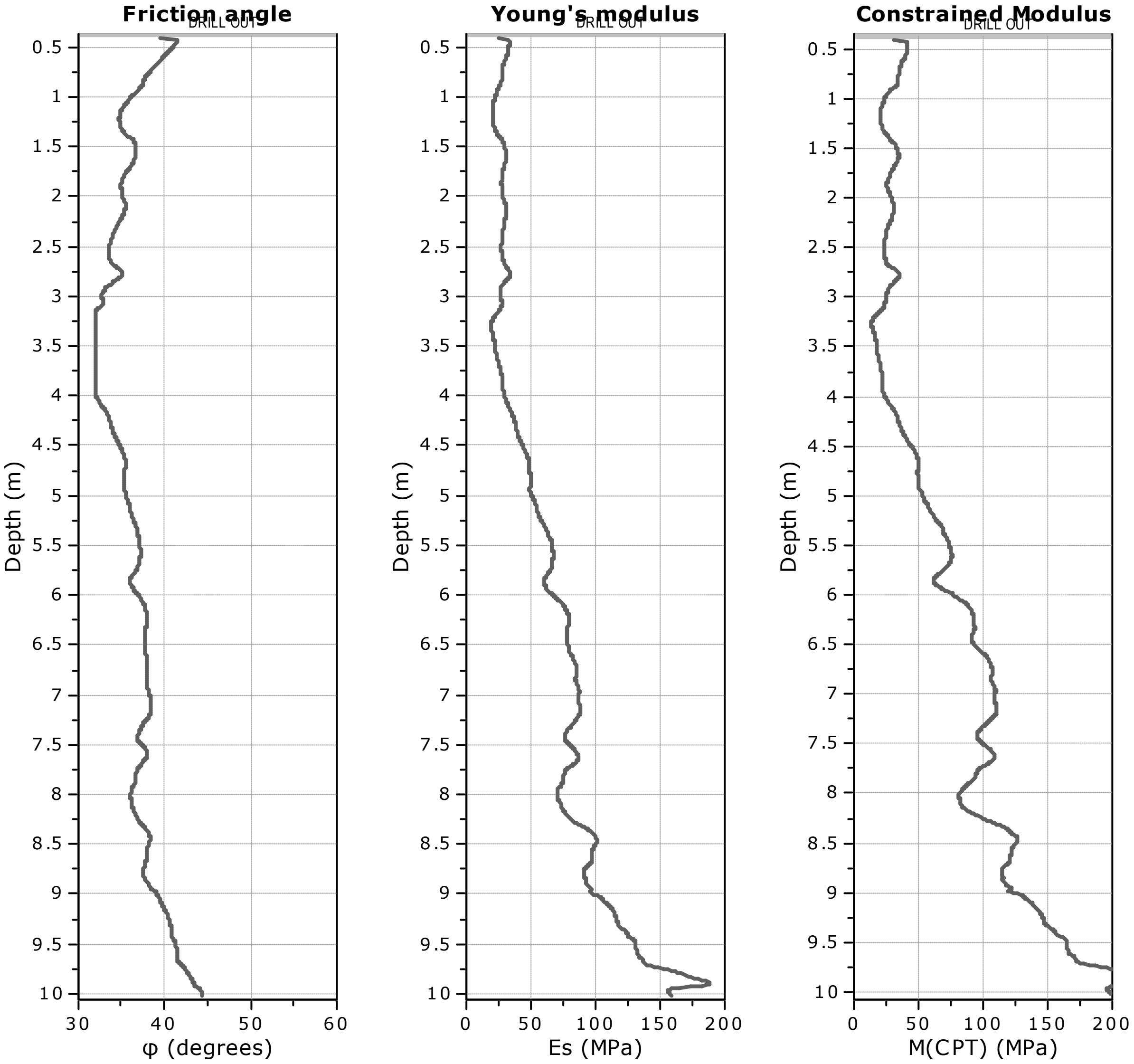
Location:

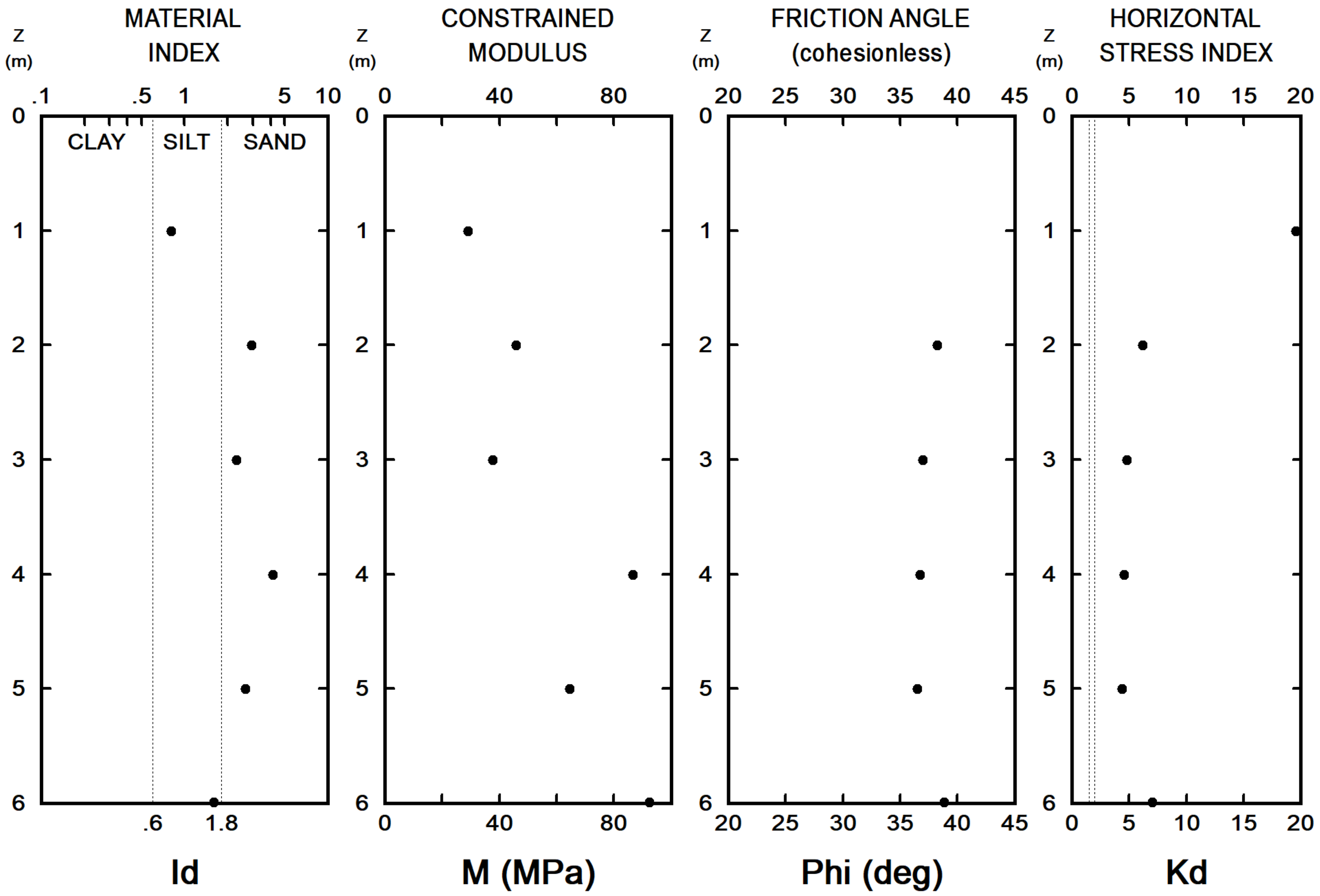


CPT: CPT-03

Total depth: 10.02 m, Date: 17/09/2024
Surface Elevation: 0.00 m, Est. GWL: 10.00 m
Coords: X:419672.25, Y:6379051.53
Cone Type: C10CFIIP.C22246
Cone Operator: DL

Project:
Location:





TETRA TECH

KIEREN F

38 STOCKTON ST, NELSON BAY

INTERPRETED GEOTECHNICAL PARAMETERS

TEST

D-CPT-01

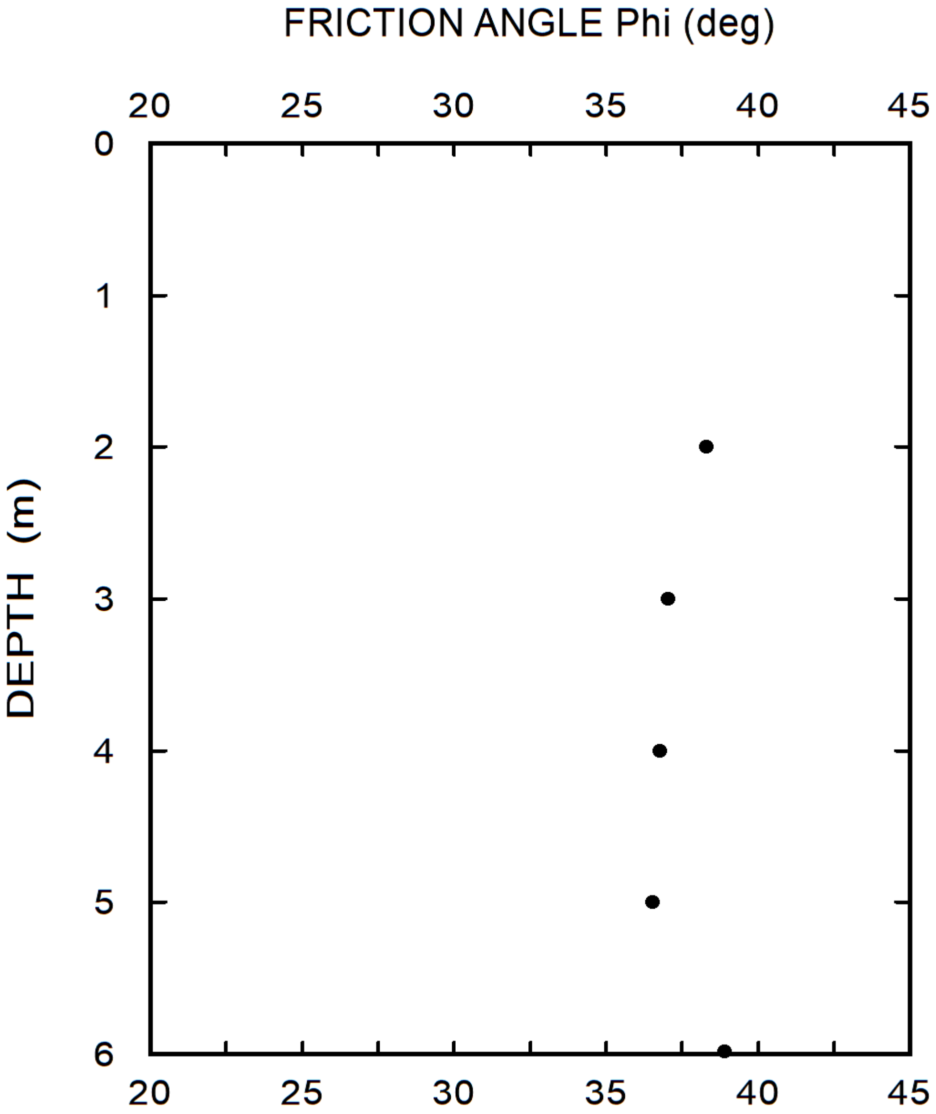
17-09-24

TETRA TECH

KIEREN F
38 STOCKTON ST, NELSON BAY

TEST
D-CPT-01
17-09-24

INTERPRETED GEOTECHNICAL PARAMETERS



D-CPT-01

17-09-24 D-CPT-01.dat

TETRA TECH

KIEREN F

38 STOCKTON ST, NELSON BAY

LEGEND

Z = Depth Below Ground Level

Po,P1,P2 = Corrected A,B,C readings

Id = Material Index

Ed = Dilatometer modulus

Ud = Pore Press. Index = (P2-Uo)/(Po-Uo)

Gamma = Bulk unit weight

Sigma' = Effective overb. stress

Uo = Pore pressure

INTERPRETED PARAMETERS

Phi = Safe floor value of Friction Angle

Ko = In situ earth press. coeff.

M = Constrained modulus (at Sigma')

Cu = Undrained shear strength

Ocr = Overconsolidation ratio

(OCR = "relative OCR"- generally

realistic. If accurate independent OCR

available, apply suitable OCR Factor)

SOUNDING PARAMETERS

DeltaA = 25 kPa

DeltaB = 100 kPa

GammaTop = 17.0 kN/m^3

FactorEd = 34.7

Zm = 0.0 kPa

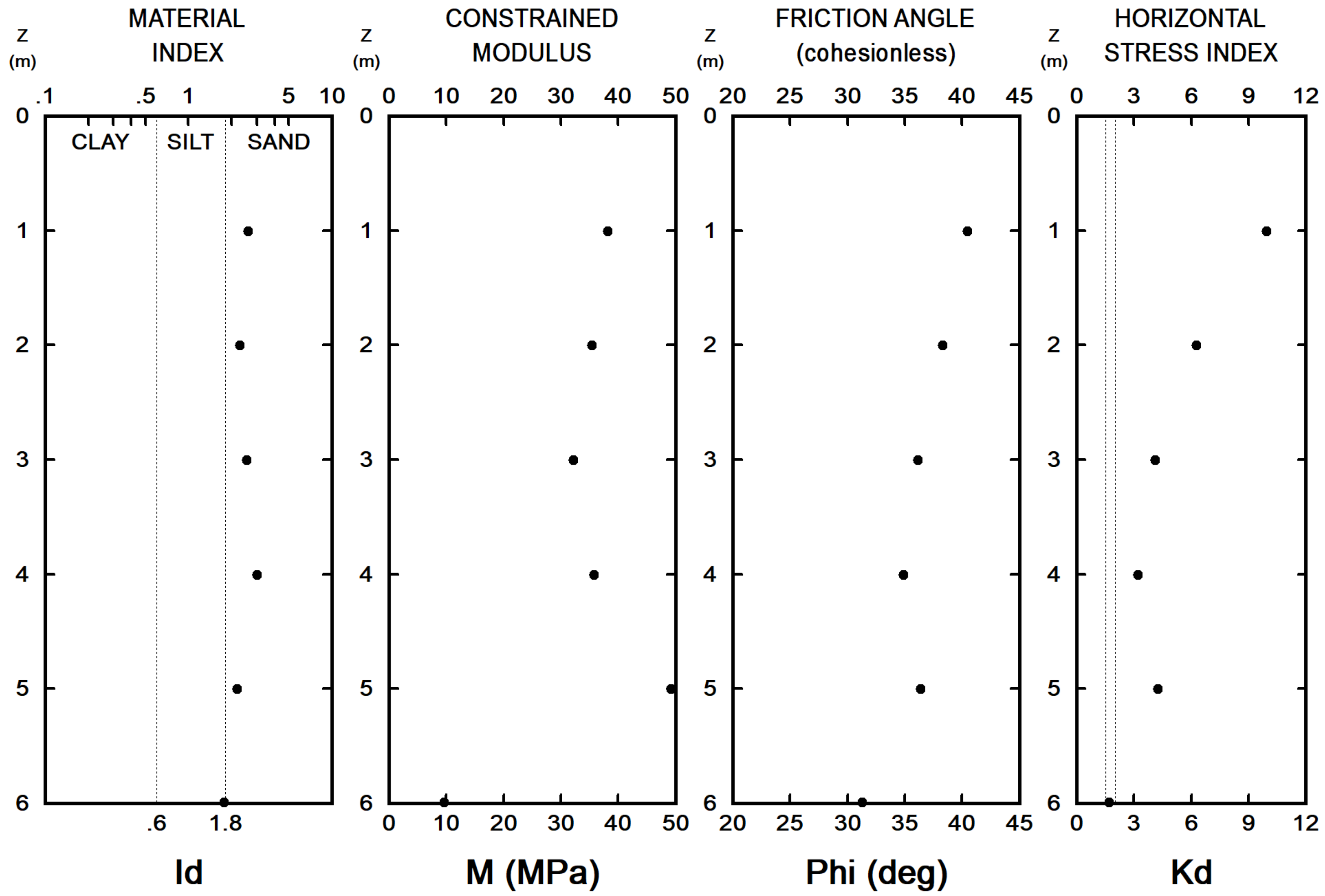
Zabs = 0.0 m

Zw > Zfinal

Water Level below end of sounding

Reduction formulae according to Marchetti, ASCE Geot.Jnl.Mar. 1980, Vol.109, 299-321; Phi according to TC16 ISSMGE, 2001

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	D-CPT-01 DESCRIPTION
1.0	320	700		332	600		17.7	17	0	0.81	19.5	9.3		2.7	35.1		29.1	65	SILT
2.0	220	950		215	850		18.6	35	0	2.96	6.2	22.0				38	45.9		SILTY SAND
3.0	260	950		257	850		18.6	53	0	2.31	4.8	20.6				37	37.7		SILTY SAND
4.0	370	1800		330	1700		18.6	72	0	4.16	4.6	47.5				37	86.7		SAND
5.0	420	1550		395	1450		18.6	91	0	2.67	4.4	36.6				36	64.5		SILTY SAND
6.0	800	2100		766	2000		19.1	109	0	1.61	7.0	42.8				39	92.5		SANDY SILT



TETRA TECH

KIEREN F

38 STOCKTON ST, NELSON BAY

INTERPRETED GEOTECHNICAL PARAMETERS

TEST

D-CPT-02

17-09-24

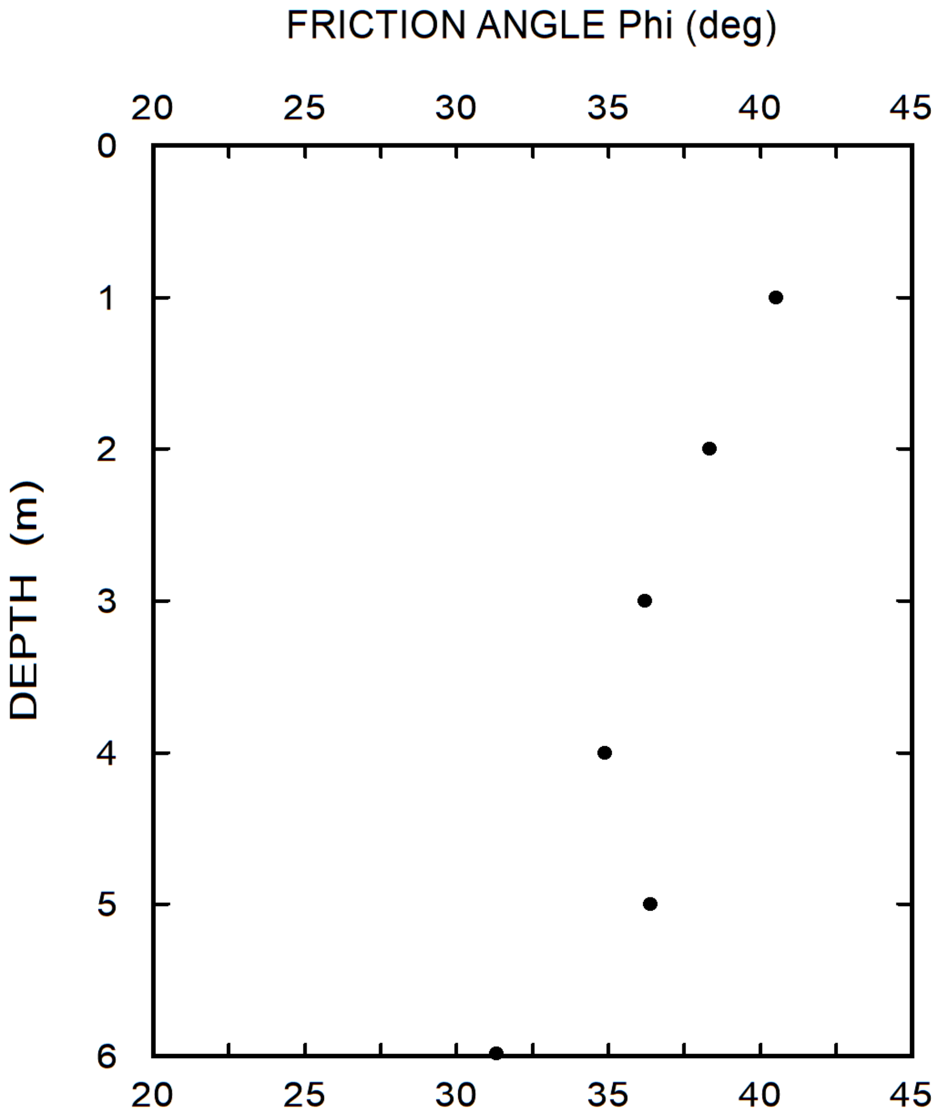
DILATOMETER TEST (D M T)

TETRA TECH

KIEREN F
38 STOCKTON ST, NELSON BAY

TEST
D-CPT-02
17-09-24

INTERPRETED GEOTECHNICAL PARAMETERS



D-CPT-02.dat

D-CPT-02

17-09-24 D-CPT-02.dat

TETRA TECH

KIEREN F

38 STOCKTON ST, NELSON BAY

LEGEND

Z = Depth Below Ground Level

Po,P1,P2 = Corrected A,B,C readings

Id = Material Index

Ed = Dilatometer modulus

Ud = Pore Press. Index = (P2-Uo)/(Po-Uo)

Gamma = Bulk unit weight

Sigma' = Effective overb. stress

Uo = Pore pressure

INTERPRETED PARAMETERS

Phi = Safe floor value of Friction Angle

Ko = In situ earth press. coeff.

