

Attachment A
Rail Bridge Crossing
Geotechnical Report

Geotechnical Investigation Report

Rail Bridge Crossing

Mountain Ridge Road, New Beith



Prepared for:

Frasers Property New Beith Pty Ltd

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Core Consultants Pty Ltd (Core) for a proposed rail bridge crossing which will provide access from Mountain Ridge Road to the new subdivision (Precinct A).

The work was carried out for Frasers Property New Beith Pty Ltd (Frasers), in accordance with our proposal Q003848-001-L-Rev1, dated 9 December 2022.

Presented in this report is a summary of the site conditions and method of investigation; along with comments and recommendations regarding site preparation, excavatability, trafficability, filling, batter slopes, erosion control, suitable footing types, foundation and retaining wall design parameters and reduction factors.

2.0 PROPOSED DEVELOPMENT

The proposed bridge will be a two-span structure with a central pier. At each end, the bridge will also be supported on concrete abutments together with reinforced earth (RE) walls to support the bridge approach embankments (ref. Image 1).

Structural working loads were not provided at the time of preparing this report, however, it is envisaged to be similar to heavy duty bridge type loads. Some minor cut earthworks (up to 0.5 m to 1 m deep) are anticipated to form level building pads for the bridge abutments and footing excavations.

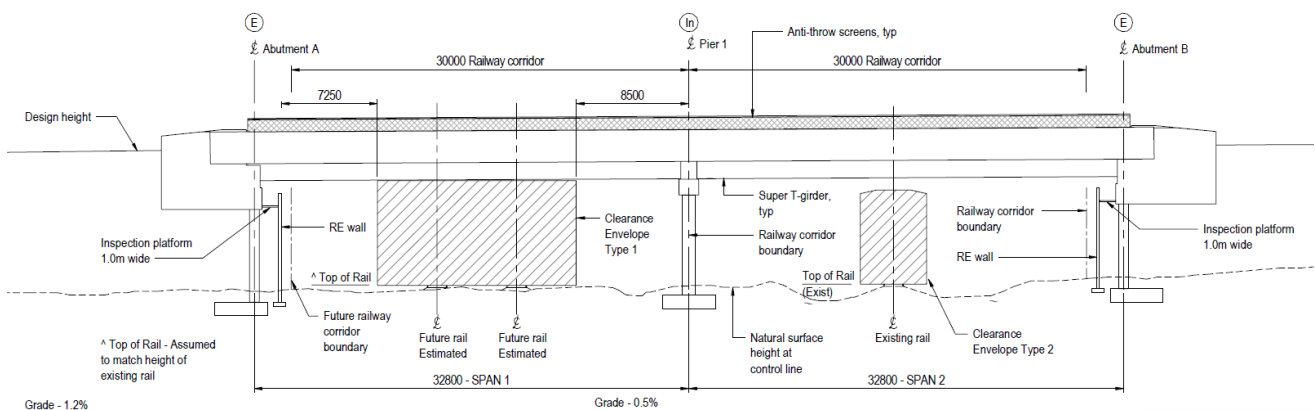


Image 1: Longitudinal elevation of proposed bridge structure (ref. Pitt & Sherry Dwg No. P.22.0175.00-STR-DRG-003, Rev A)

3.0 PREVIOUS INVESTIGATION BY OTHERS

In 2022, Frasers commissioned others to carry out geotechnical investigations for the proposed bridge (report ref. Douglas Partners (DP) Project 216621.00, dated October 2022).

The investigation comprised two boreholes (designated Bores 1 and 2) to depths of 15 m, together with laboratory testing and geotechnical advice and recommendations.

Factual and laboratory testing data from that report has been incorporated into this report.

4.0 SITE DESCRIPTION

The proposed bridge is located at the western end of Mountain Ridge Road. The bridge will span the existing ARTC rail corridor.

At the time of the investigation, an unsealed at-grade road provided access over the rail line approximately adjacent to and parallel to the location of the proposed bridge. The surrounding area generally comprised bushland including grasses, shrubs, and small to large mature trees.

General site conditions during our investigation are shown in Photograph 1.



Photograph 1: General site conditions (view looking northeast towards borehole BHB and rail line)

5.0 METHOD OF INVESTIGATION

5.1 Review of Available Information

To assess likely ground conditions a review of the published geological maps and available geotechnical information was undertaken (refer Section 6.1).

5.2 Fieldwork

The fieldwork was carried out between 9 and 11 January 2023.

Three boreholes (denoted BHA to BHC) were drilled at client nominated locations at the proposed abutment and central pier locations to depths of 15 m below ground level (bgl).