M = Constrained modulus (at Sigma')

Cu = Undrained shear strength

Ocr = Overconsolidation ratio

(OCR = "relative OCR"- generally

realistic. If accurate independent OCR

available, apply suitable OCR Factor)

SOUNDING PARAMETERS

DeltaA = 20 kPa

DeltaB = 90 kPa

GammaTop = 17.0 kN/m^3

FactorEd = 34.7

Zm = 0.0 kPa

Zabs = 0.0 m

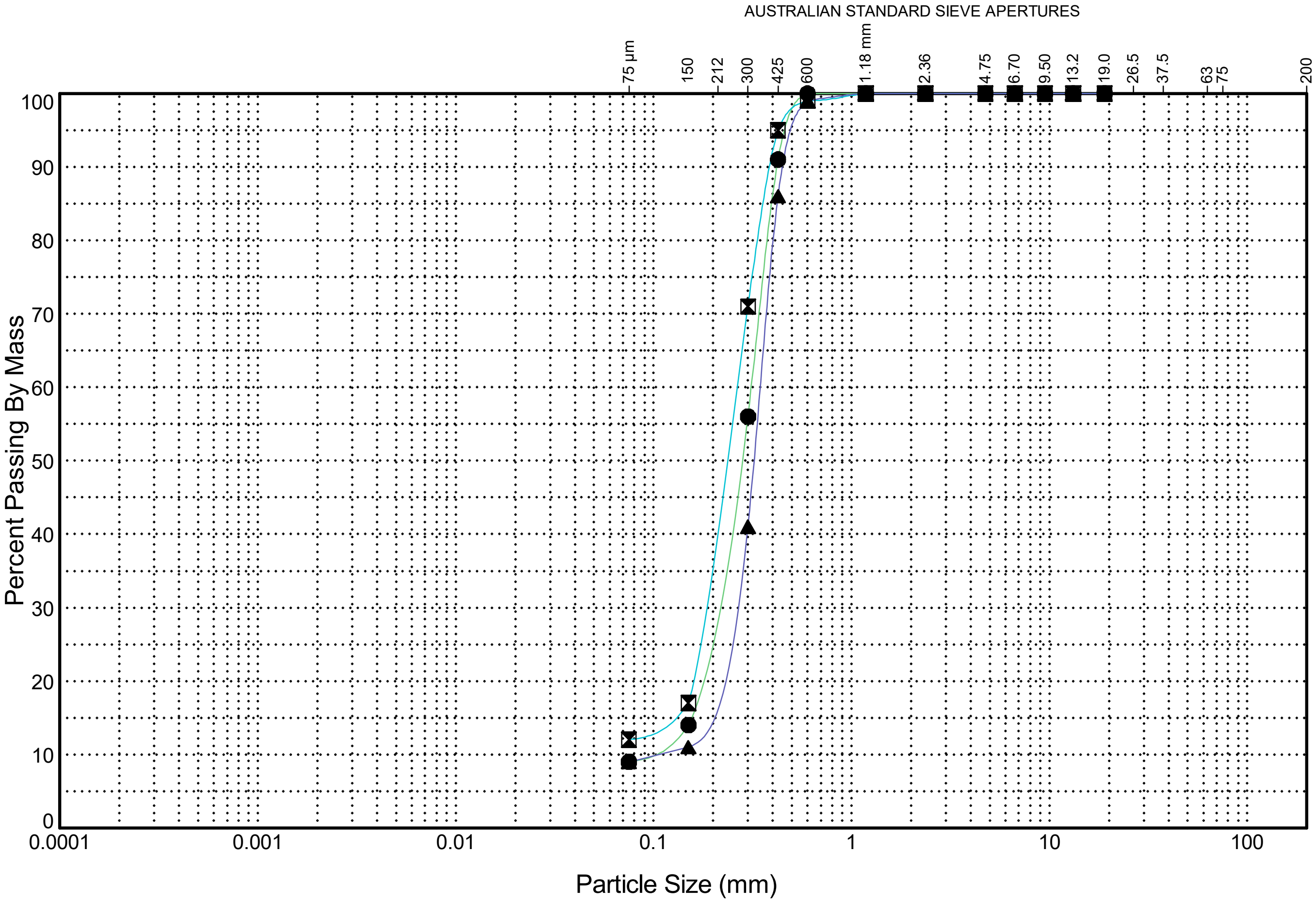
Zw > Zfinal

Water Level below end of sounding

Reduction formulae according to Marchetti, ASCE Geot.Jnl.Mar. 1980, Vol.109, 299-321; Phi according to TC16 ISSMGE, 2001

Z (m)	A (kPa)	B (kPa)	C (kPa)	Po (kPa)	P1 (kPa)	P2 (kPa)	Gamma (kN/m^3)	Sigma' (kPa)	Uo (kPa)	Id	Kd	Ed (MPa)	Ud	Ko	Ocr	Phi (Deg)	M (MPa)	Cu (kPa)	D-CPT-02 DESCRIPTION
1.0	170	700		169	610		17.7	17	0	2.61	9.9	15.3				40	38.2		SILTY SAND
2.0	220	800		217	710		17.7	35	0	2.28	6.2	17.1				38	35.4		SILTY SAND
3.0	220	850		214	760		18.6	52	0	2.55	4.1	18.9				36	32.2		SILTY SAND
4.0	240	1000		228	910		18.6	71	0	3.00	3.2	23.7				35	35.8		SILTY SAND
5.0	400	1300		381	1210		18.6	90	0	2.18	4.2	28.8				36	49.2		SILTY SAND
6.0	180	600		185	510		16.7	108	0	1.76	1.7	11.3				31	9.6		SANDY SILT

APPENDIX C: LABORATORY TEST RESULTS



CLAY	SILT			SAND			GRAVEL			COBBLES
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	

	Hole ID	Depth	Description from Log	LL	PL	PI
●	D-BH1	2.60	(SP) sand			
☒	D-BH1	4.00	(SW) sand			
▲	D-BH1	13.00	(SP) sand			

	Hole ID	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	D-BH1	2.60	19	0.312	0.195	0.086	0.0	91.0	9.0	
☒	D-BH1	4.00	19	0.26	0.177		0.0	88.0	12.0	
▲	D-BH1	13.00	19	0.348	0.233	0.106	0.0	91.0	9.0	

drawn	**M.J**
approved	
date	**02/10/2024**
scale	**N.T.S.**
original size	**A4**

TETRA TECH
COFFEY

client: COHO Property Pty Ltd

project: Proposed Development Nelson Bay
38 Stockton and 8A Tomaree Street, Nelson Bay

title: Particle Size Distribution Summary

project no: 754-NTLGE368007
fig no: 1
rev:

CDF_0_10_00.3_LIBRARY.GLB Graph COF GRAIN SIZE DETAILED-7 PER PAGE 754-NTLGE368007.KF.GPJ <<DrawingFile>> 01/10/2024 18:55

Material Test Report

Report Number: **NEWC24169-1**
Issue Number: 1
Date Issued: 01/10/2024
Client: Tetra Tech Coffey Pty Ltd (Newcastle)
Unit 4, 60 Griffiths Road, Lambton NSW 2299
Contact: Simon Baker
Project Number: NEWC24169
Project Name: Stockton Street Assessment
Client Reference: 754-NTLGE368007
Work Request: 2518
Sample Number: NEWC2518A
Date Sampled: 16/09/2024
Dates Tested: 23/09/2024 - 01/10/2024
Sampling Method: Sampled by Client
The results apply to the sample as received
Preparation Method: In accordance with the test method
Sample Location: **D - BH1 (2.6 - 2.8m)**

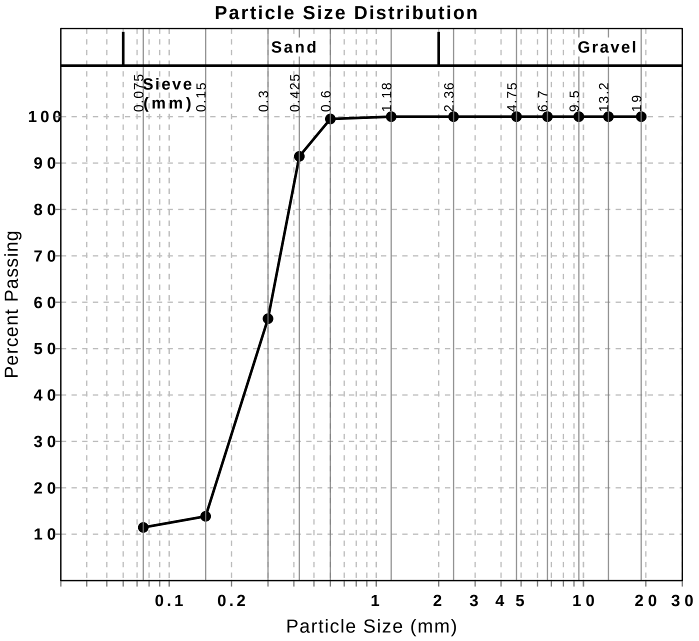
coffey TESTING
Newcastle Laboratory
16 Callistemon Close Warabrook NSW 2304
Phone: 0424 521 225
Email: Kerrina.Christiansen@coffeytesting.com

Accredited for compliance with ISO/IEC 17025 - Testing

R. Kirby-Faust

Approved Signatory: Raphael Kirby-Faust
Geotechnician
Laboratory Accreditation Number: 431

Particle Size Distribution (AS1289 3.6.1)						
Sieve	Passed %	Passing Limits		Retained %	Retained Limits	
19 mm	**100**			0		
13.2 mm	**100**			0		
9.5 mm	**100**			0		
6.7 mm	**100**			0		
4.75 mm	**100**			0		
2.36 mm	**100**			0		
1.18 mm	**100**			0		
0.6 mm	**100**			0		
0.425 mm	**91**			8		
0.3 mm	**56**			35		
0.15 mm	**14**			43		
0.075 mm	**11**			2		



Material Test Report

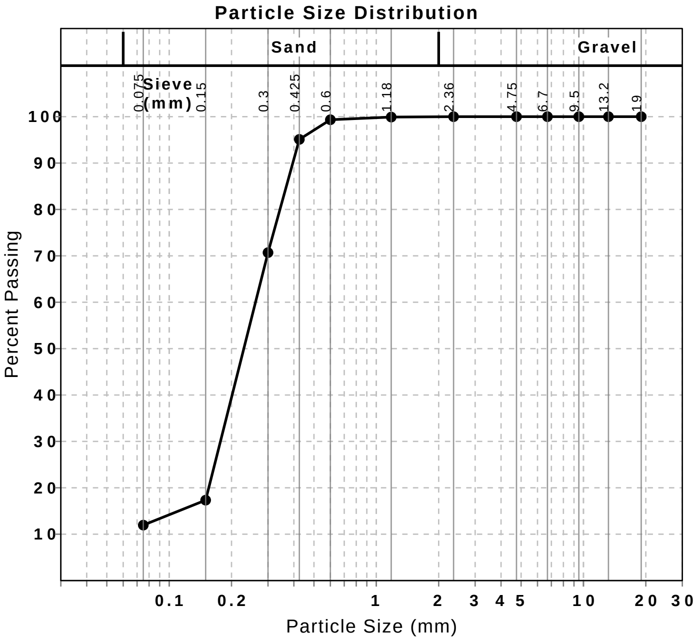
Report Number: **NEWC24169-1**
Issue Number: 1
Date Issued: 01/10/2024
Client: Tetra Tech Coffey Pty Ltd (Newcastle)
Unit 4, 60 Griffiths Road, Lambton NSW 2299
Contact: Simon Baker
Project Number: NEWC24169
Project Name: Stockton Street Assessment
Client Reference: 754-NTLGE368007
Work Request: 2518
Sample Number: NEWC2518B
Date Sampled: 16/09/2024
Dates Tested: 23/09/2024 - 01/10/2024
Sampling Method: Sampled by Client
The results apply to the sample as received
Preparation Method: In accordance with the test method
Sample Location: **D - BH1 (4.0 - 4.45m)**

Newcastle Laboratory
16 Callistemon Close Warabrook NSW 2304
Phone: 0424 521 225
Email: Kerrina.Christiansen@coffeytesting.com

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Raphael Kirby-Faust
Geotechnician
Laboratory Accreditation Number: 431

Particle Size Distribution (AS1289 3.6.1)						
Sieve	Passed %	Passing Limits		Retained %	Retained Limits	
19 mm	**100**			0		
13.2 mm	**100**			0		
9.5 mm	**100**			0		
6.7 mm	**100**			0		
4.75 mm	**100**			0		
2.36 mm	**100**			0		
1.18 mm	**100**			0		
0.6 mm	**99**			1		
0.425 mm	**95**			4		
0.3 mm	**71**			24		
0.15 mm	**17**			53		
0.075 mm	**12**			5		



Material Test Report

Newcastle Laboratory
16 Callistemon Close Warabrook NSW 2304
Phone: 0424 521 225
Email: Kerrina.Christiansen@coffeytesting.com

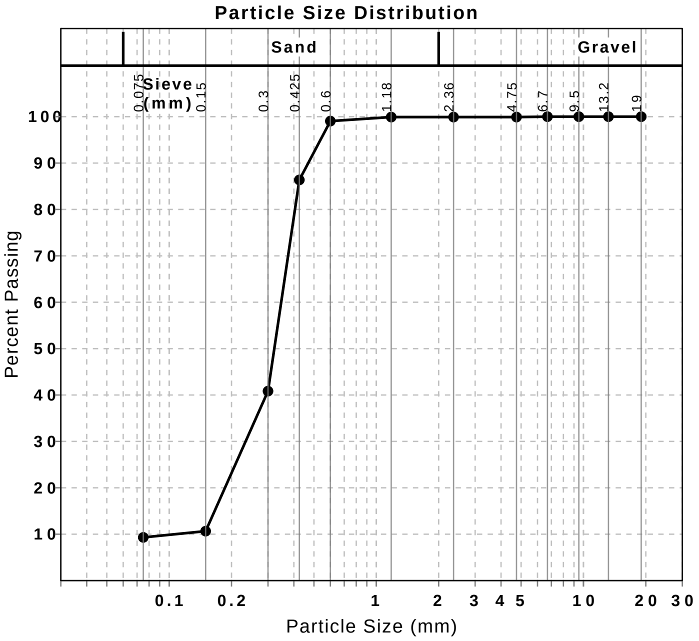
Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Raphael Kirby-Faust
Geotechnician
Laboratory Accreditation Number: 431

Report Number:	**NEWC24169-1**
Issue Number:	1
Date Issued:	01/10/2024
Client:	Tetra Tech Coffey Pty Ltd (Newcastle) Unit 4, 60 Griffiths Road, Lambton NSW 2299
Contact:	Simon Baker
Project Number:	NEWC24169
Project Name:	Stockton Street Assessment
Client Reference:	754-NTLGE368007
Work Request:	2518
Sample Number:	NEWC2518C
Date Sampled:	16/09/2024
Dates Tested:	23/09/2024 - 01/10/2024
Sampling Method:	Sampled by Client *The results apply to the sample as received*
Preparation Method:	In accordance with the test method
Sample Location:	**D - BH1 (13.0 - 13.45m)**

Particle Size Distribution (AS1289 3.6.1)

Sieve	Passed %	Passing Limits		Retained %	Retained Limits	
19 mm	**100**			0		
13.2 mm	**100**			0		
9.5 mm	**100**			0		
6.7 mm	**100**			0		
4.75 mm	**100**			0		
2.36 mm	**100**			0		
1.18 mm	**100**			0		
0.6 mm	**99**			1		
0.425 mm	**86**			13		
0.3 mm	**41**			46		
0.15 mm	**11**			30		
0.075 mm	**9**			1		



APPENDIX D: LIQUIFACTION RESULTS

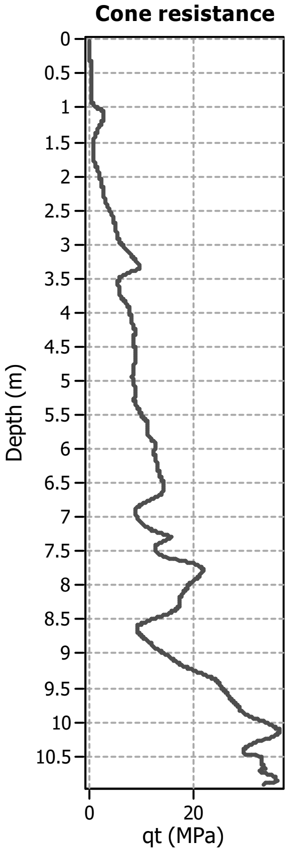
LIQUEFACTION ANALYSIS REPORT

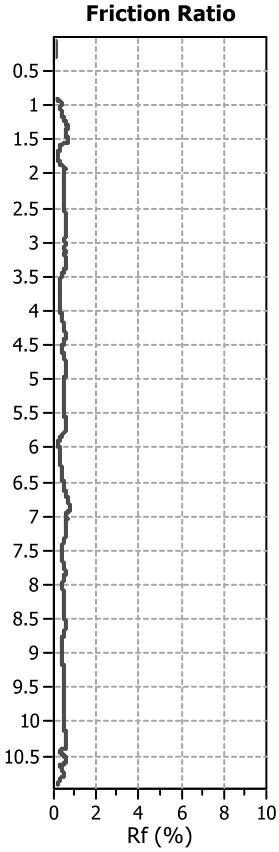
Project title : 38 Stockton & 8A Tomaree Street **Location : Nelson Bay**

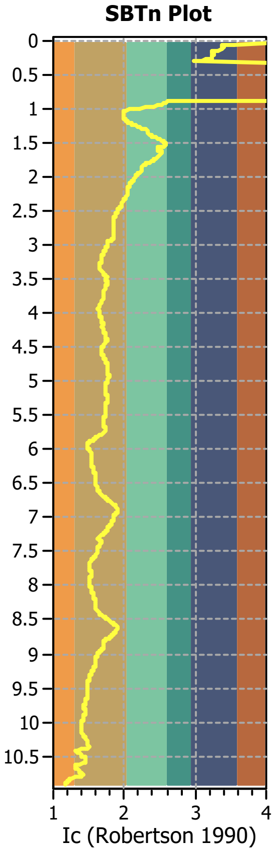
CPT file : CPT1

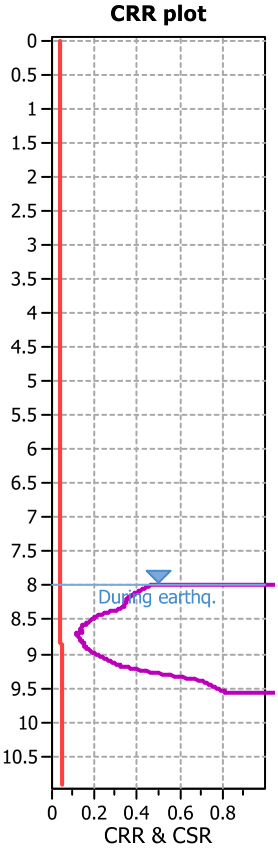
Input parameters and analysis data

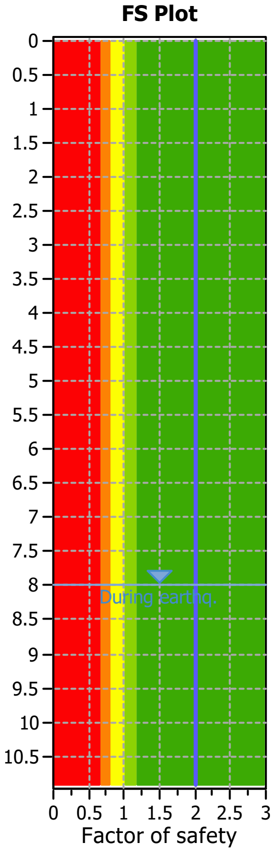
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	12.00 m	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.11	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

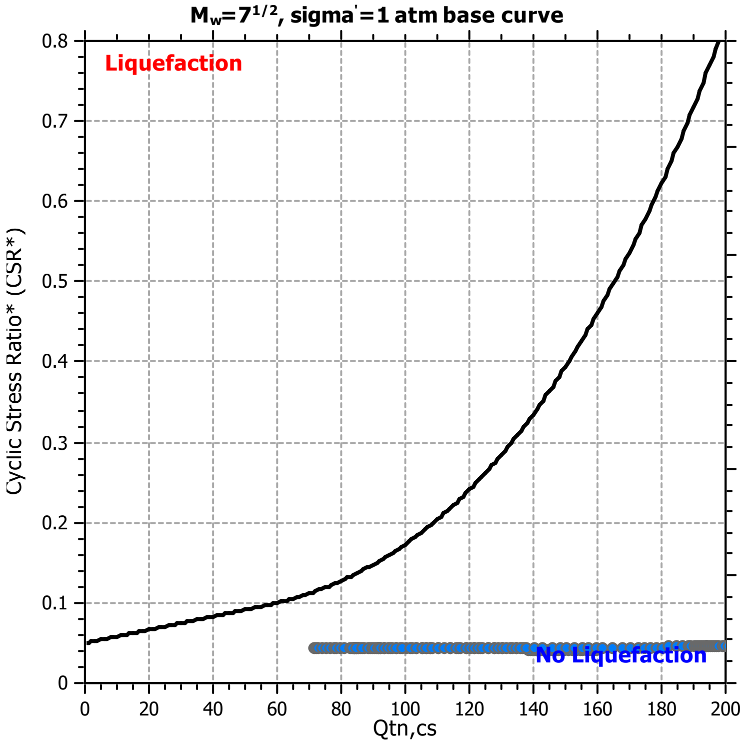

Cone resistance

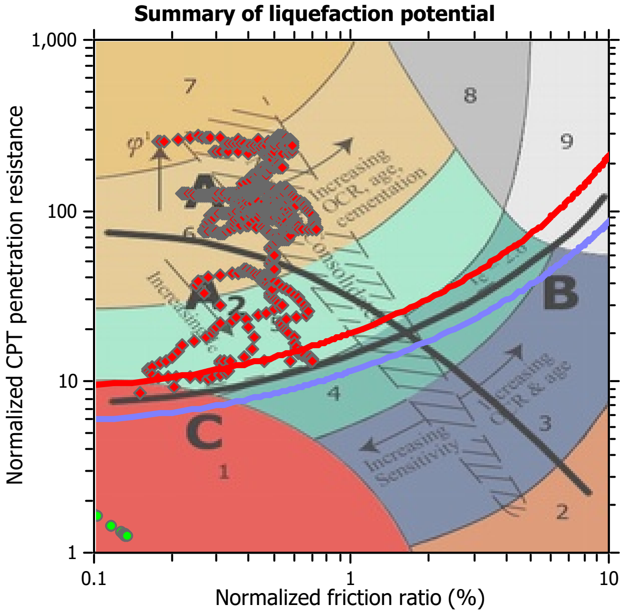

Friction Ratio


SBTn Plot


CRR plot

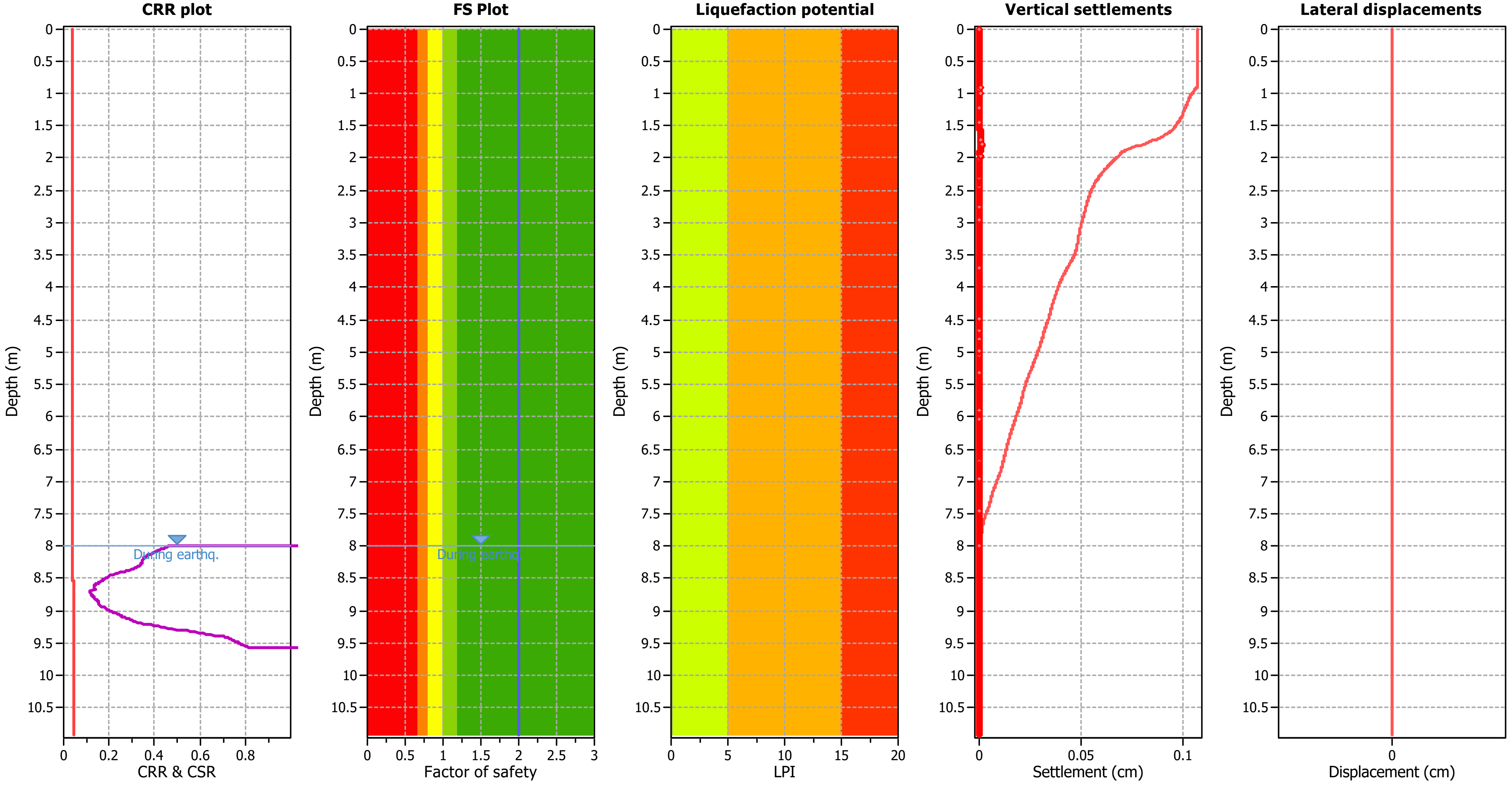

FS Plot


$M_w=7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

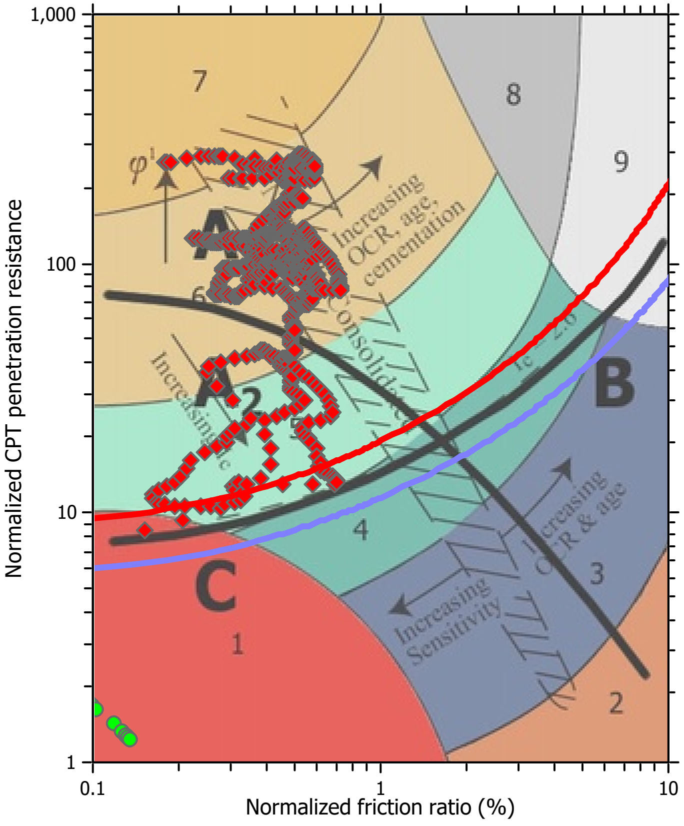
F.S. color scheme

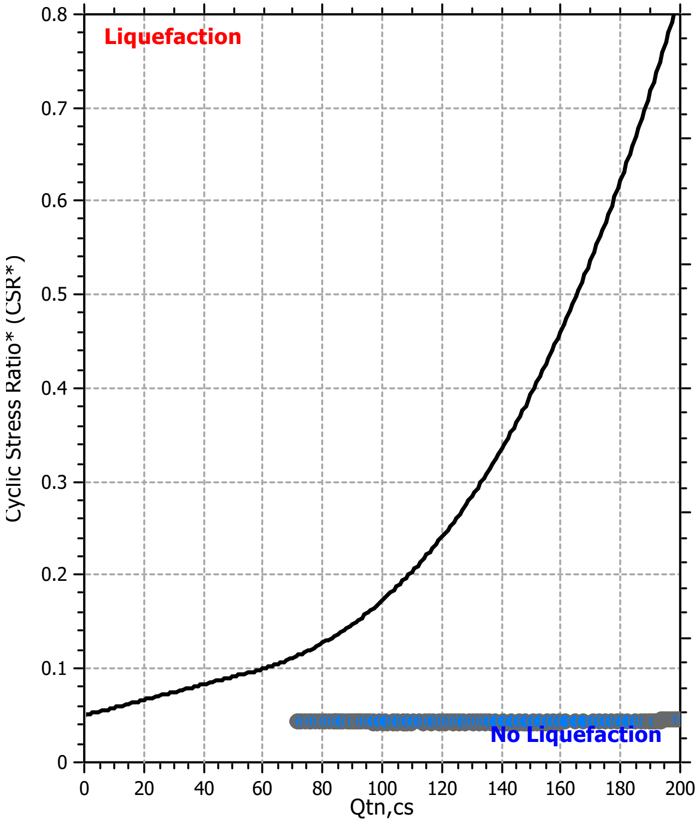
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

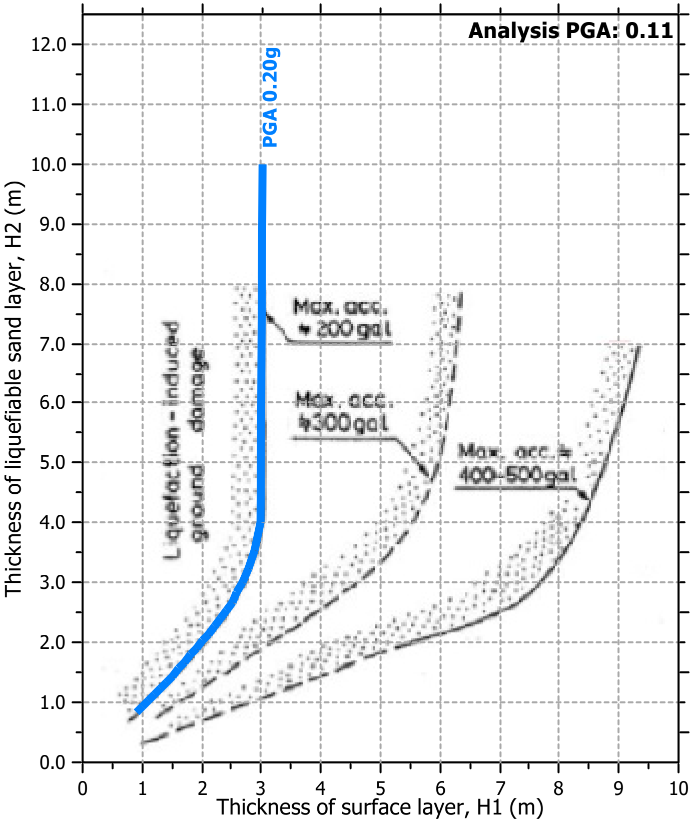
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

LIQUEFACTION ANALYSIS REPORT

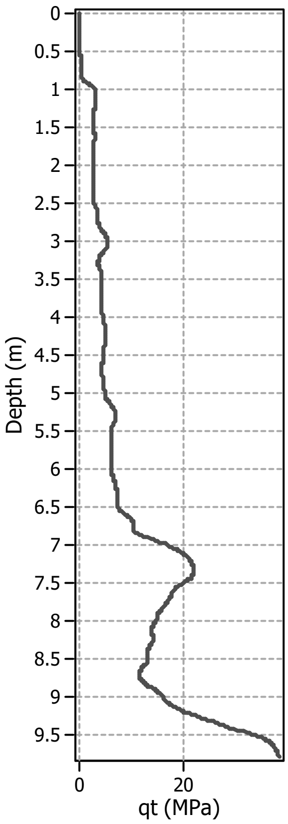
Project title : 38 Stockton & 8A Tomaree Street **Location : Nelson Bay**

CPT file : CPT2

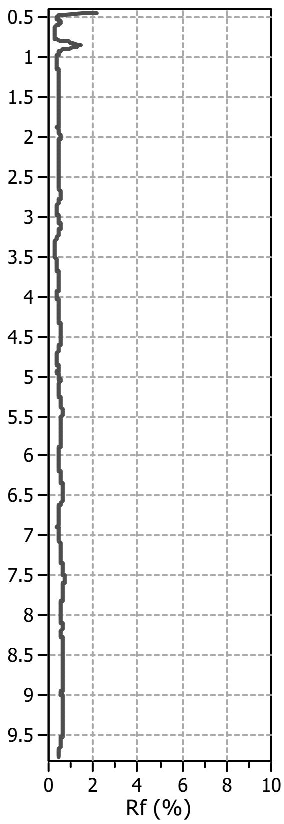
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	12.00 m	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.11	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

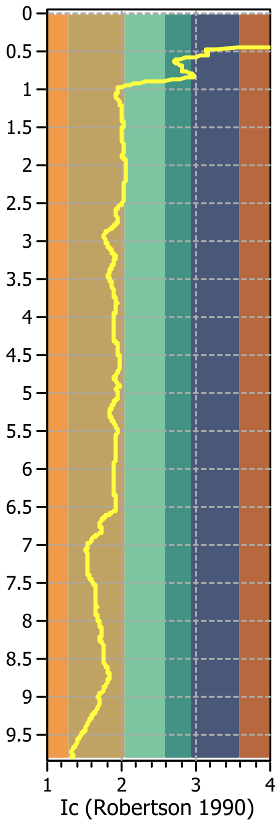
Cone resistance



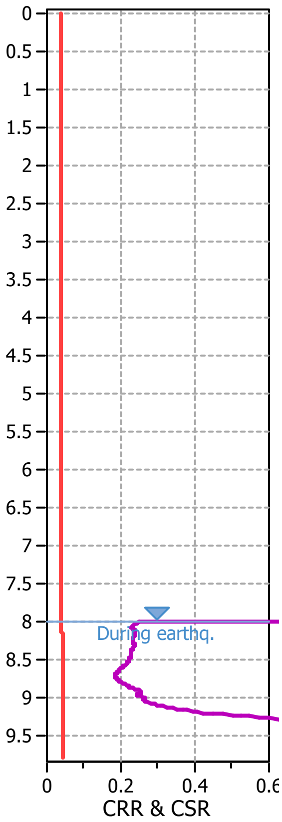
Friction Ratio



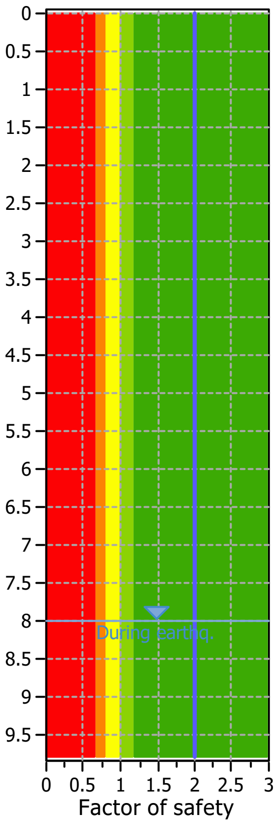
SBTn Plot



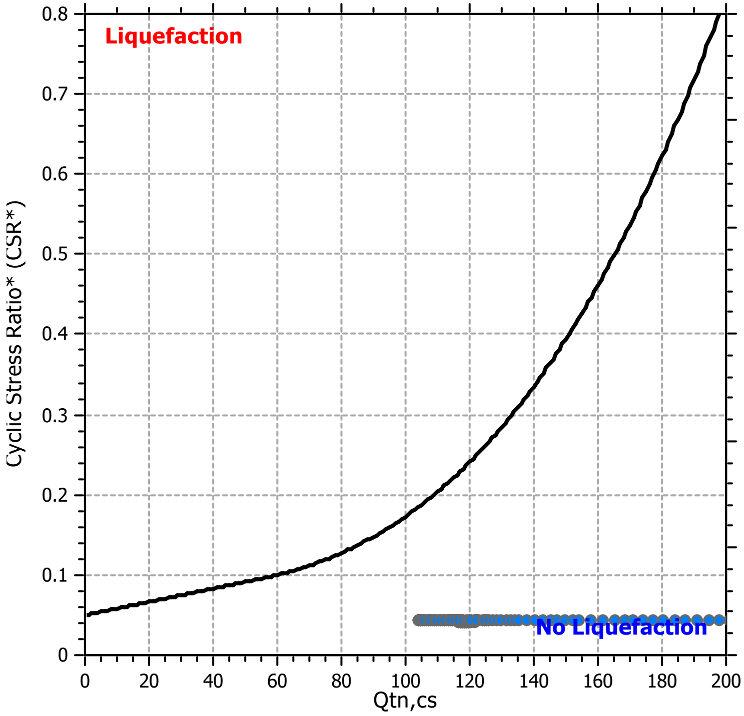
CRR plot



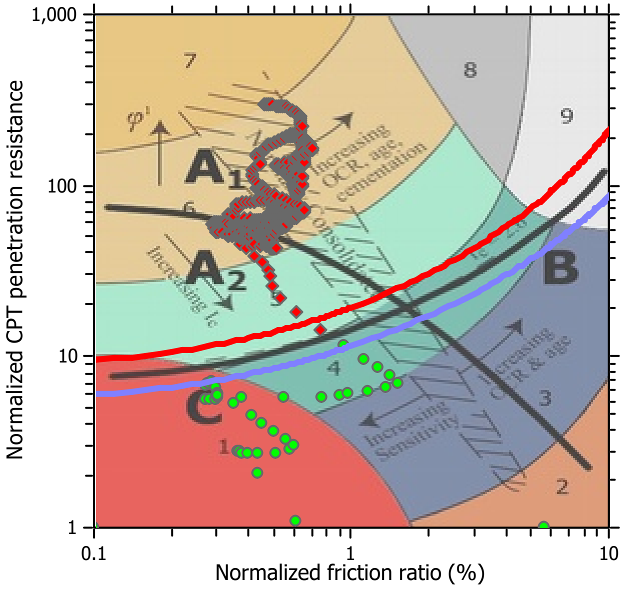
FS Plot



M_w=$7^{1/2}$, sigma'=1 atm base curve

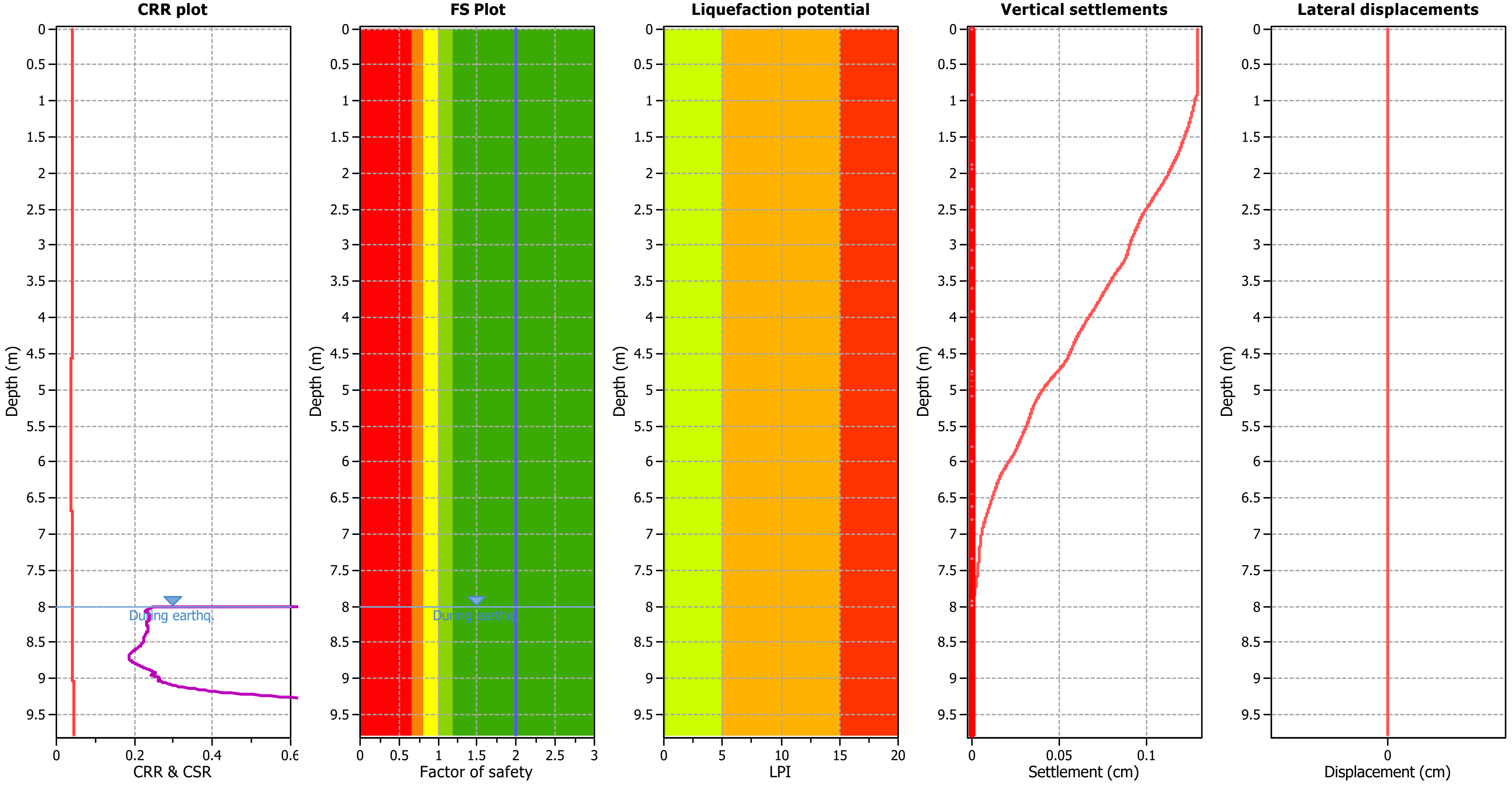


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

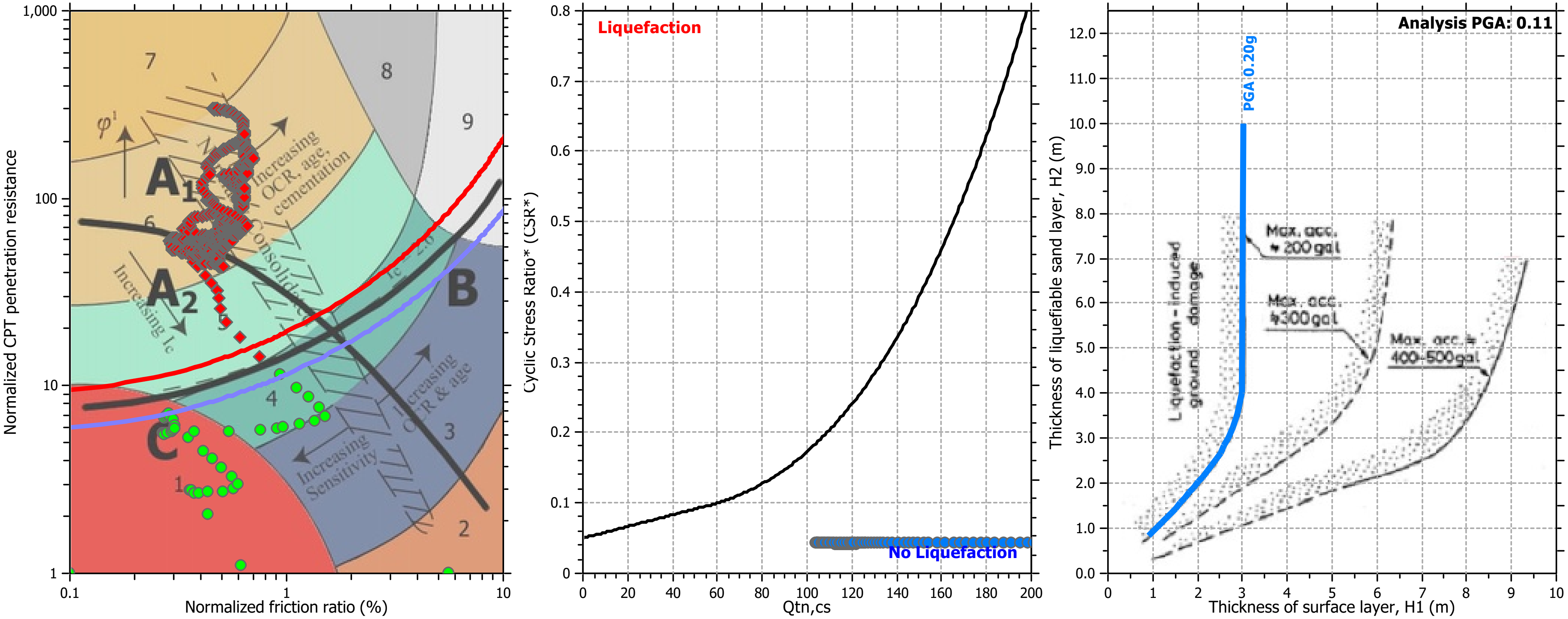
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