The boreholes were drilled using a track mounted Hydrapower Scout drill rig, utilising auger, wash boring and NMLC coring techniques.

Dynamic cone penetrometer (DCP) tests were undertaken from the ground surface to assess the shallow soils prior to commencing drilling.

Standard penetrometer testing (SPT) was also carried out within borehole BHC at 1.5 m bgl to assess the rock strength prior to commencing NMLC coring.

Disturbed soil samples and NMLC core was recovered from all boreholes for subsequent laboratory testing (refer Section 3.4).

The boreholes were backfilled with spoil on completion of drilling.

The boreholes were drilled under the direction of a geotechnical engineer from Core, who logged subsurface materials in accordance with Australian Standard AS 1726-2017 *Geotechnical site investigations*, directed the field testing and sampling, and made groundwater observations during drilling.

Borehole coordinates were obtained using a hand-held GPS unit, and locations are shown on the attached Figure 1 in Appendix A, together with previous boreholes by DP.

Reports of Boreholes are presented in Appendix B, together with explanatory notes. Previous borehole reports prepared by DP are also attached. Subsurface conditions are discussed in Section 4.3.

5.3 Geotechnical Laboratory Testing

Laboratory testing on selected soil and rock samples was carried out at NATA accredited laboratories in accordance with Australian Standard test methods (AS1289 and AS4133) and comprised the following:

- Particle size distribution
- Soil Aggressivity
- Point load index on NMLC core
- Unconfined compressive strength (UCS) on NMLC core

Laboratory test results are summarised in Section 6.2.

6.0 RESULTS OF INVESTIGATION

6.1 Published Geological Information

Published information¹ indicates that the bridge site is underlain by Early Jurassic Age Gatton Sandstone comprising '*Lithic labile and feldspathic labile sandstone*'. The subsurface conditions encountered in the boreholes (described below) generally confirm the published geology.

6.2 Laboratory Testing

Laboratory testing on selected soil and rock samples were carried out at NATA-accredited laboratories in accordance with Australian Standard test methods (AS1289 and AS4133). The test results were used to confirm field classifications of soil and strength of rock. Laboratory test results are summarised in Tables 1 to 3, together with the results of the previous testing by DP.

Results of the point load index testing are shown on the Reports of Boreholes in Appendix B. Laboratory test certificates are attached in Appendix C.

Table 1: Summary of laboratory test results.

BH ID	Sample Depth (m BGL)	Material Type	Moisture Content (%)	Particle Size Distribution			Atterberg Limits and Linear Shrinkage		
				% Fines	% Sand	% Gravel	Liquid Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
BHB	0.9 – 1.1	Sandy Clay	-	45	-	-	40	22	10.0
Bore 2	1.0 – 1.3	Sandy Clay	18.0	-	-	-	40	22	11.5

Table 2: Summary of soil aggressiveness test results.

BH ID	Sample Depth (m BGL)	pH	Sulfate (SO ₄), mg/kg	Chloride, mg/kg	Resistivity, ohm.m	Conductivity, µS/cm	Material Type
BHC	1.5 – 1.74	7.8	98	300	56	180	Sandstone
Bore 2	0.5 – 1.0	5.1	30	20	-	-	Clayey Sand / Sandy Clay

Results of soil aggressiveness testing are discussed in Section 5.8.

Table 3: Summary of Unconfined Compressive Strength test results.

BH ID	Sample Depth (m BGL)	Material Type	Unconfined Compressive Strength (MPa)
BHC	7.00 – 7.25	Sandstone	19.9
Bore 1	5.12 – 5.345	Sandstone	27.0
Bore 2	13.09 – 13.29	Sandstone	44.3

¹ Queensland Geotechnical Database Website

6.3 Subsurface Conditions

The subsurface conditions encountered in the boreholes generally comprised a thin layer of silty sand topsoil or silty sandy gravel fill to depths of between 0.1 m and 0.2 m bgl. These materials were underlain by residual soils comprising a layer of loose to very dense silty sand overlying stiff to hard sandy clay to depths of between 0.8 m and 1.2 m bgl.

Extremely to highly weathered, very low to low strength sandstone was encountered below the residual soils, becoming moderately to slightly weathered, medium to high strength from 1 m to 3.5 m bgl, and then becoming high to very high strength from about 14.5 m bgl.

Groundwater was not encountered in the boreholes during auger drilling to depths of between 1 m and 1.74 m bgl. Below such depths, groundwater could not be observed due to the introduction of drilling fluid.