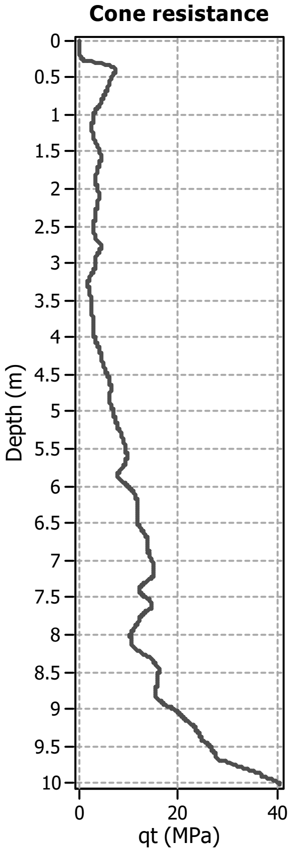
LIQUEFACTION ANALYSIS REPORT

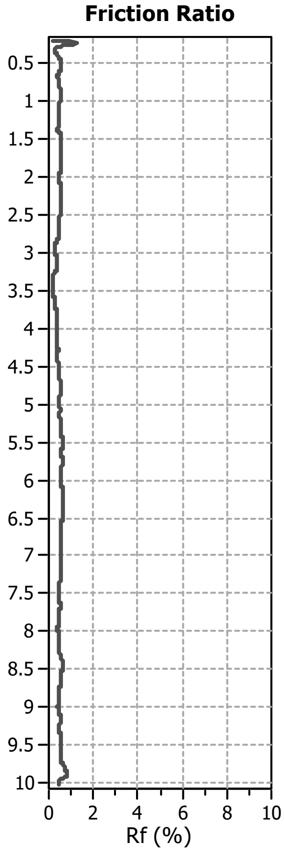
Project title : 38 Stockton & 8A Tomaree Street **Location : Nelson Bay**

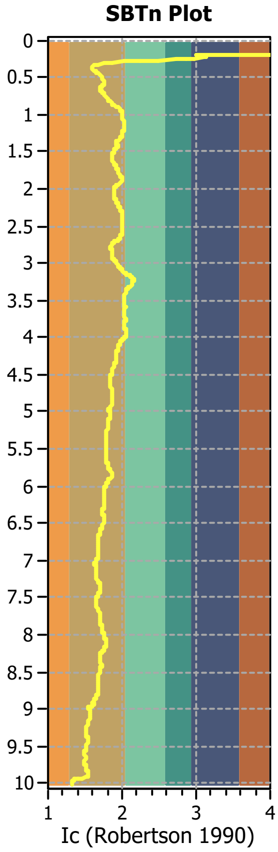
CPT file : CPT3

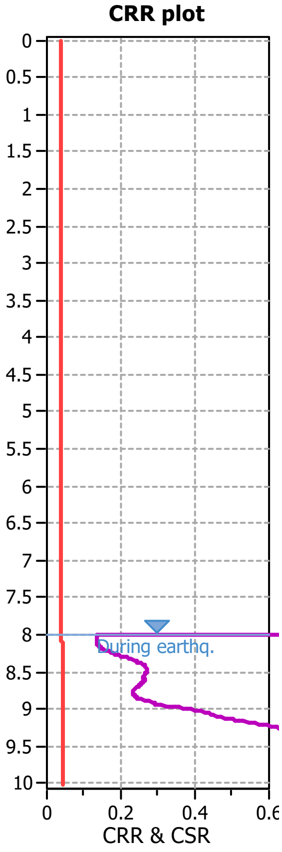
Input parameters and analysis data

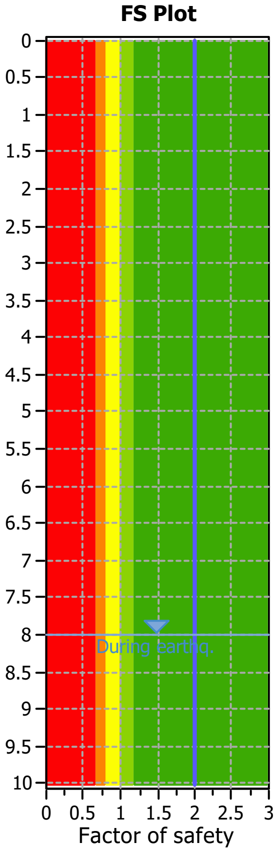
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	12.00 m	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.11	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

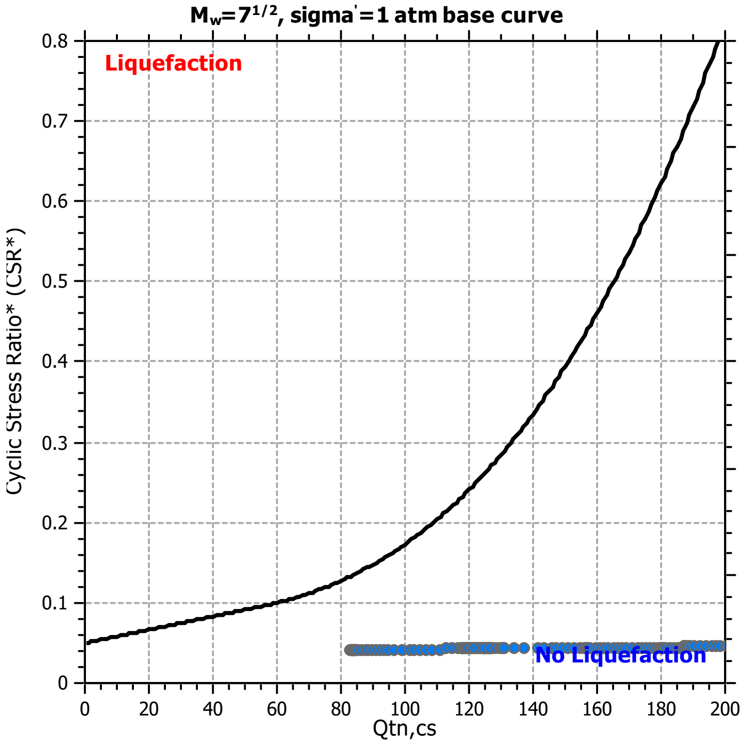


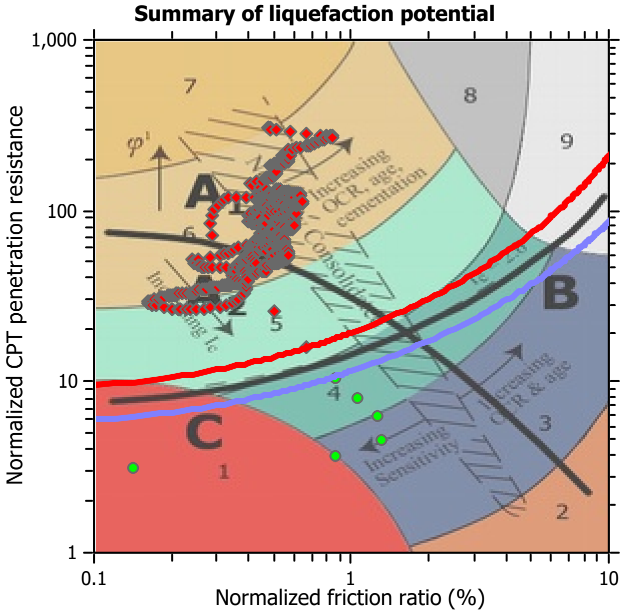






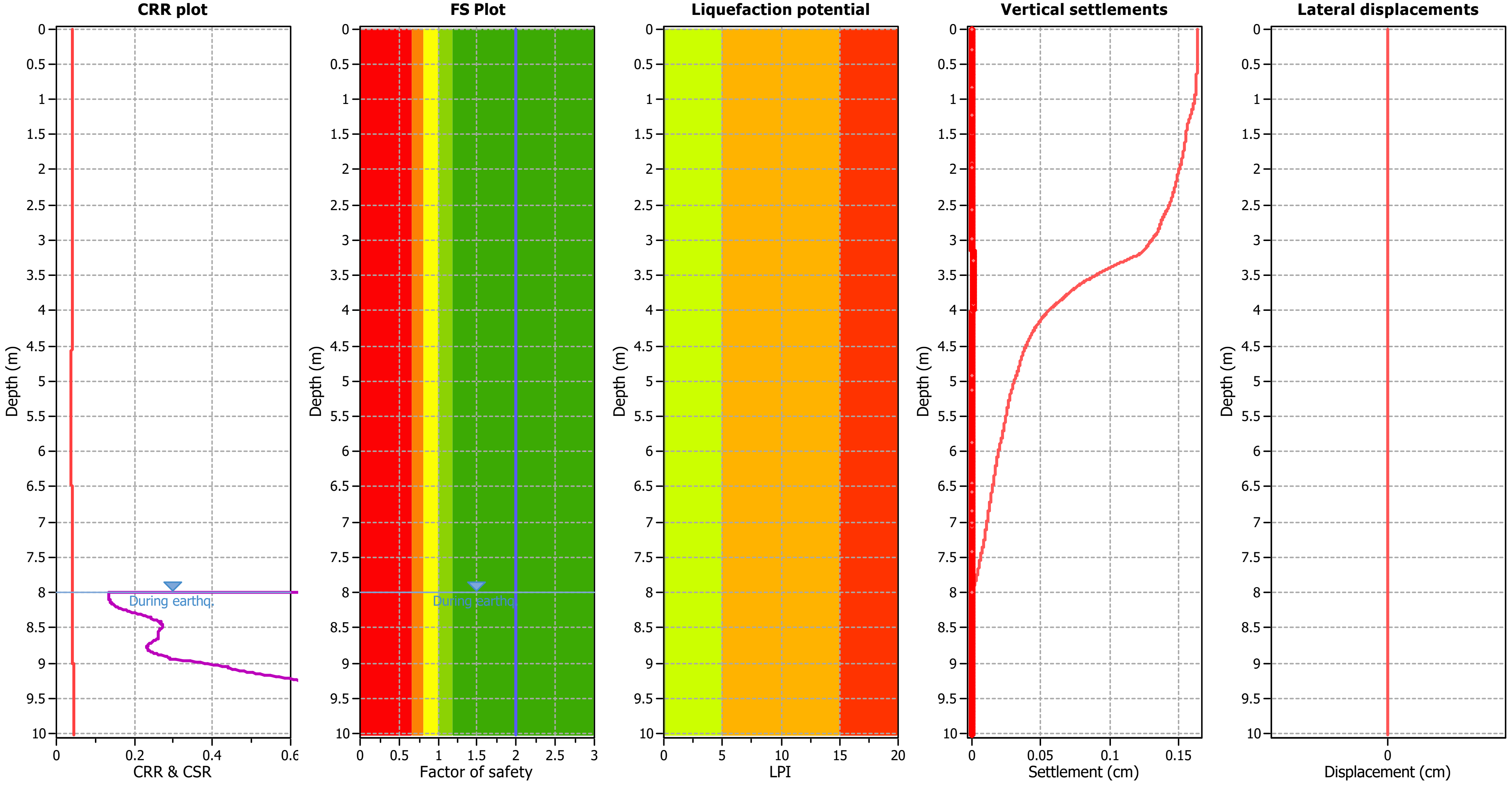






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

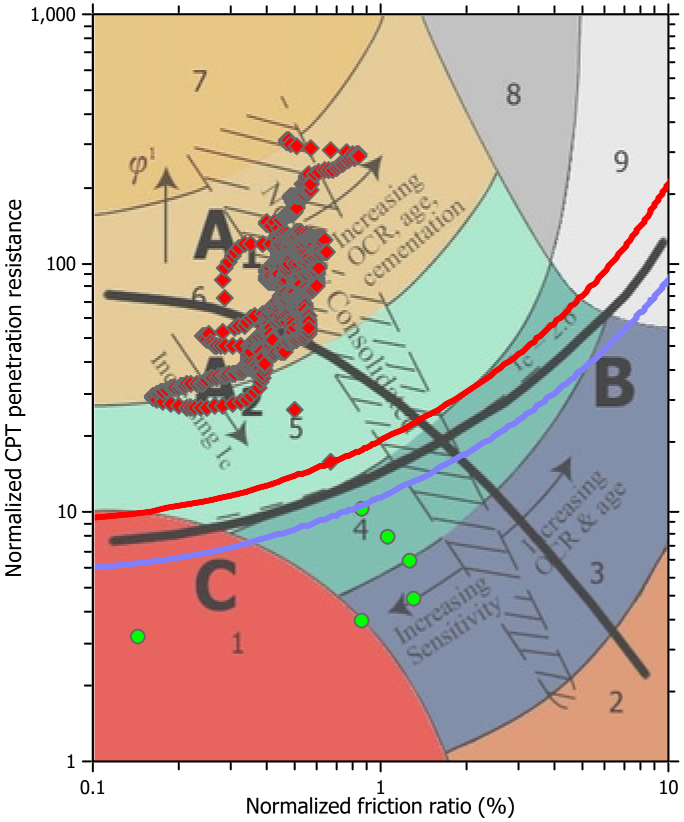
F.S. color scheme

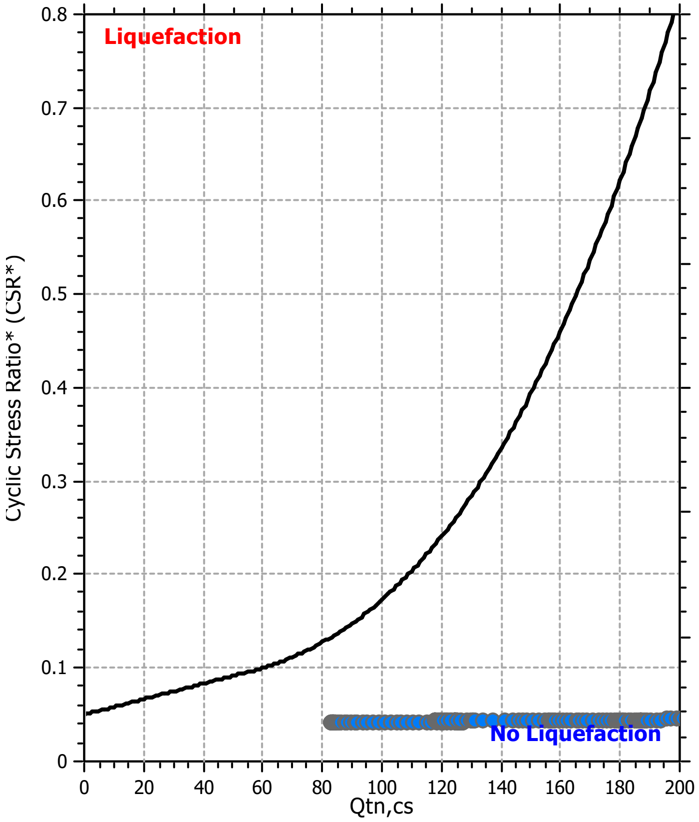
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

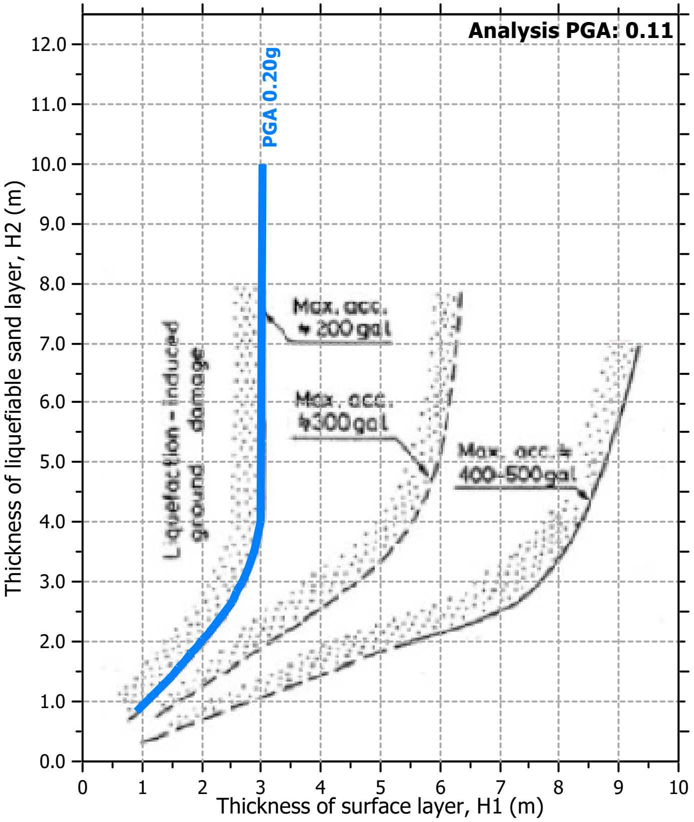
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots







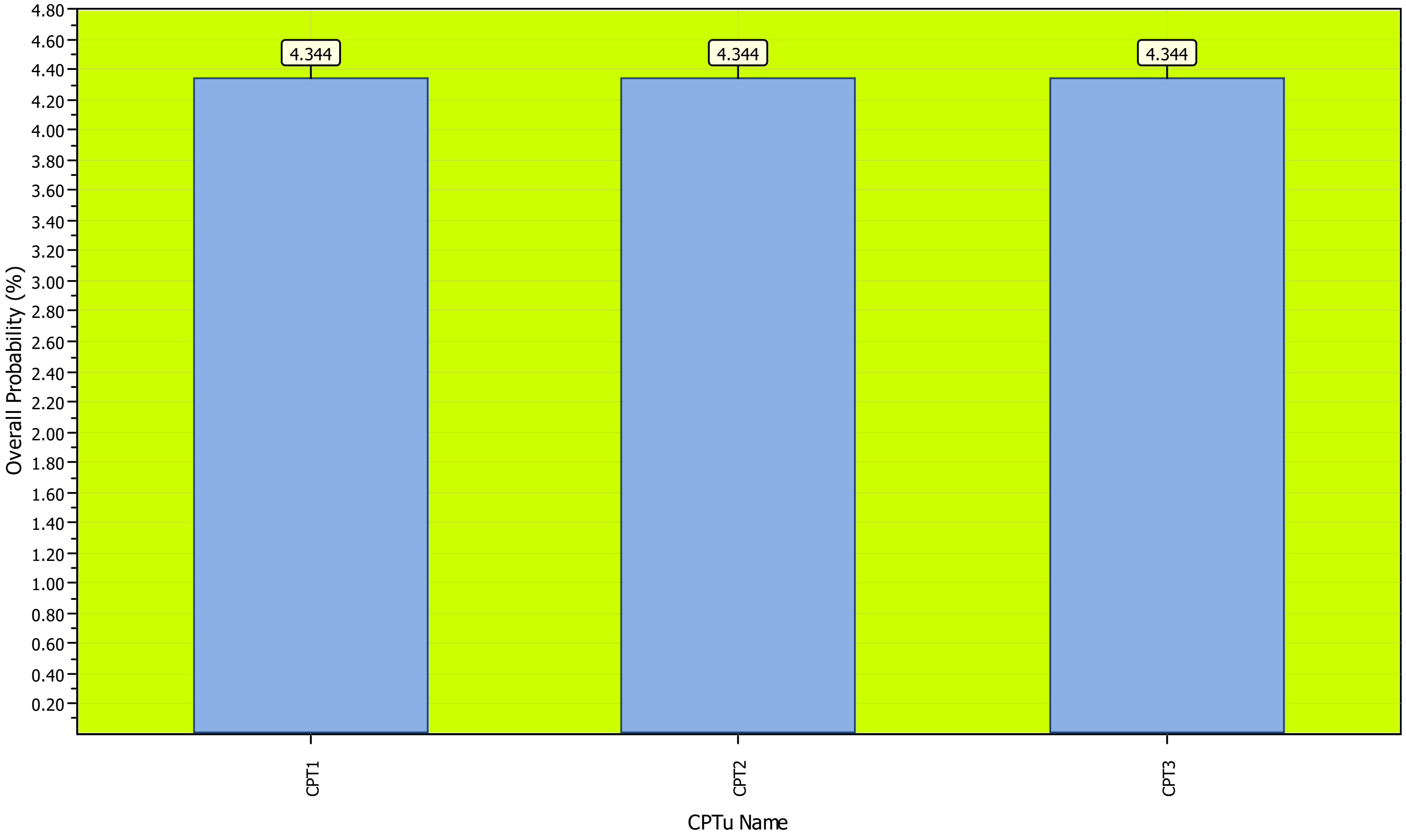
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 m	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.11	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 m	Fill height:	N/A	Limit depth:	N/A

Project title :

Location :

Overall Probability for Liquefaction report



Probability color scheme

🟥 Very High Probability

🟧 High Probability

🟩 Low Probability

Basic statistics

Total CPT number: 3

100% low probability

0% high probability

0% very high probability

APPENDIX E: PERMEABILITY TEST RESULTS

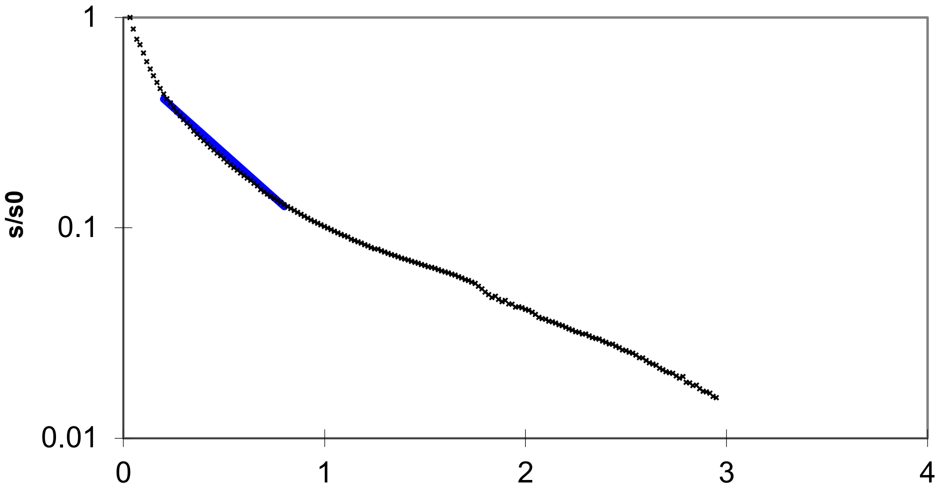
RISING OR FALLING HEAD TEST ANALYSIS

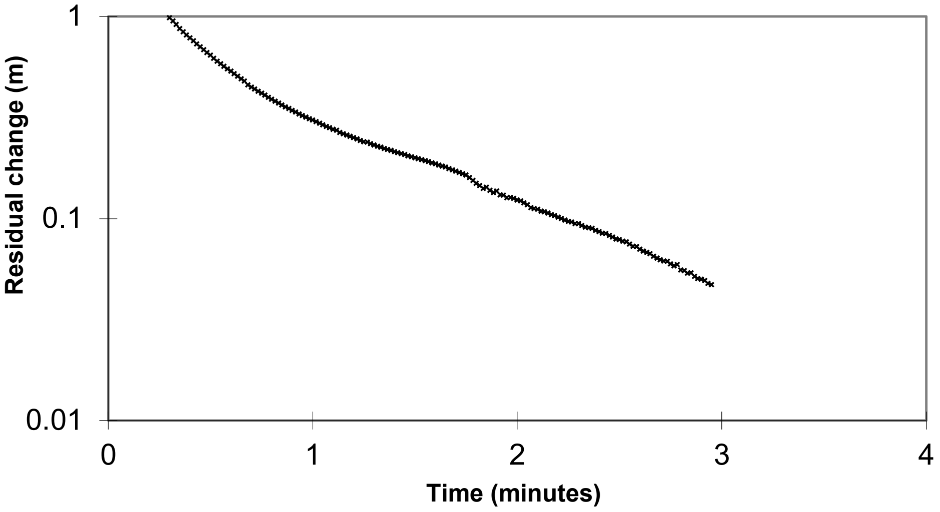
Bore Data	Units	Value
Initial groundwater level	m	3.02
Groundwater level at t=0	m	0.000
Casing radius (r)	m	0.03
Bore radius (R)	m	0.0625
Screened interval length (L	m	3
Match time start	min	0.2
Match time end	min	0.8
Characteristic Time (t_0)	**min**	**0.52**
Hydraulic Conductivity (K)	**m/day**	**1.62**
Hydraulic Conductivity (K)	**m/sec**	**2E-05**

Piezometer: S-BH1

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	TETRA TECH	client:	COHO Property
approved	SB		project:	Development for 36 Stockton St and 8A Tomaree St, Nelson Bay
date	11 Oct 2024			
scale	AS SHOWN		title:	Falling Head Test S-BH1
original size	A4		project no:	754-NTLGE368007 Test 1

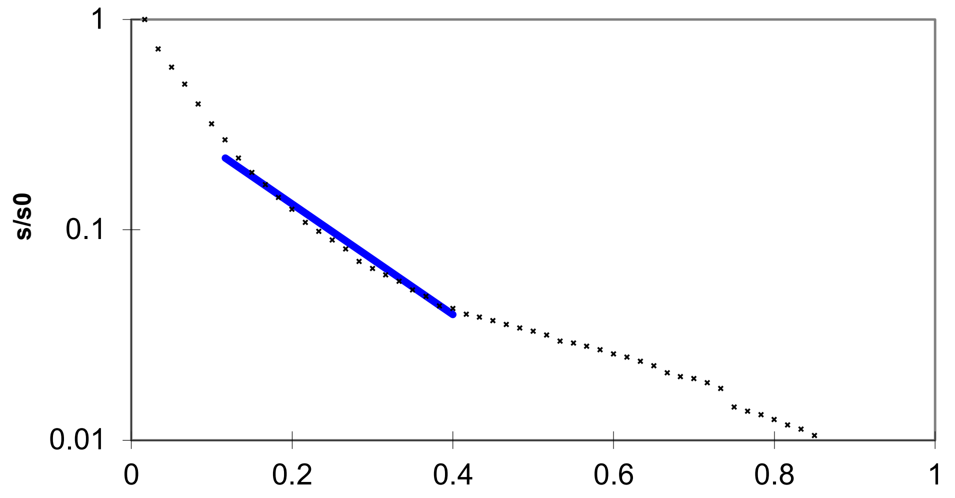
RISING OR FALLING HEAD TEST ANALYSIS

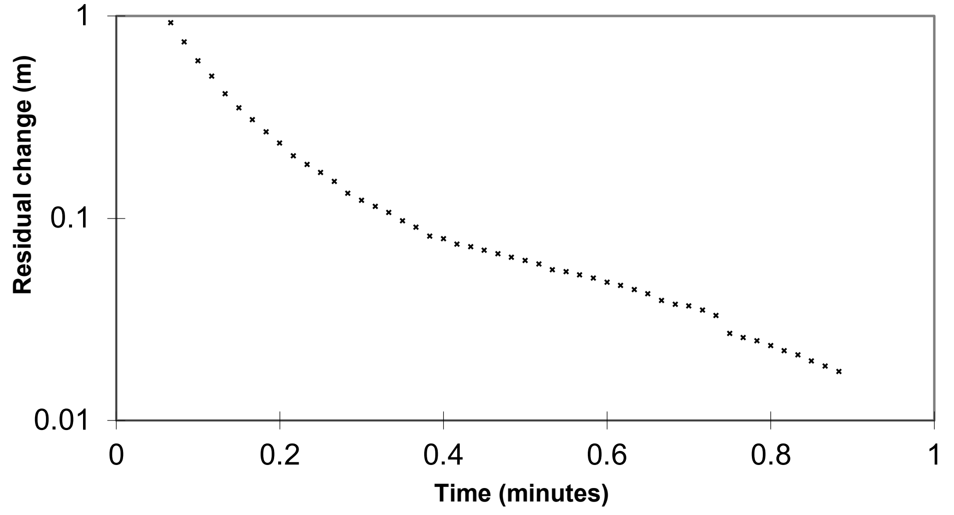
Bore Data		Units	Value
Initial groundwater level		m	2.83
Groundwater level at t=0		m	0.953
Casing radius	(r)	m	0.03
Bore radius	(R)	m	0.0625
Screened interval length	(L	m	3
Match time start		min	0.13
Match time end		min	0.4
Characteristic Time (t_0)		**min**	**0.17**
Hydraulic Conductivity (K)		**m/day**	**5.00**
Hydraulic Conductivity (K)		**m/sec**	**6E-05**

Piezometer: S-BH1

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**	TETRA TECH	client:	**COHO Property**
approved	**SB**		project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	**11 Oct 2024**		title:	**Falling Head Test S-BH1**
scale	**AS SHOWN**		project no:	**754-NTLGE368007** **Test 2**
original size	**A4**			

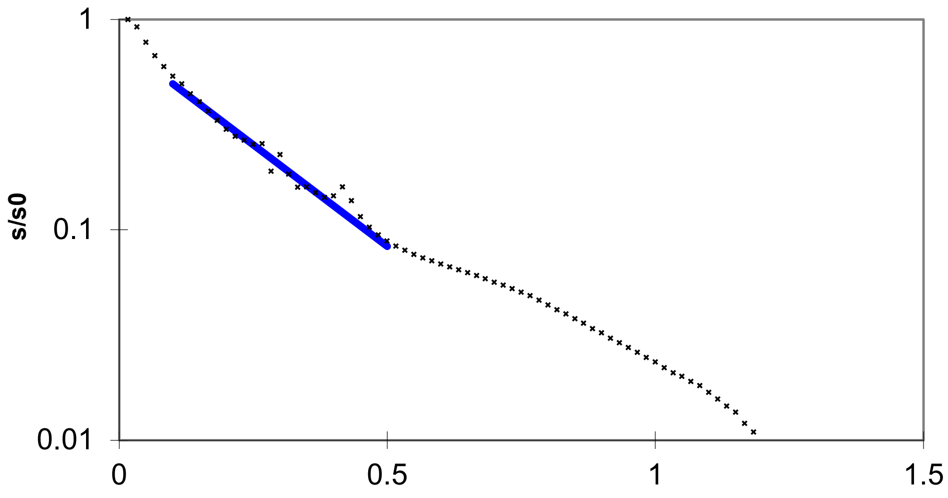
RISING OR FALLING HEAD TEST ANALYSIS

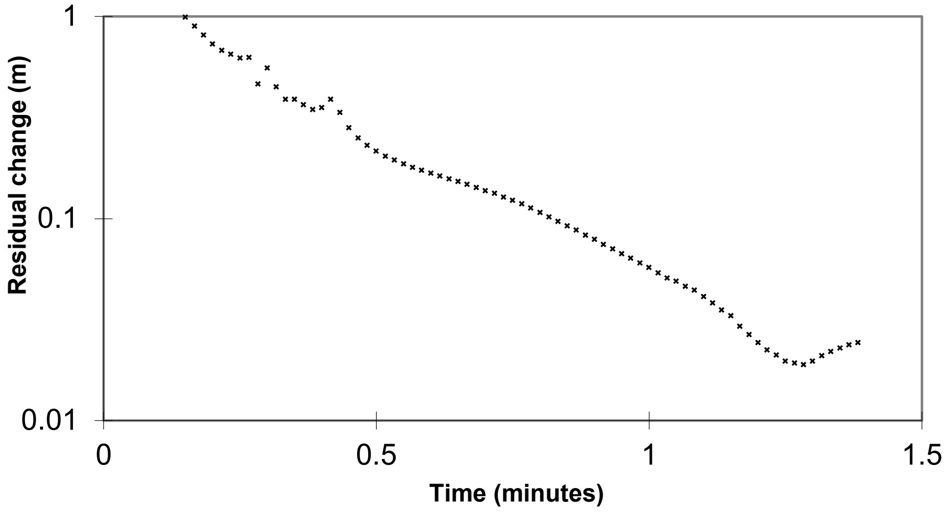
Bore Data	**Units**	**Value**
Initial groundwater level	m	2.88
Groundwater level at t=0	m	0.443
Casing radius (r)	m	0.03
Bore radius (R)	m	0.0625
Screened interval length (L	m	3
Match time start	min	0.1
Match time end	min	0.5
Characteristic Time (t_0)	**min**	**0.24**
Hydraulic Conductivity (K)	**m/day**	**3.48**
Hydraulic Conductivity (K)	**m/sec**	**4E-05**

Piezometer: S-BH1

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**
approved	**SB**
date	**11 Oct 2024**
scale	**AS SHOWN**
original size	**A4**

TETRA TECH

client:	**COHO Property**	
project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**	
title:	**Falling Head Test S-BH1**	
project no:	**754-NTLGE368007**	**Test 3**

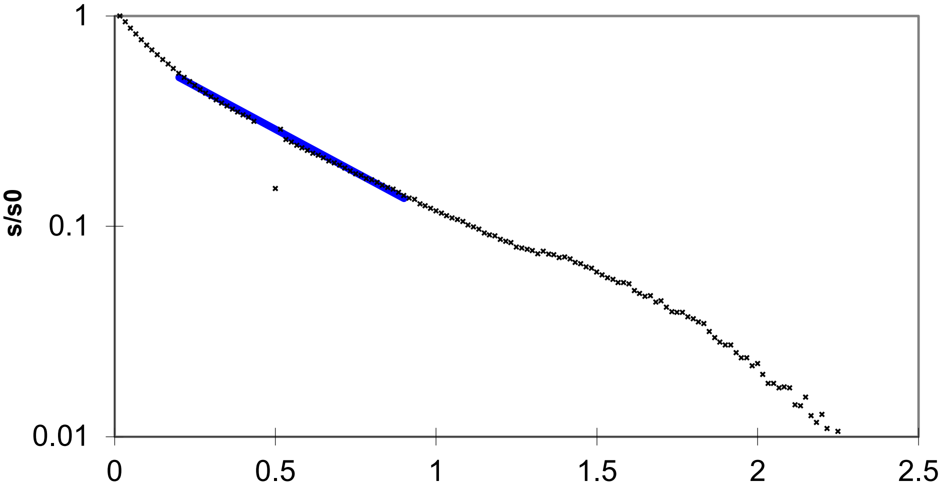
RISING OR FALLING HEAD TEST ANALYSIS

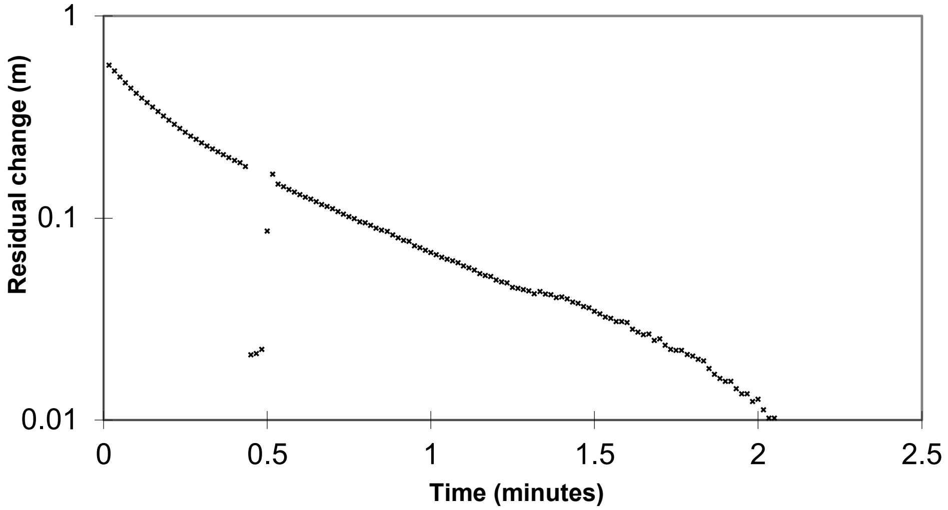
Bore Data	Units	Value
Initial groundwater level	m	3
Groundwater level at t=0	m	2.431
Casing radius (r)	m	0.03
Bore radius (R)	m	0.0625
Screened interval length (L	m	3
Match time start	min	0.2
Match time end	min	0.9
Characteristic Time (t_0)	**min**	**0.32**
Hydraulic Conductivity (K)	**m/day**	**2.63**
Hydraulic Conductivity (K)	**m/sec**	**3E-05**

Piezometer: S-BH2

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**	TETRA TECH	client:	**COHO Property**	
approved	**SB**		project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**	
date	**11 Oct 2024**				
scale	**AS SHOWN**		title:	**Falling Head Test S-BH2**	
original size	**A4**		project no:	**754-NTLGE368007**	**Test 1**

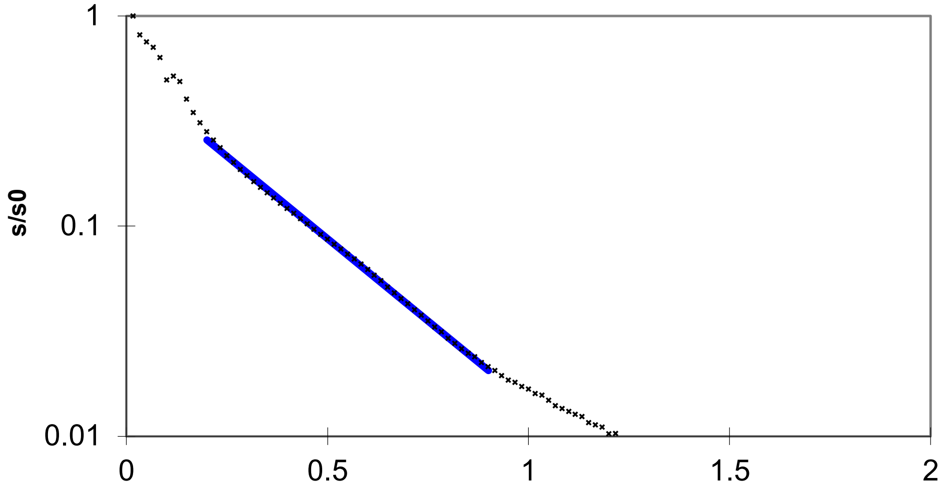
RISING OR FALLING HEAD TEST ANALYSIS

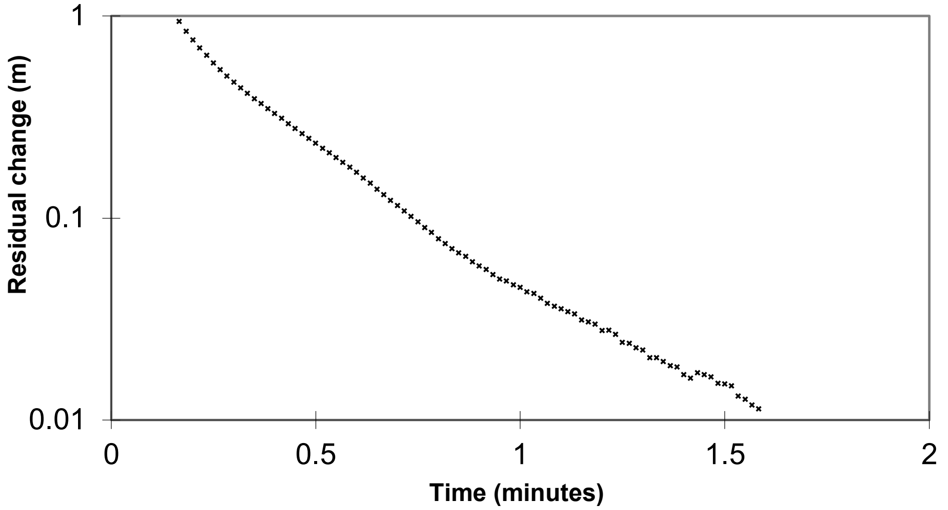
Bore Data	Units	Value
Initial groundwater level	m	3.18
Groundwater level at t=0	m	0.474
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.2
Match time end	min	0.9
Characteristic Time (t_0)	**min**	**0.28**
Hydraulic Conductivity (K)	**m/day**	**3.16**
Hydraulic Conductivity (K)	**m/sec**	**4E-05**

Piezometer: S-BH2

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**	client:	**COHO Property**
approved	**SB**	project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	**11 Oct 2024**	title:	**Falling Head Test S-BH2**
scale	**AS SHOWN**	project no:	**754-NTLGE368007** **Test 2**
original size	**A4**		

TETRA TECH

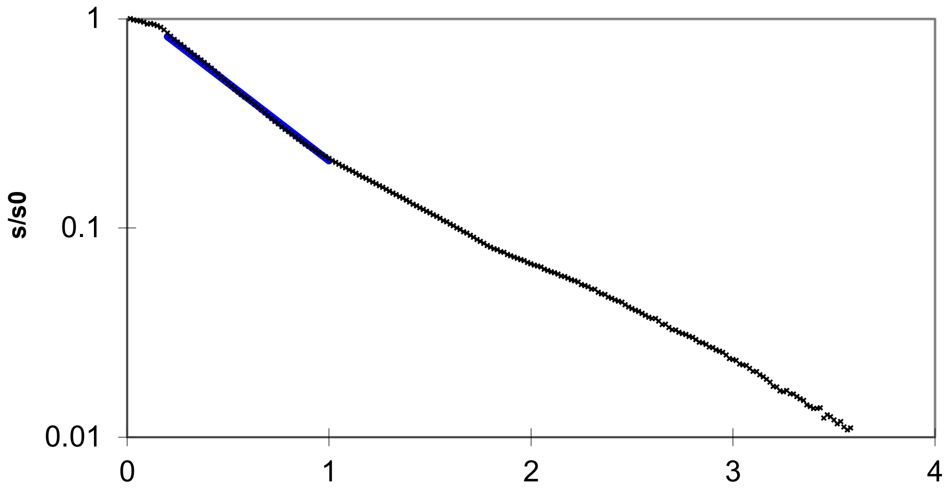
RISING OR FALLING HEAD TEST ANALYSIS

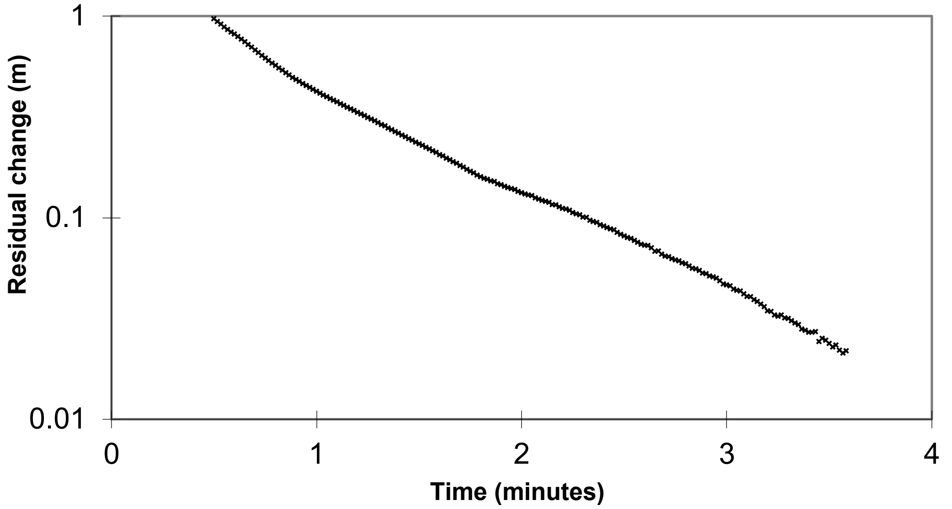
Bore Data	Units	Value
Initial groundwater level	m	3
Groundwater level at t=0	m	1.028
Casing radius (r)	m	0.03
Bore radius (R)	m	0.0625
Screened interval length (L	m	3
Match time start	min	0.2
Match time end	min	1
Characteristic Time (t_0)	**min**	**0.57**
Hydraulic Conductivity (K)	**m/day**	**1.47**
Hydraulic Conductivity (K)	**m/sec**	**2E-05**