7.0 GEOTECHNICAL COMMENTS & RECOMMENDATIONS

Based on the subsurface profiles encountered in the boreholes, bridge foundation loads should be transferred to the underlying weathered rock. Depending on lateral loads on the bridge structure, high level spread footings could be considered. If greater support and capacity is required (e.g. for train impact loads), hold down anchors or piled foundations may need to be adopted.

7.1 High Level Footings

7.1.1 Bearing Pressures and Settlements

Conventional high-level strip or pad footings founded in the very low strength (or stronger) sandstone could be adopted using a maximum ultimate (unfactored) bearing pressure of 1.5 MPa. This value assumes that suitable foundation preparation has been carried out and that the ground strength profile is uniform through each layer (with no underlying lower strength material). Based on the borehole findings at each pier/abutment, we consider this to be a relatively low risk.

Ultimate geotechnical capacities ($R_{d,ug}$) determined from the parameter provided must be multiplied by a geotechnical strength reduction factor (ϕ_g) in accordance with Section 4 of AS2159-2009 to determine the design geotechnical strength ($R_{d,g}$). The geotechnical strength reduction factor should be determined in accordance with AS2159 and is expected to be in the range of 0.45 to 0.6. An upper bound value may be adopted if supervision and certification of the foundation construction is carried out by a geotechnical engineer

At the pressure nominated above, footing settlements would not be expected to exceed around 1% of footing width for strips and pads, under serviceability loads. Differential settlements between adjacent footings founded in the same materials would not be expected to exceed approximately 50% of the total settlement for individual footings. Elastic modulus values are provided in Table 4 below to assess settlements.

A geotechnical engineer must inspect the footing excavations to confirm the design bearing pressures during construction.

Where greater capacity or lesser settlements are required, piled foundations will be required as discussed below.

7.1.2 Hold Down Anchors

Where greater resistance to lateral movements is required to support abutment loads, hold down anchors could be adopted.

The use of high-capacity grout injected micropiles tied into the high-level foundations could be considered. The rock drilling technology used in the construction of micropiles enables the micropiles to be drilled and bonded within the underlying rock without the need for additional specialist equipment or drilling techniques.

Tungsten carbide drill bits are generally adopted to enable the micropiles to be drilled through high strength rock and other obstructions (such as cobbles, if encountered) to achieve the required rock socket length.

The piles are installed with a relatively small drill rig (or excavator) reducing the requirement for a specialist piling platform to support the rig. Photographs 2 and 3 below displays the use of micropiles for a bridge abutment.



Photographs 2 and 3: Micropile installation equipment and view of micropiles installed in a bridge abutment

Pile design parameters are provided in Table 4 below.

7.2 Reinforced Earth (RE) Walls

Footings for the proposed RE walls could be designed using the parameters provided in Section 7.1.

Global stability analyses of the RE walls will also need to be carried out during the design stage which could influence the depth(s) of the wall footings. Such analysis could be carried out using the parameters provided in Table 4 below.

7.3 Piled Foundations

Piles should be extended to found with adequate socket penetration into the very low to low strength (or stronger) sandstone, or the underlying medium to high strength (or stronger) sandstone if greater axial capacity is required. Where high strength rock is encountered, coring will likely be required to advance the pile excavations using heavy-duty drilling plant.

The following pile types could be considered:

- Bored piles
- Grout-injected micro-piles

Bored piles may be advanced using bucket and flight auger techniques through soils and very low to low strength rock until higher strength rock is encountered and coring might be required (but depending on machine size and drilling bucket capability) to advance the pile excavations. Bored piles should extend to found with adequate socket penetration into the medium to high strength (or stronger) rock, and allowance should therefore be made for the use of heavy-duty drilling plant.

It is recommended that an experienced piling contractor be consulted to confirm the most suitable pile type and design parameters.

Table 4 presents recommended ultimate soil parameters that may be used for the design of concrete piles.

Table 4: Design values for foundation design

Material type	Unit weight γ (kN/m ³)	ϕ (degrees)	Effective Cohesion, c' (kPa)	C_u (kPa)	Elastic Modulus (MPa)	Ultimate skin friction (kPa)	Ultimate base resistance (kPa)
Stiff Clay	18	25	3	50	8	10	Not Suitable
Very Stiff Clay	19	30	5	100	15	30	
Hard Clay	20			200	40	50	
Dense Sand	19	32	-	-	50	50	
Very Dense Sand	20	34	-	-	80	80	
Very Low to Low Strength Sandstone	21	40	-	-	150	150	1,500
Medium to High Strength Sandstone	24	45	-	-	500	600	10,000

Notes:

- 1) N/R denotes not recommended
- 2) Skin friction should be discounted for sections of pile shafts where liners are in place.
- 3) Parameters for sandstone only applicable where piles socket 1.5 times pile diameter into this material and no weaker foundation material exists within four pile diameters below the base of the pile footing.
- 4) Piles socketing into medium (or higher) strength rock will require the use of heavy-duty drilling plant.