Piezometer: S-BH2

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**	client:	**COHO Property**
approved	**SB**	project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	**11 Oct 2024**		
scale	**AS SHOWN**	title:	**Falling Head Test S-BH2**
original size	**A4**	project no:	**754-NTLGE368007** **Test 3**

TETRA TECH

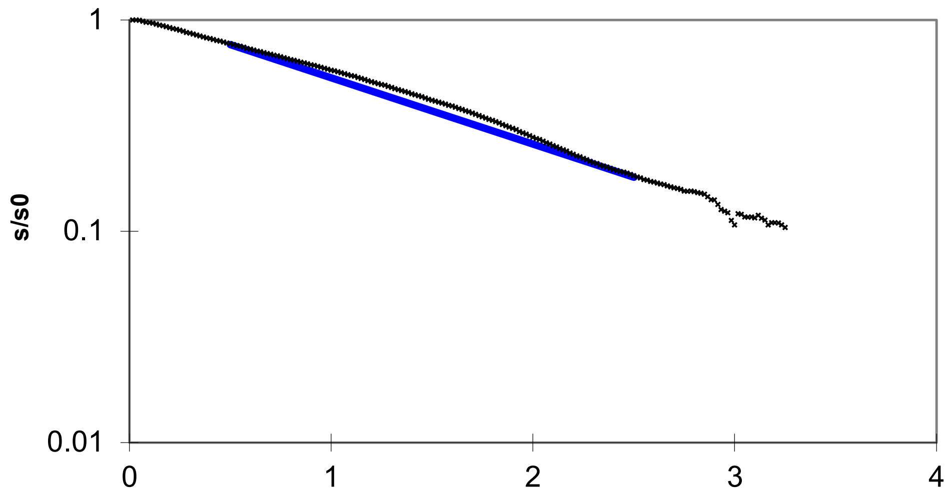
RISING OR FALLING HEAD TEST ANALYSIS

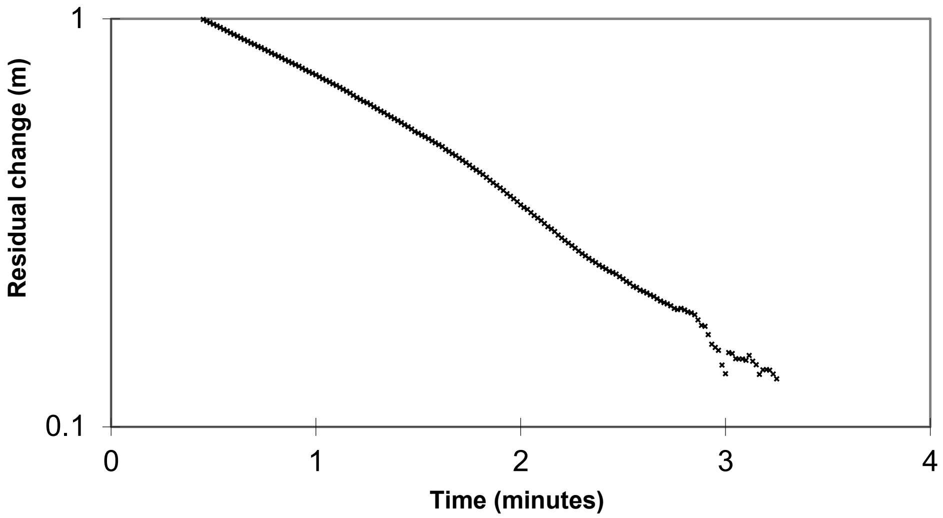
Bore Data	Units	Value
Initial groundwater level	m	5.4
Groundwater level at t=0	m	4.143
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.5
Match time end	min	2.5
Characteristic Time (t_0)	**min**	**1.37**
Hydraulic Conductivity (K)	**m/day**	**0.65**
Hydraulic Conductivity (K)	**m/sec**	**8E-06**

Piezometer: D-BH01

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	**KF**	client:	**COHO Property**
approved	**SB**	project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	**3 Oct 2024**	title:	**Falling Head Test D-BH01**
scale	**AS SHOWN**	project no:	**754-NTLGE368007** **Test 1**
original size	**A4**		

TETRA TECH

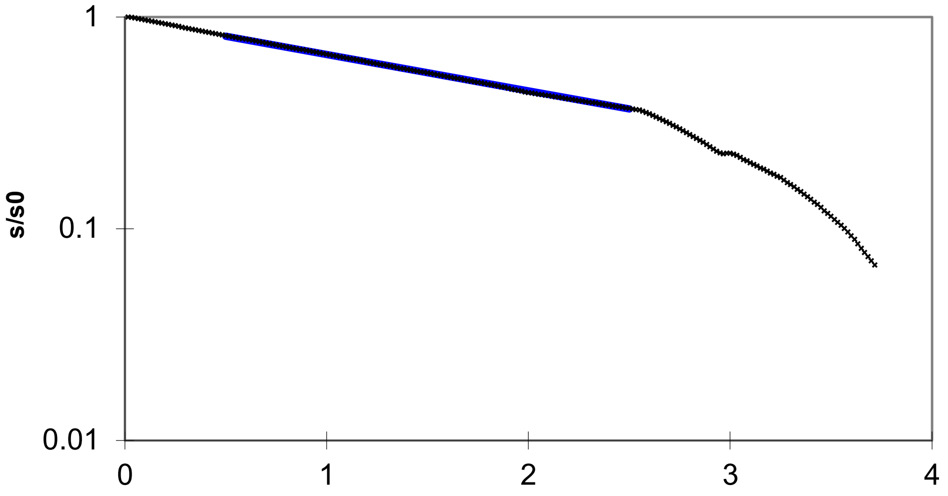
RISING OR FALLING HEAD TEST ANALYSIS

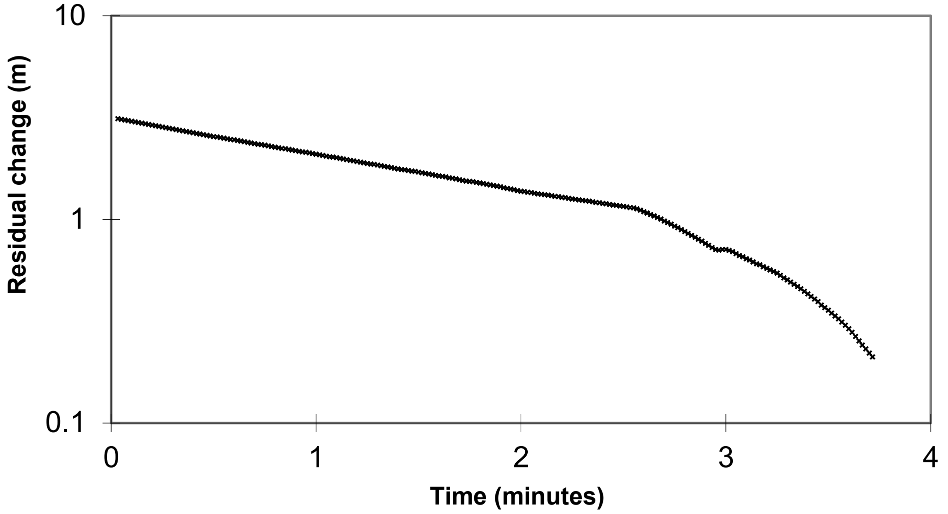
Bore Data	Units	Value
Initial groundwater level	m	4.3
Groundwater level at t=0	m	1.171
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.5
Match time end	min	2.5
Characteristic Time (t_0)	**min**	**2.48**
Hydraulic Conductivity (K)	**m/day**	**0.36**
Hydraulic Conductivity (K)	**m/sec**	**4E-06**

Piezometer: D-BH01

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	client:	COHO Property
approved	SB	project:	Development for 36 Stockton St and 8A Tomaree St, Nelson Bay
date	3 Oct 2024	title:	Falling Head Test D-BH01
scale	AS SHOWN	project no:	754-NTLGE368007 Test 2
original size	A4		

TETRA TECH

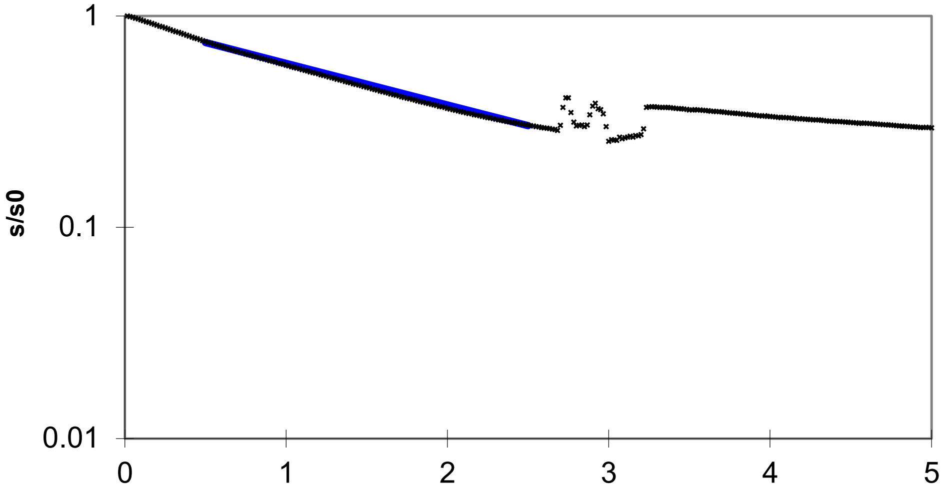
RISING OR FALLING HEAD TEST ANALYSIS

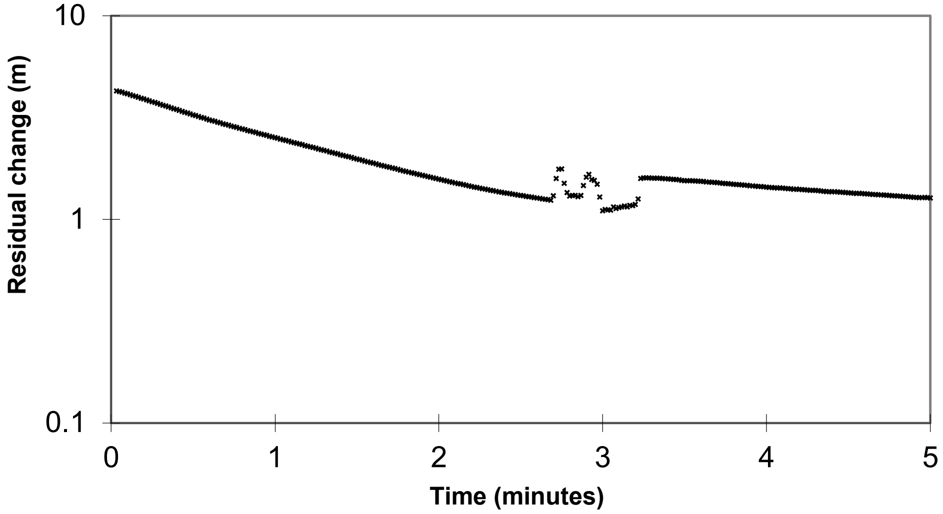
Bore Data	Units	Value
Initial groundwater level	m	10.9
Groundwater level at t=0	m	6.590
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.5
Match time end	min	2.5
Characteristic Time (t_0)	**min**	**2.17**
Hydraulic Conductivity (K)	**m/day**	**0.41**
Hydraulic Conductivity (K)	**m/sec**	**5E-06**

Piezometer: D-BH01

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	TETRA TECH	client:	COHO Property
approved	SB		project:	Development for 36 Stockton St and 8A Tomaree St, Nelson Bay
date	3 Oct 2024			
scale	AS SHOWN		title:	Falling Head Test D-BH01
original size	A4		project no:	754-NTLGE368007 Test 3

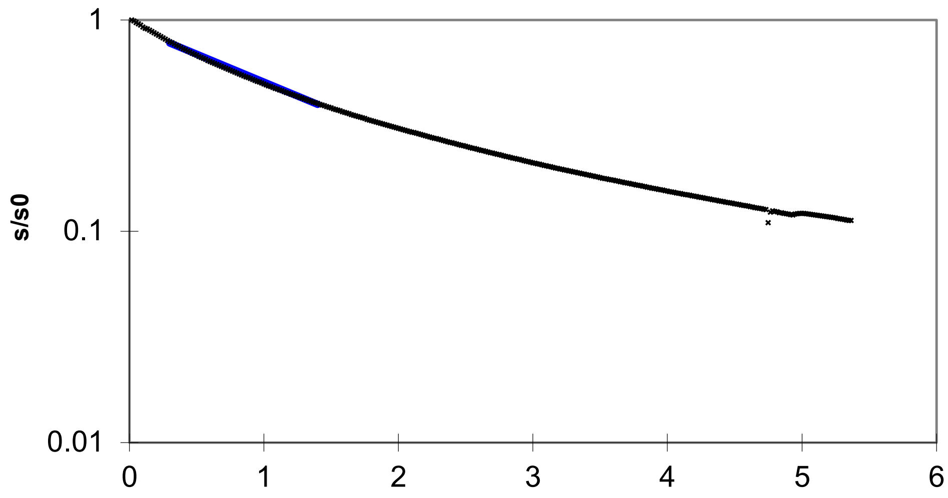
RISING OR FALLING HEAD TEST ANALYSIS

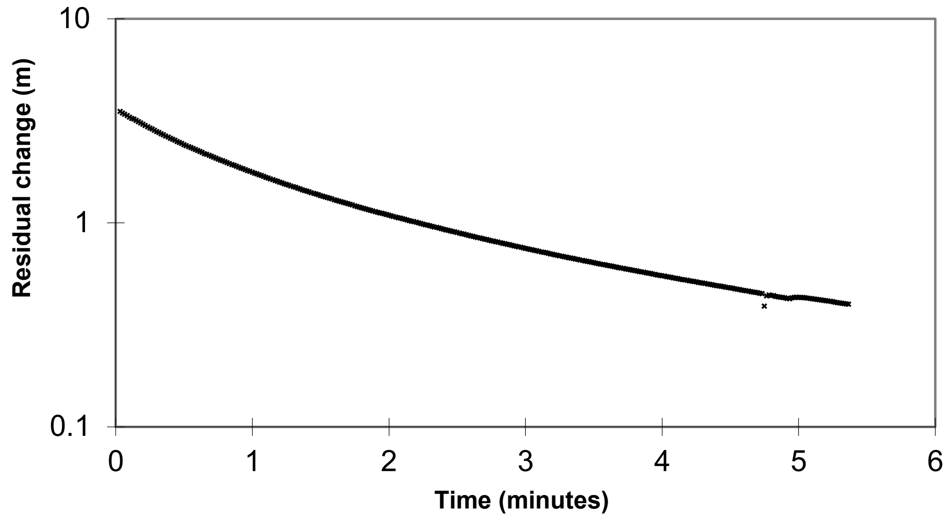
Bore Data	Units	Value
Initial groundwater level	m	10.5
Groundwater level at t=0	m	6.948
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.3
Match time end	min	1.4
Characteristic Time (t_0)	**min**	**1.66**
Hydraulic Conductivity (K)	**m/day**	**0.54**
Hydraulic Conductivity (K)	**m/sec**	**6E-06**

Piezometer: D-BH02

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	
approved	SB	
date	3 Oct 2024	
scale	AS SHOWN	
original size	A4	

TETRA TECH

client: **COHO Property**

project: **Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**

title: **Falling Head Test D-BH02**

project no: **754-NTLGE368007** **Test 1**

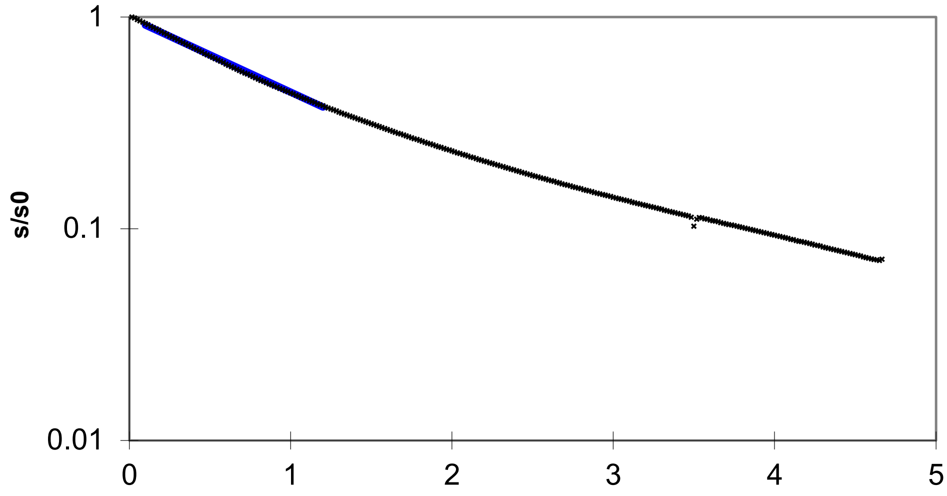
RISING OR FALLING HEAD TEST ANALYSIS

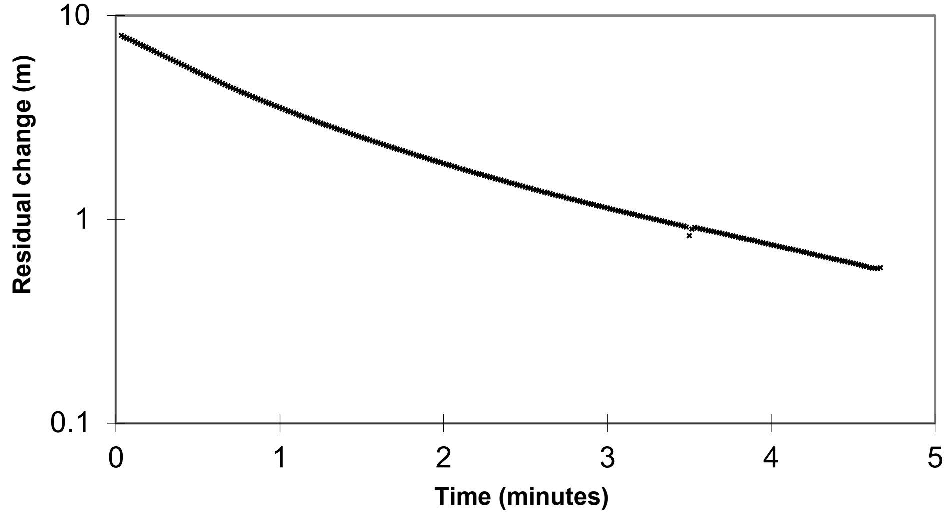
Bore Data	Units	Value
Initial groundwater level	m	10.2
Groundwater level at t=0	m	2.123
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.1
Match time end	min	1.2
Characteristic Time (t_0)	**min**	**1.22**
Hydraulic Conductivity (K)	**m/day**	**0.73**
Hydraulic Conductivity (K)	**m/sec**	**8E-06**

Piezometer: D-BH02

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	client:	COHO Property
approved	SB	project:	Development for 36 Stockton St and 8A Tomaree St, Nelson Bay
date	3 Oct 2024	title:	Falling Head Test D-BH02
scale	AS SHOWN	project no:	754-NTLGE368007 Test 2
original size	A4		

TETRA TECH

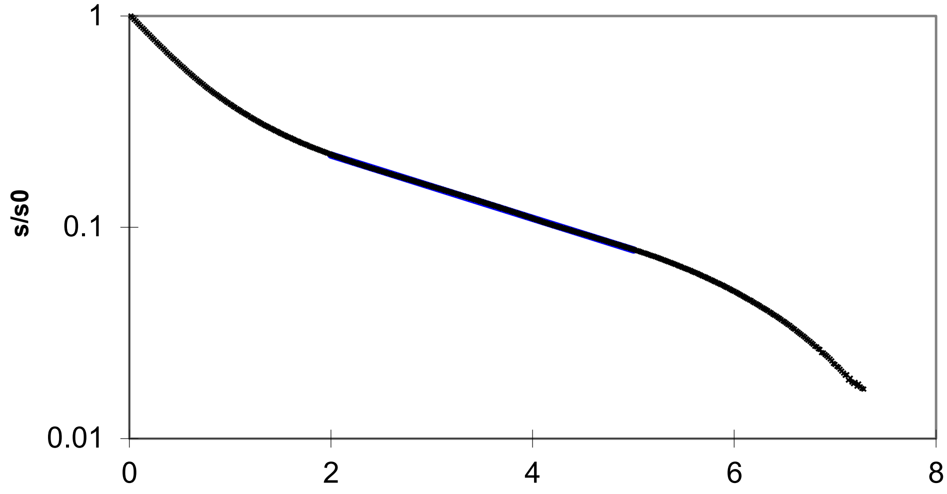
RISING OR FALLING HEAD TEST ANALYSIS

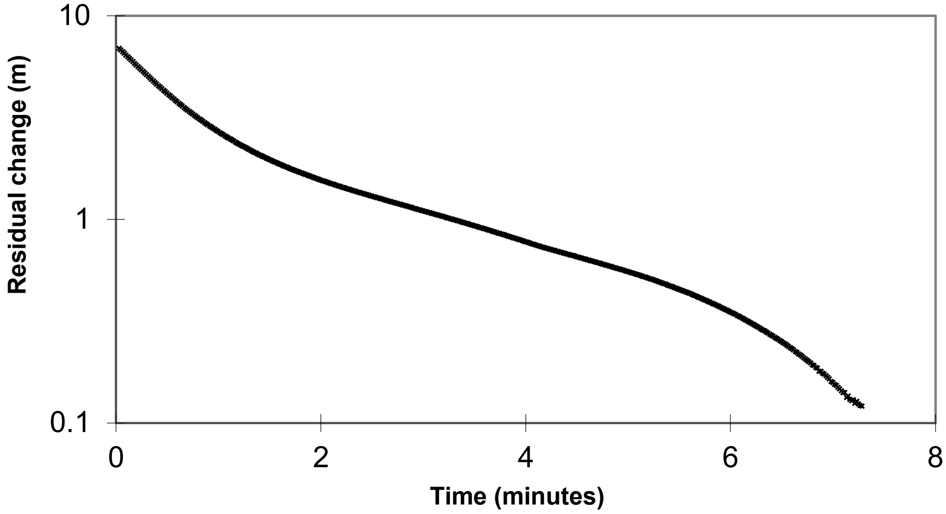
Bore Data	**Units**	**Value**
Initial groundwater level	m	11
Groundwater level at t=0	m	3.943
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	2
Match time end	min	5
Characteristic Time (t_0)	**min**	**2.91**
Hydraulic Conductivity (K)	**m/day**	**0.31**
Hydraulic Conductivity (K)	**m/sec**	**4E-06**

Piezometer: D-BH02

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	client:	**COHO Property**
approved	SB	project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	3 Oct 2024	title:	**Falling Head Test D-BH02**
scale	AS SHOWN	project no:	**754-NTLGE368007** **Test 3**
original size	A4		

TETRA TECH

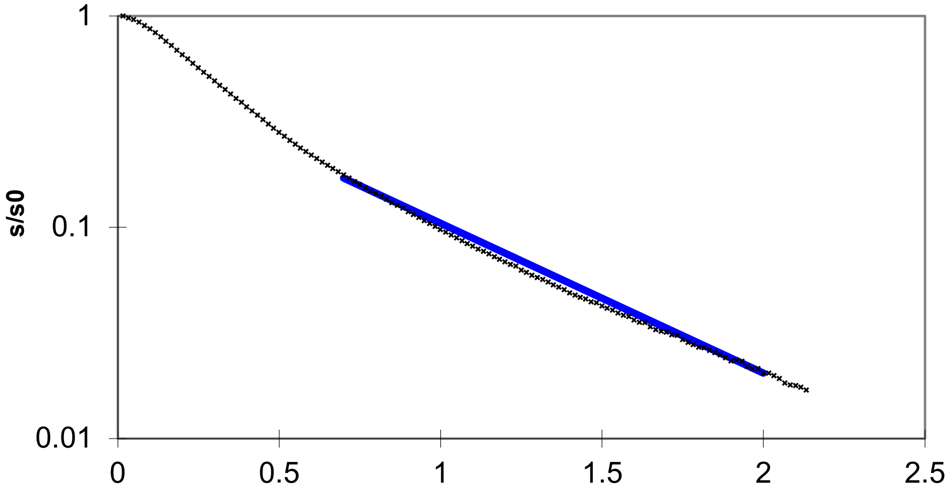
RISING OR FALLING HEAD TEST ANALYSIS

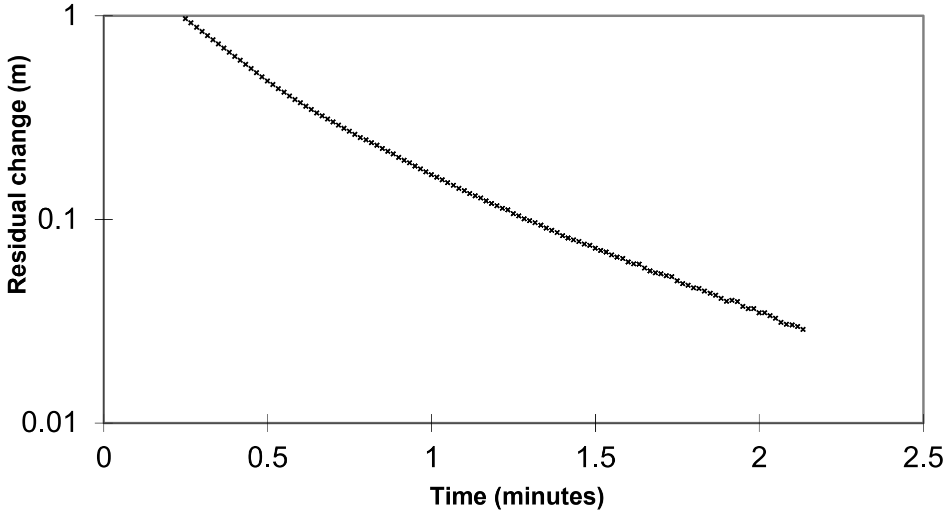
Bore Data	Units	Value
Initial groundwater level	m	11.51
Groundwater level at t=0	m	9.809
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.7
Match time end	min	2
Characteristic Time (t_0)	**min**	**0.62**
Hydraulic Conductivity (K)	**m/day**	**1.44**
Hydraulic Conductivity (K)	**m/sec**	**2E-05**

Piezometer: GW

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF
approved	SB
date	3 Oct 2024
scale	AS SHOWN
original size	A4

TETRA TECH

client: **COHO Property**

project: **Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**

title: **Falling Head Test GW**

project no: **754-NTLGE368007** **Test 1**

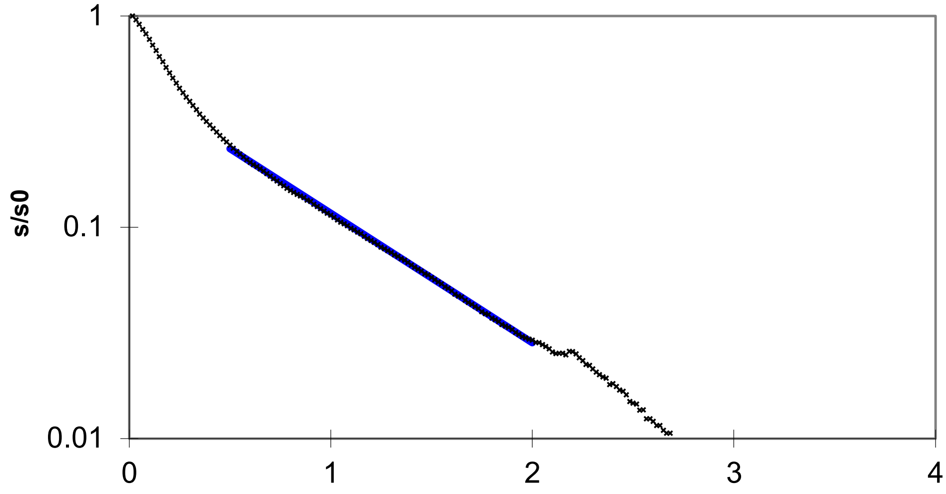
RISING OR FALLING HEAD TEST ANALYSIS

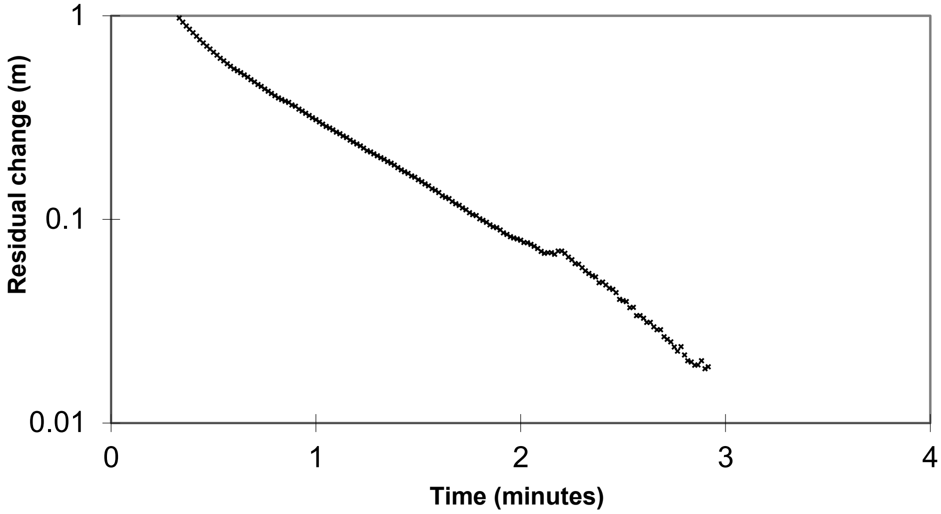
Bore Data		Units	Value
Initial groundwater level		m	11.48
Groundwater level at t=0		m	8.771
Casing radius	(r)	m	0.03
Bore radius	(R)	m	0.049
Screened interval length	(L	m	3
Match time start		min	0.5
Match time end		min	2
Characteristic Time (t_0)		**min**	**0.72**
Hydraulic Conductivity (K)		**m/day**	**1.24**
Hydraulic Conductivity (K)		**m/sec**	**1E-05**

Piezometer: GW

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

drawn	KF	client:	COHO Property
approved	SB	project:	Development for 36 Stockton St and 8A Tomaree St, Nelson Bay
date	3 Oct 2024	title:	Falling Head Test GW
scale	AS SHOWN	project no:	754-NTLGE368007 Test 2
original size	A4		

TETRA TECH

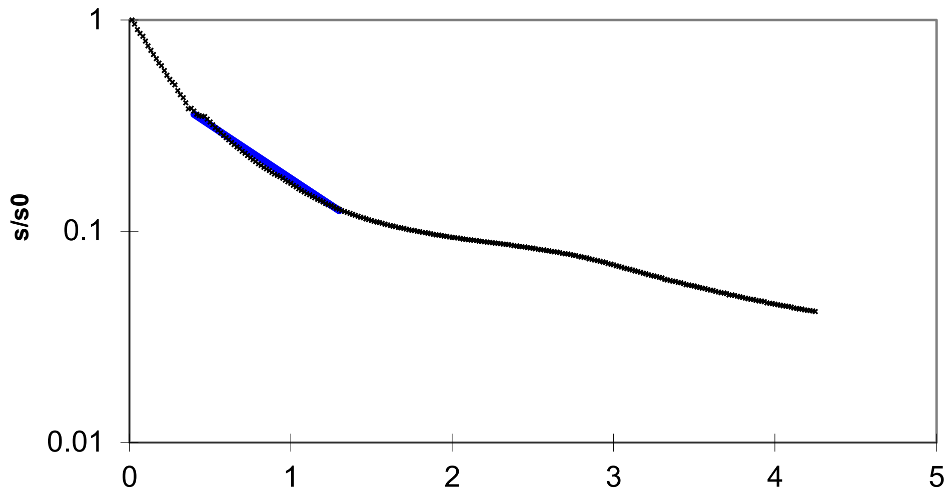
RISING OR FALLING HEAD TEST ANALYSIS

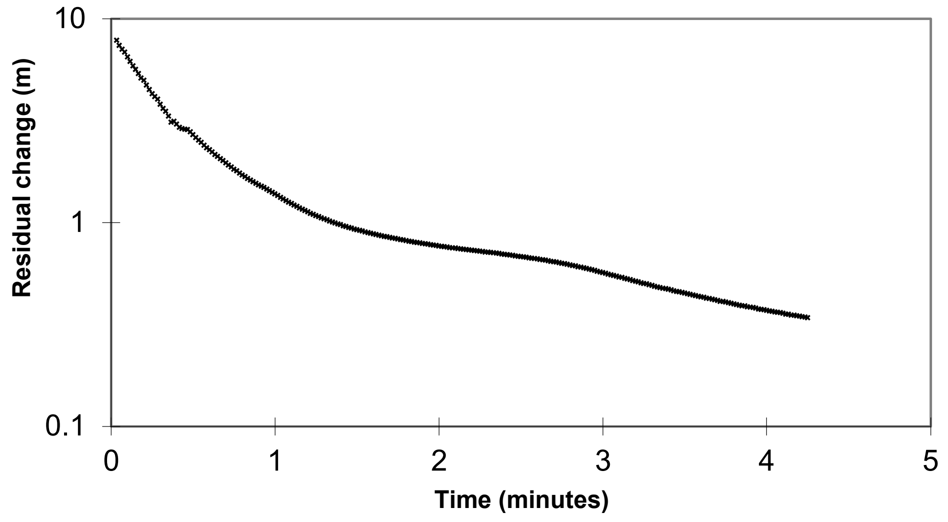
Bore Data	**Units**	**Value**
Initial groundwater level	m	11.2
Groundwater level at t=0	m	2.988
Casing radius (r)	m	0.03
Bore radius (R)	m	0.049
Screened interval length (L	m	3
Match time start	min	0.4
Match time end	min	1.3
Characteristic Time (t_0)	**min**	**0.83**
Hydraulic Conductivity (K)	**m/day**	**1.07**
Hydraulic Conductivity (K)	**m/sec**	**1E-05**

Piezometer: GW

Method Developed by Hvorslev (1951)

$$K = \frac{r^2 \ln(L/R)}{2Lt_0}$$





Reference: Hvorslev, M.J. (1951), Time lag and soil permeability in ground water observations. U.S. Army Corps of Engineers Waterway Experimentation Station, Bulletin 36.

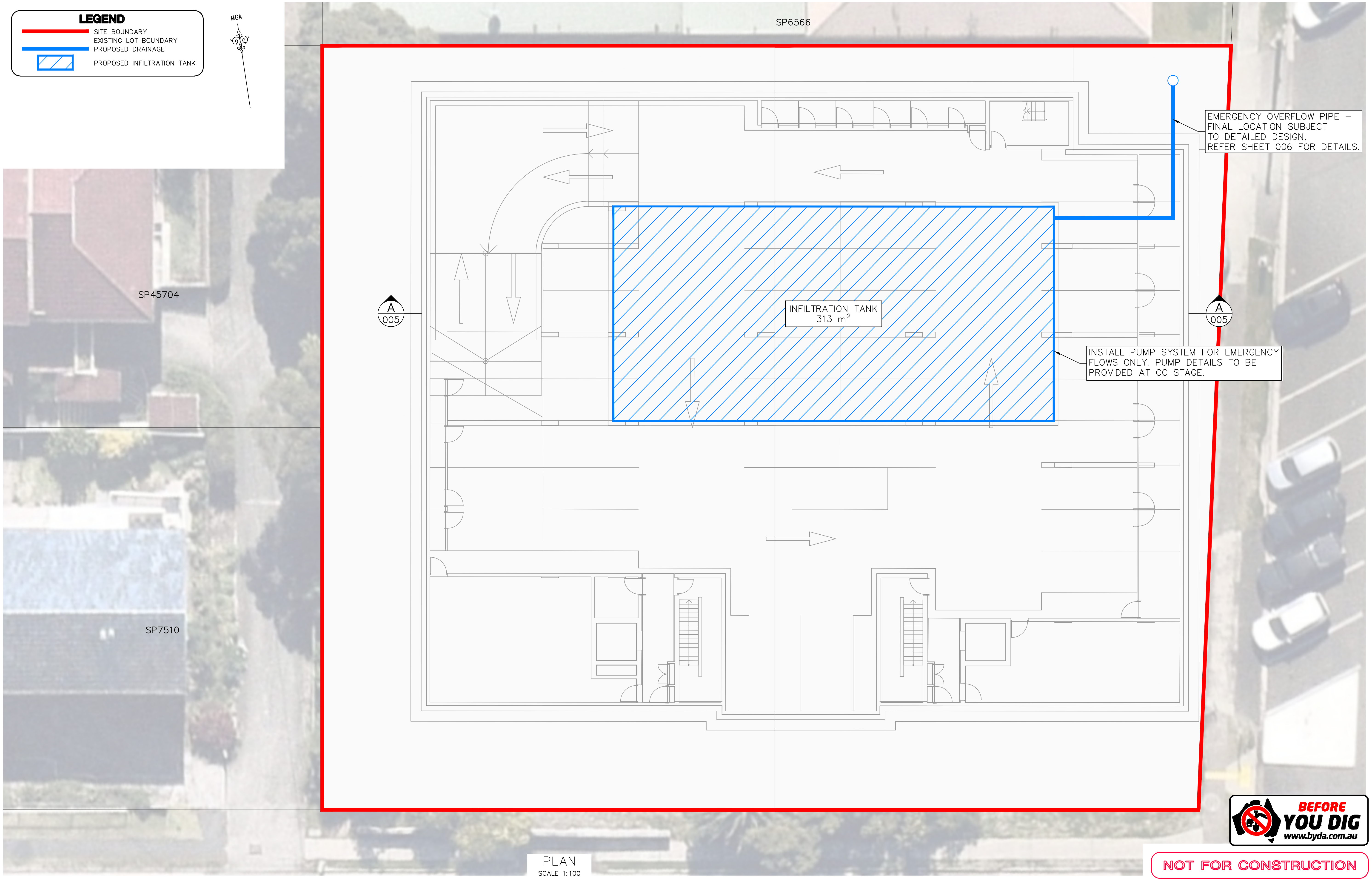
drawn	**KF**	client:	**COHO Property**
approved	**SB**	project:	**Development for 36 Stockton St and 8A Tomaree St, Nelson Bay**
date	**3 Oct 2024**	title:	**Falling Head Test GW**
scale	**AS SHOWN**	project no:	**754-NTLGE368007** **Test 3**
original size	**A4**		

TETRA TECH

Appendix C

ENGINEERING SKETCHES

190996-ESK-001 TO 005[E]



EMERGENCY OVERFLOW PIPE – FINAL LOCATION SUBJECT TO DETAILED DESIGN. REFER SHEET 006 FOR DETAILS.

INSTALL PUMP SYSTEM FOR EMERGENCY FLOWS ONLY. PUMP DETAILS TO BE PROVIDED AT CC STAGE.

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PLAN
SCALE 1:100

REV.	DATE	AMENDMENT	DRAWN	CHECK	DESIGN	VERIFY
A	22.10.2024	INITIAL ISSUE	R.C.	B.M.	C.C.	B.M.
B	22.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
C	23.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
D	07.02.2025	REVISED ISSUE	L.S.	O.M.	C.C.	B.M.
E	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

DESIGN FILE N:\JOBNUMBER\Design\12D\

SCALES
0 1 2 3 4
A1 / A3
1:100 / 1:200

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Hunter Office
Unit 7/335 Hillsborough Rd
Warners Bay N.S.W. 2282
Phone: (02) 4978 5100
Fax: (02) 4978 5199
email: hunter@adwjohnson.com.au
www.adwjohnson.com.au
ABN 62 129 445 398

CLIENT
COHO PROPERTY

PROPERTY DESCRIPTION
LOTS 781 & 782 DP.802108
CORNER OF STOCKTON AND
TOMAREE STREETS,
NELSON BAY, NSW 2315

SURVEYED: ADW Johnson
DATUM: GDA2020 M.G.A. ZONE 56 A.H.D.

PROJECT
TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE
STORMWATER MANAGEMENT
BASEMENT LEVEL

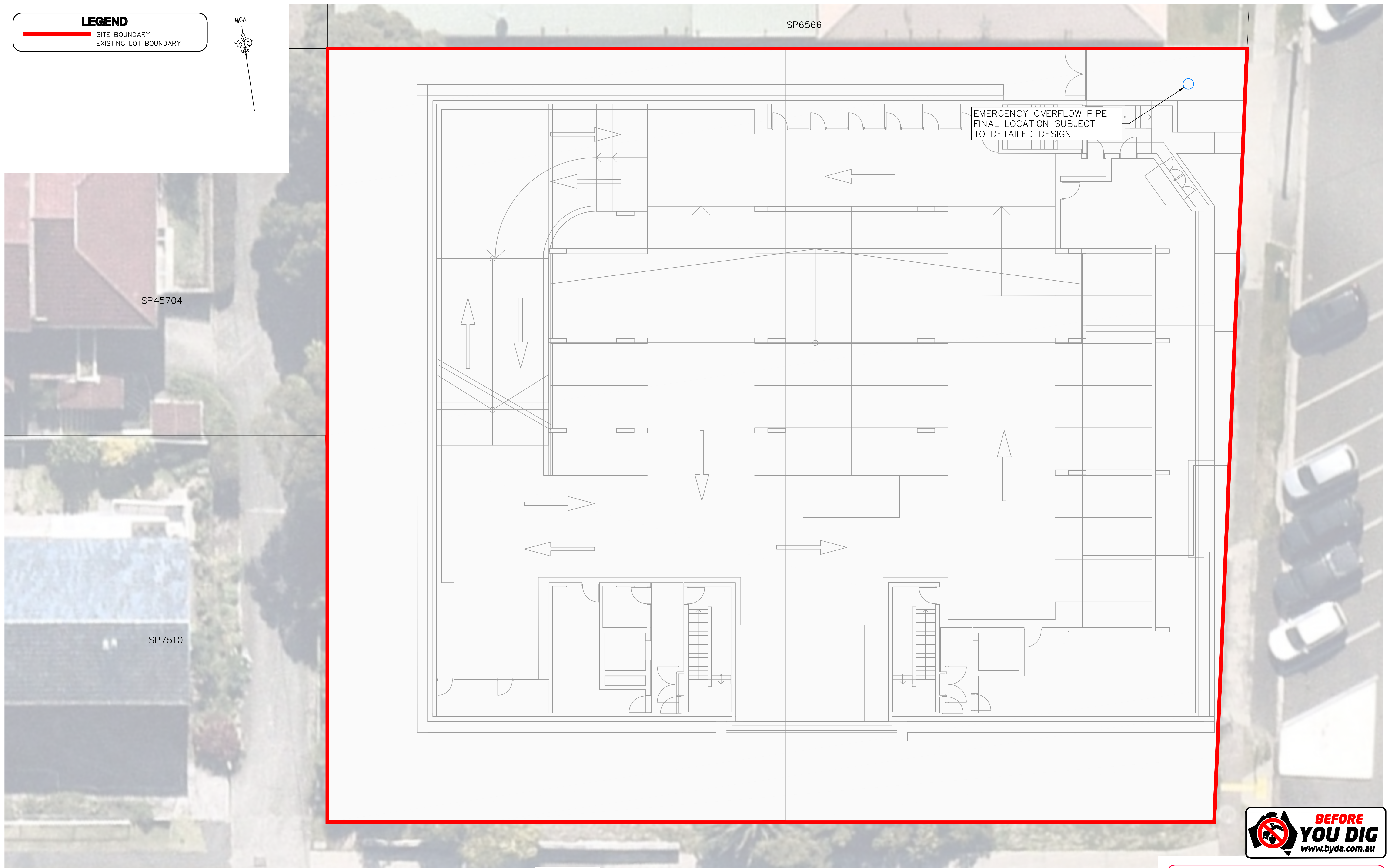
PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	001	E

This plan includes coloured information. If you have a black and white copy you do not have all of the information. This note is coloured RED.