The $F_{m,s}$ and F_b values given in Table 4 are defined in AS 2159-2009 "Piling – Design and installation", with $F_{m,s}$ being the average skin friction for the condition of full mobilisation for a compression pile, and F_b being the ultimate base pressure for a compression pile. For Limit State design it is suggested that ultimate values be multiplied by a Geotechnical Strength Reduction Factor (ϕ_{gb}) calculated in accordance with Section 4 of AS 2159-2009 to obtain ultimate limit state "design" values.

Our assessment of the basic geotechnical strength reduction factor (ϕ_{gb}) as per AS 2159-2009 is based on an overall design average risk rating (ARR) value of 2.2. It should be noted that this ARR value is based on the assumption of an individual risk rating (IRR) of 1 for *level of construction control during pile installation*, and IRR of 3 for *level of performance monitoring*, as set out in Table 4.3.2(A) of AS 2159-2009 (i.e. "detailed with professional geotechnical supervision, construction processes that are well established and relatively straightforward, and correlation of installed parameters with on-site static load tests carried out in accordance with this standard").

For an ARR value of 2.2, Table 4.3.2(C) of AS 2159-2009 indicates:

- An overall risk category of 'Low'
- $\phi_{gb} = 0.56$ (for low redundancy systems).

The design parameters set out in Table 4, together with the ARR value derived above are also based on the assumption of good quality pile construction practice, which assumes the following measures are adopted:

- Installation of all piles to be supervised by a suitably qualified geotechnical engineer.
- Suitably qualified geotechnical engineer to log drilling returns during rock socket installation.
- Once the pile has been advanced to founding depth the base will need to be effectively cleaned to remove debris on the base of the excavation. Use of a cleaning bucket with a similar base profile to the drilling bucket used to excavate the socket is recommended. This will reduce the risk of settlement associated with the presence of softer materials at the pile base at the time of pouring.
- The use of appropriate tools to remove smear from the pile shaft and to roughen the pile socket to achieve the design shaft resistance value. After drilling the socket to the target depth, we recommend attaching one or more offset reamer teeth to the drilling bucket and then thoroughly reaming out the pile shaft prior to the final clean out of the base using a purpose-built cleaning bucket.
- Constructing the pile so as to limit water ingress to the pile excavation and thereby reduce the potential for slaking to occur. Core recommends initially drilling the pile in the dry and observing the progress of the pile excavation. If the pile excavation fills with water we recommend that a submersible pump be

used to extract this water from the pile socket prior to pouring the concrete. We also recommend that the concrete be placed using a tremie pipe to place the concrete from the pile base up. This procedure uses the concrete to displace water remaining on the pile base allowing concrete to be placed directly onto the base of the pile.

- The use of temporary steel liners to stabilise the upper soils may be required. The liner could be removed after casting of the pile whilst the concrete is still fluid. Drilling into rock and advancing steel liners would however require specialised drilling equipment.
- On site pile load testing of the bored pile construction procedure would be advisable to verify construction quality, following which the appropriate design values based on observations and on-site trial testing could be assigned. We recommend at least one test per bridge abutment.

Piles designed and constructed in accordance with the above recommendations should not encounter settlements greater than 0.5% to 1% of the pile diameter.

The lateral capacity of a pile is provided by the passive resistance of the surrounding soil. The following should be ignored in the assessment of lateral capacity:

- Upper-level soils to a depth 1.5 times the pile diameter below the surface; and
- The material above the potential scour zone.

The effect of group action should be considered where piles are to act in groups. An overall lateral capacity of less than the sum of individual lateral pile capacities could result where piles are closely spaced.

As part of the installation program for bored piles, a suitably qualified geotechnical engineer should be on site to observe construction of the bored piles and to confirm consistency between the founding materials and design assumptions. It is also recommended that the pile drilling, cleaning and pouring be undertaken in one sequential operation, and that the pile is poured within twenty-four hours of reaching target depth.

It is recommended that the preferred piling subcontractor prepares a work construction methodology prior to commencing work on site. This plan should be reviewed by a geotechnical engineer to check that the construction methodology aligns to the quality standards outlined in this document for pile construction.

A working platform assessment must also be carried out by a suitably qualified person (RPEQ) for heavy machinery or equipment (e.g., crane or piling rig).