100mm AT FULL SIZE

Plotted By: Dipendra Neupane Plot Date: 21/03/25 2:06:06PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-001 TO 005[E].DWG

190996-ESK-001 TO 005[E]



PLAN
SCALE 1:100

NOT FOR CONSTRUCTION

100mm AT FULL SIZE

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REV.	DATE	AMENDMENT	DRAWN	CHECK	DESIGN	VERIFY
A	22.10.2024	INITIAL ISSUE	R.C.	B.M.	C.C.	B.M.
B	22.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
C	23.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
D	07.02.2025	REVISED ISSUE	L.S.	O.M.	C.C.	B.M.
E	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

DESIGN FILE N:\JOBNUMBER\Design\12D\

SCALES
0 1 2 3 4
A1 / A3
1:100 / 1:200

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Hunter Office
Unit 7/335 Hillsborough Rd
Warners Bay N.S.W. 2282
Phone: (02) 4978 5100
Fax: (02) 4978 5199
email: hunter@adwjohnson.com.au
www.adwjohnson.com.au
ABN 62 129 445 398

CLIENT

PROPERTY DESCRIPTION
LOTS 781 & 782 DP.802108
CORNER OF STOCKTON AND
TOMAREE STREETS,
NELSON BAY, NSW 2315

SURVEYED
ADW Johnson

DATUM
GDA2020 M.G.A. ZONE 56 A.H.D.

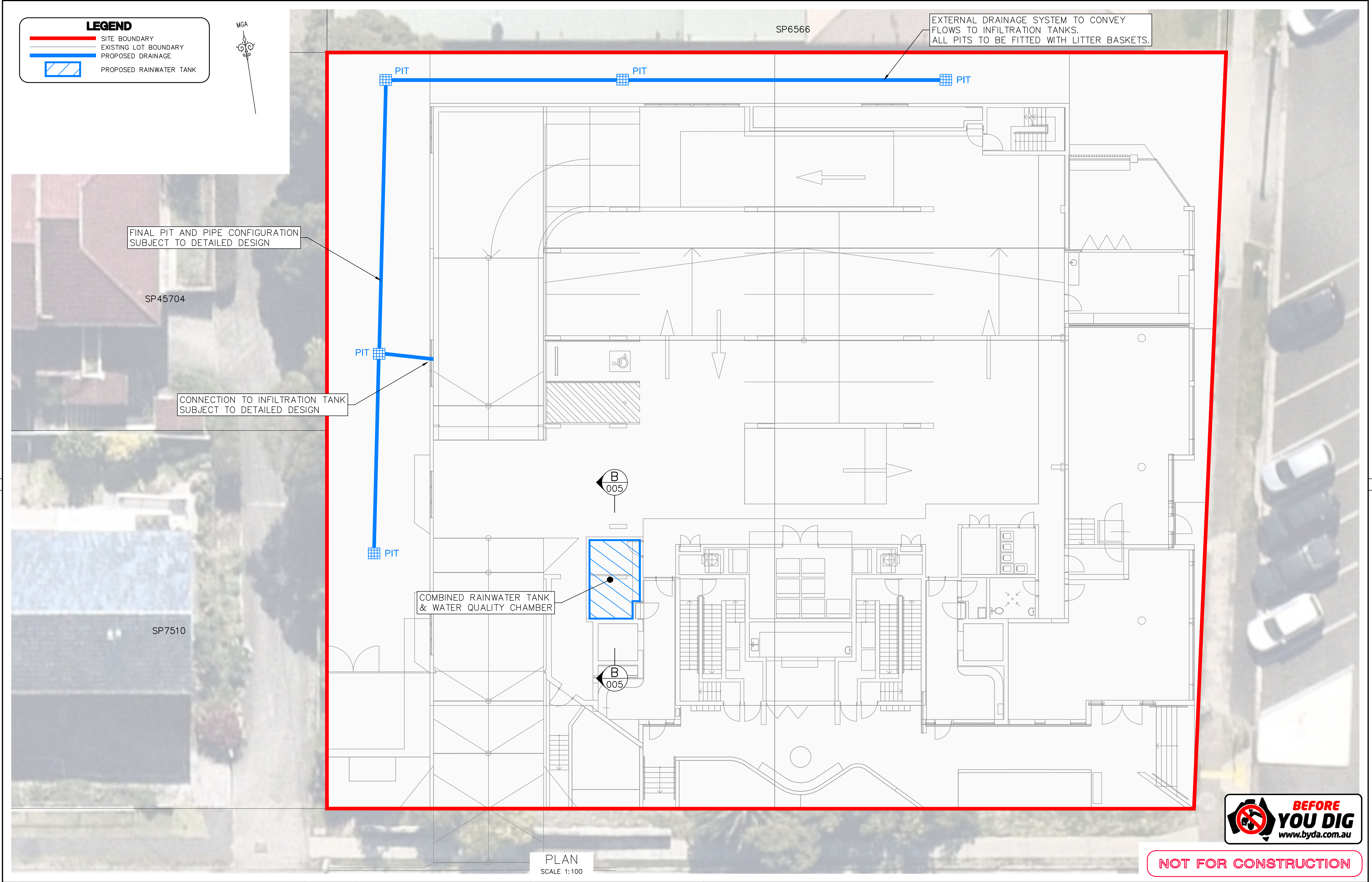
PROJECT
TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE
STORMWATER MANAGEMENT
LOWER GROUND LEVEL

PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	002	E

Plotted By: Dipendra Neupane Plot Date: 21/03/25 2:06:08PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-001 TO 005[E].DWG

190996-ESK-001 TO 005[E]



PLAN
SCALE 1:100

NOT FOR CONSTRUCTION

REV.	DATE	AMENDMENT	DRAWN	CHECK	DESIGN	VERIFY
A	22.10.2024	INITIAL ISSUE	R.C.	B.M.	C.C.	B.M.
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C	23.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
D	07.02.2025	REVISED ISSUE	L.S.	O.M.	C.C.	B.M.
E	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

DESIGN FILE N:\JOBNUMBER\Design\12D\

SCALES
0 1 2 3 4
A1 / A3
1:100 / 1:200

ALL DIMENSIONS ARE IN METRES U.N.O. DO NOT SCALE

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Hunter Office
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CLIENT
COHO PROPERTY

PROPERTY DESCRIPTION
LOTS 781 & 782 DP.802108
CORNER OF STOCKTON AND
TOMAREE STREETS,
NELSON BAY, NSW 2315

SURVEYED
ADW Johnson

DATUM
GDA2020 M.G.A. ZONE 56 A.H.D.

PROJECT
TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE
STORMWATER MANAGEMENT
GROUND LEVEL

PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	003	E

100mm AT FULL SIZE

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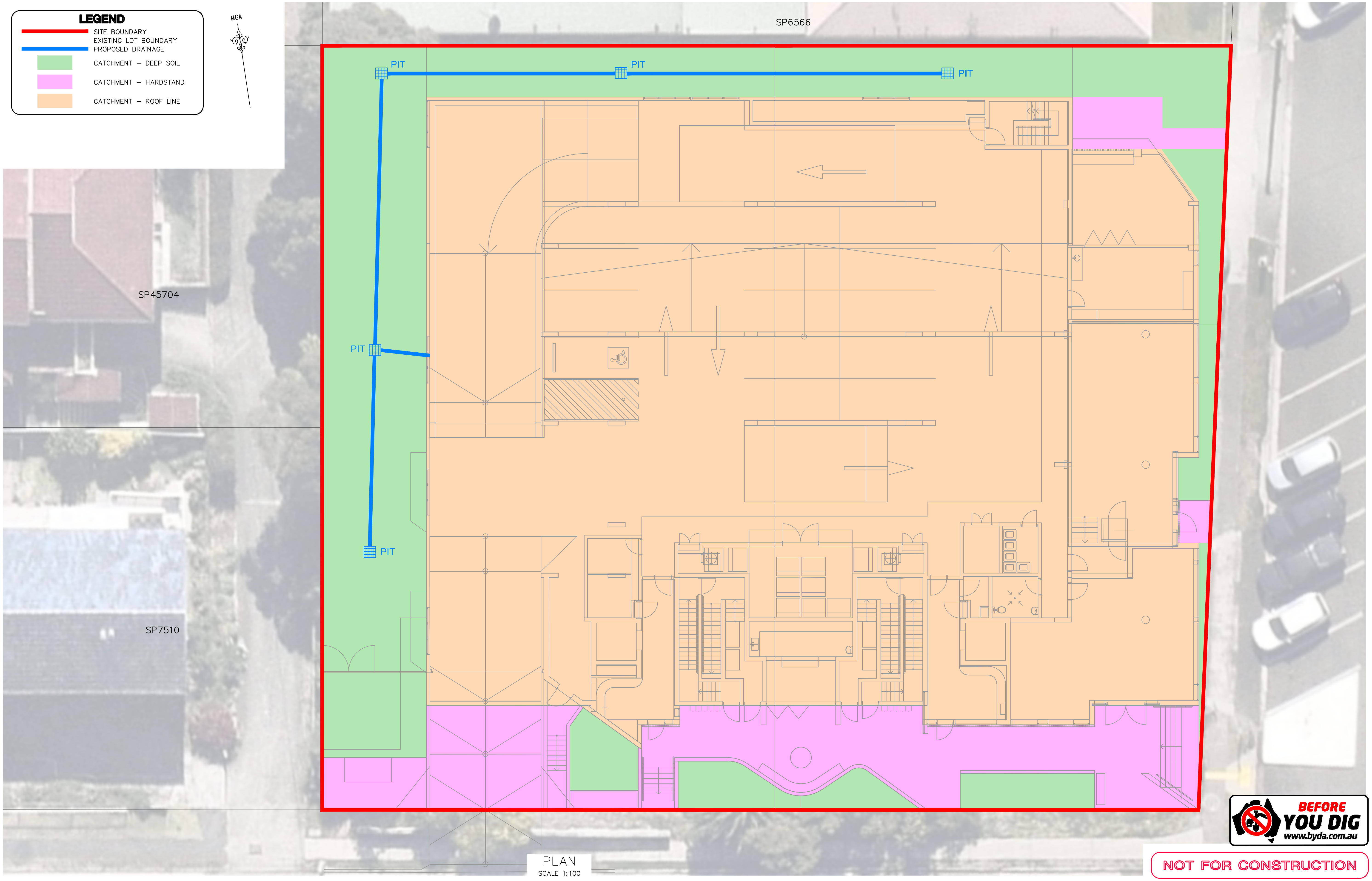
Plotted By: Dipendra Neupane Plot Date: 21/03/25 2:06:10PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-001 TO 005[E].DWG

190996-ESK-001 TO 005[E]



PLAN
SCALE 1:100

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100mm AT FULL SIZE

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C	23.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
D	07.02.2025	REVISED ISSUE	L.S.	O.M.	C.C.	B.M.
E	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

DESIGN FILE N:\JOBNUMBER\Design\12D\

SCALES
0 1 2 3 4
A1 / A3
1:100 / 1:200

ALL DIMENSIONS ARE IN METRES U.N.O. DO NOT SCALE

adw johnson

Hunter Office
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Phone: (02) 4978 5100
Fax: (02) 4978 5199
email: hunter@adwjohnson.com.au
www.adwjohnson.com.au
ABN 62 129 445 398

CLIENT
COHO PROPERTY

PROPERTY DESCRIPTION
LOTS 781 & 782 DP.802108
CORNER OF STOCKTON AND
TOMAREE STREETS,
NELSON BAY, NSW 2315

SURVEYED: ADW Johnson
DATUM: GDA2020 M.G.A. ZONE 56 A.H.D.

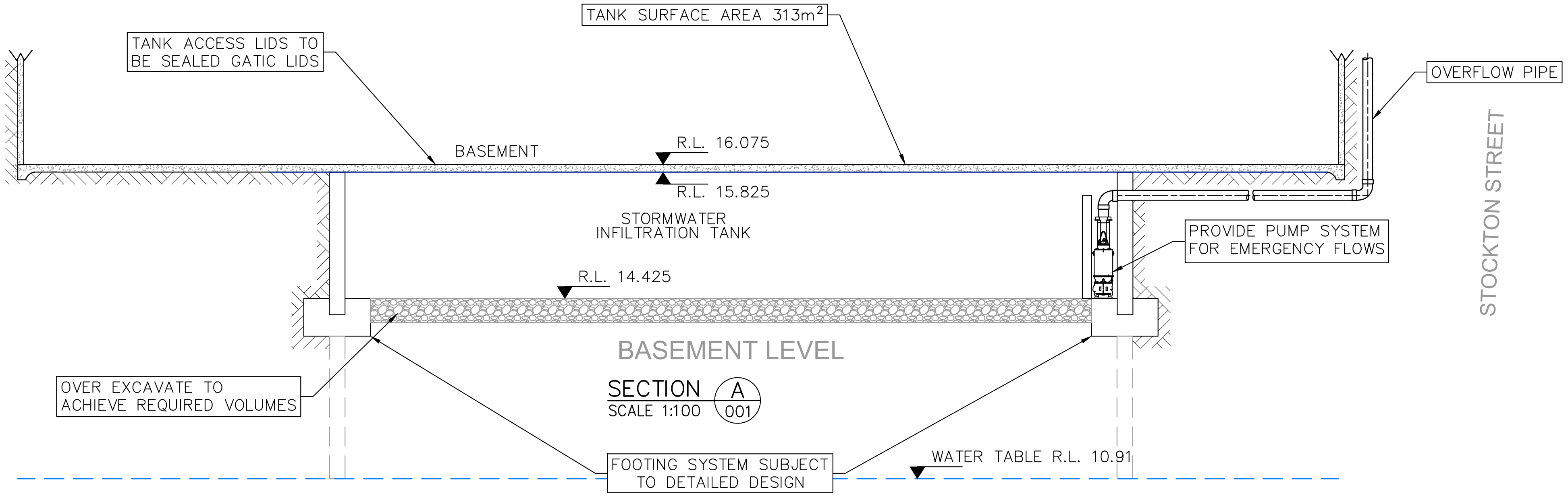
PROJECT
TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE
STORMWATER MANAGEMENT
GROUND LEVEL CATCHMENT AREAS

PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	004	E

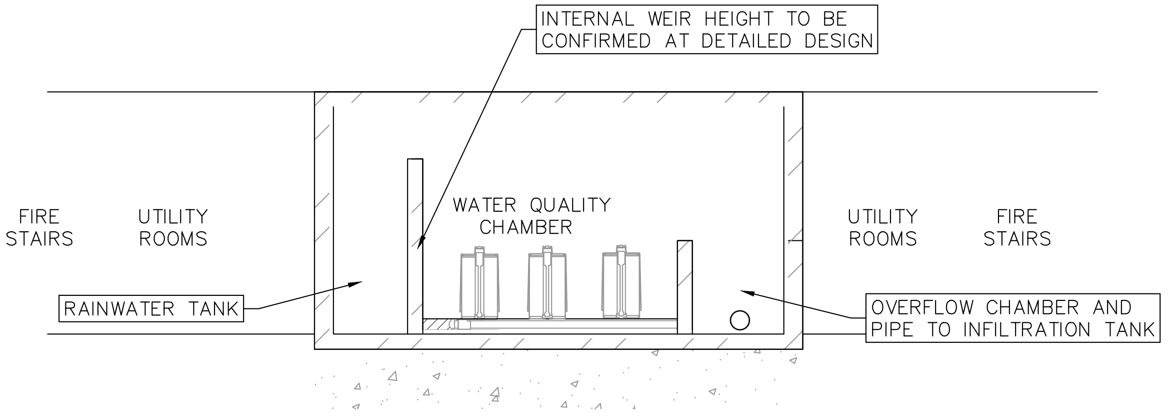
Plotted By: Dipendra Neupane Plot Date: 21/03/25 2:06:11PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-001 TO 005[E].DWG

190996-ESK-001 TO 005[E]



BASEMENT LEVEL

SECTION A
SCALE 1:100 001



STRUCTURAL DETAILS, INCLUDING FOOTINGS TO BE PROVIEDED AT DETAILED DESIGN PHASE

GROUND LEVEL

SECTION B
SCALE 1:50 003

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100mm AT FULL SIZE

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REV.	DATE	AMENDMENT	DRAWN	CHECK	DESIGN	VERIFY
A	22.10.2024	INITIAL ISSUE	R.C.	B.M.	C.C.	B.M.
B	22.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
C	23.10.2024	LAYOUT AMENDED	R.C.	B.M.	C.C.	B.M.
D	07.02.2025	REVISED ISSUE	L.S.	O.M.	C.C.	B.M.
E	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

DESIGN FILE N:\JOBNUMBER\Design\12D\

SCALES
0 1 2 3 4
A1 / A3
1:100 / 1:200

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CORNER OF STOCKTON AND
TOMAREE STREETS,
NELSON BAY, NSW 2315

SURVEYED
ADW Johnson

DATUM
GDA2020 M.G.A. ZONE 56 A.H.D.

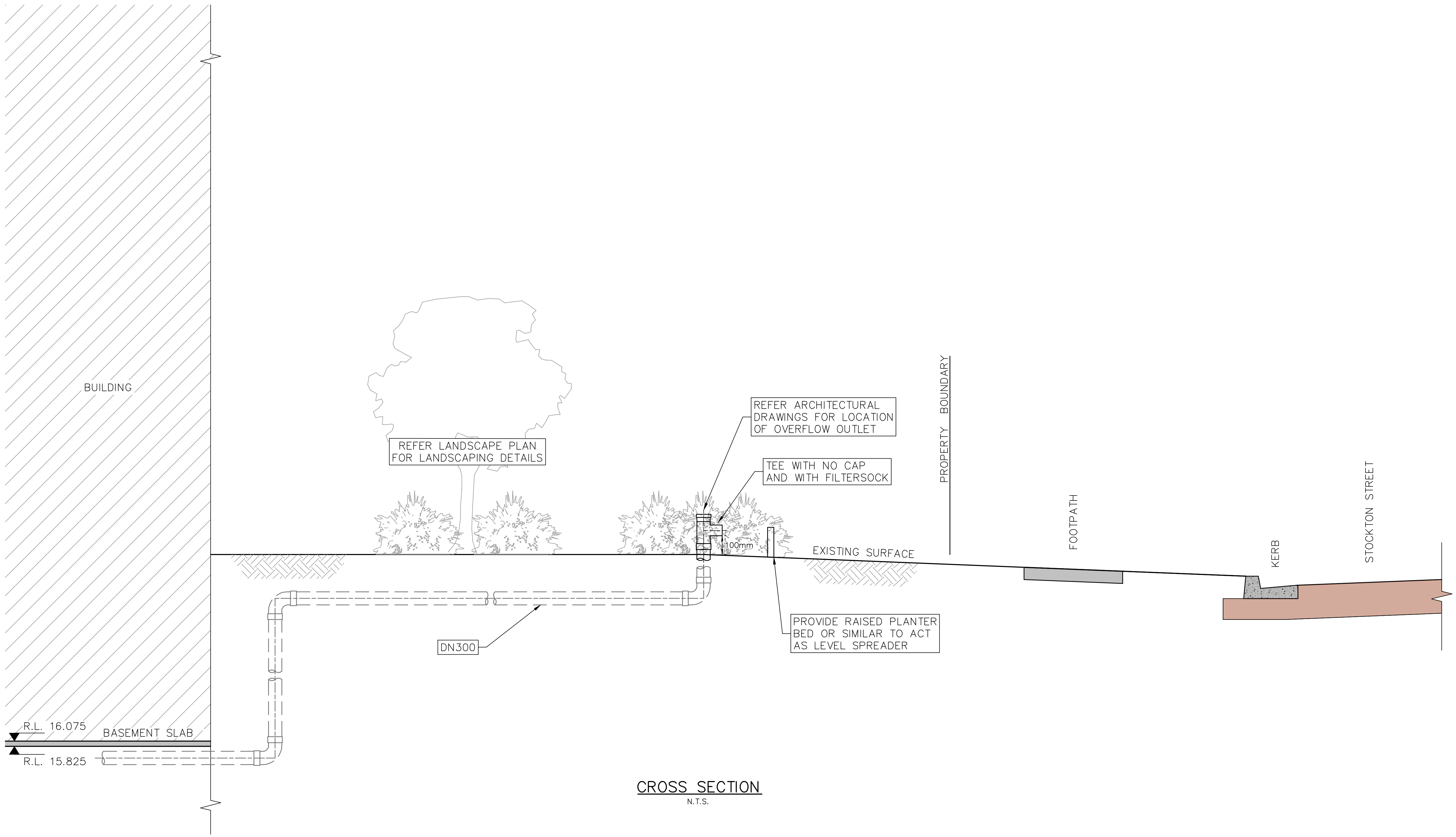
PROJECT
TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE
STORMWATER MANAGEMENT
TYPICAL SECTIONS

PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	005	E

Plotted By: Dipendra Neupane Plot Date: 21/03/25 2:06:12PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-001 TO 005[E].DWG

190996-ESK-006[B]



CROSS SECTION

N.T.S.

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REV.	DATE	AMENDMENT	DRAWN	CHECK	DESIGN	VERIFY
A	07.02.2025	INITIAL ISSUE	L.S.	O.M.	C.C.	B.M.
B	21.03.2025	REVISED ISSUE	D.N.	B.M.	C.C.	B.M.

SCALES

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PROJECT

TOMAREE STREET RESIDENTIAL APARTMENTS

PLAN TITLE

PROPOSED EMERGENCY OVERFLOW OUTLET DETAIL

PROJECT No.		DISCIPLINE		NUMBER	REV.
190996	–	ESK	–	006	B

Plotted By: Dipendra Neupane Plot Date: 21/03/25 12:45:56PM Cad File: \\ADW-SVR1\DRAFTING\190996\DWG\ENGINEERING\ESK\190996-ESK-006[B].DWG