7.4 General Comments on Footings

All proposed footings should found such that they are not adversely affected by any adjacent excavations, batter slopes, trenches, or retaining walls that are not designed to support structure/equipment loads. To minimise the potential for any adverse interaction effects, footings should found at least below a plane extending 1 m horizontally from the base of trenches/batter slopes/excavations/retaining walls, then rising up at 1V:1H, as illustrated in Diagram 1.

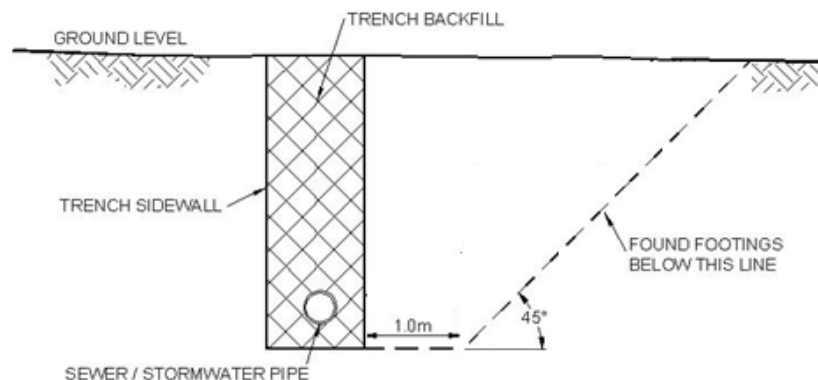


Diagram 1

Excavations adjacent to existing structures/equipment must be careful not to undermine existing footings.

Any high-level footing excavations should be cleaned following mechanical excavation to expose undisturbed materials over the full base area. It is recommended that footings be inspected by a suitably qualified and experienced geotechnical engineer prior to placement of blinding/mass concrete or steel reinforcement to confirm allowable bearing pressures and cleanliness of excavations.

If any soil conditions encountered during construction are found to differ from those noted in the geotechnical investigation, Core should be notified immediately, and an inspection carried out to determine if changes to footing design are required.

Adequate site drainage should be carried out to ensure that natural runoff is directed off construction area platforms and away from any footings. If water is allowed to pond on the platform, associated softening of the soil may occur, and the allowable bearing capacity is likely to be less than those values given above. Increased soil reactivity may also result.

7.5 Site Preparation and Trafficability

Site preparation in the proposed bridge abutment areas and approach embankments, should include stripping of topsoil and any loose/soft sediment, followed by 'proof rolling' of the stripped surface by several passes of a minimum 10 tonne roller to identify any further loose or soft areas.

A suitably qualified geotechnical engineer should be present during the subgrade 'proof roll'. Loose or soft areas identified by proof rolling may require treatment by either of the following methods:

- Over-excavation and replacement with suitable fill compacted in layers; or
- Moisture conditioning and re-compaction of the loose or soft material, provided that it does not contain substantial organic matter or deleterious substances.

Trafficability is anticipated to be poor over grassed and stripped areas following periods of rainfall. It is recommended that allowance be made for the placement of a 150 mm (minimum) layer of granular material across trafficked areas. Further discussion is provided below regarding working platforms.

7.6 Preliminary Working Platform Assessment

A working platform will be required to support piling rigs and cranes during construction. No rig specifications have been provided at the time of this report, however our experience on similar projects indicate that the rig could have a nominal bearing pressure of 300 kPa, a track width of 0.9 m and effective track length of 1.6 m. These specifications have been adopted for the purpose of this preliminary assessment.

The subsurface conditions used for the preliminary assessment were based on the borehole findings from this investigation.

Based on the proposed nominal bearing pressure of 300 kPa, piling from the surface would require a minimum 0.45 m thick working platform. Further geotechnical assessment of the natural subgrade (by DCP testing or Plate Load testing) is recommended to assess the bearing capacity of the natural subgrade across the extent of the proposed piling platform.

The working platform thickness could be reduced if rigid steel plates are used below the tracks or if geogrids are incorporated into the design.

The platform should comprise compacted gravel or crushed rock or metamorphic or igneous origin. Sedimentary rock is not considered suitable. Soft/loose or waterlogged material, including topsoil must be removed from the ground surface prior to constructing the platform. Monitoring of the ground surface must be carried out during trafficking and operations to observe signs of deflection or settlement. If deflections or settlement are observed, the rig must relocate to a stable location, and additional assessment must be carried out. All personnel on site must be made aware of this requirement.

7.7 Excavatability

Excavations within the overlying residual soils, extremely weathered material, and very low to low strength sandstone should be achievable using conventional medium earthmoving equipment (i.e., 20 to 30 tonne excavators). In confined excavations such as trenching, a ripping tyne or hydraulic rock breakers may be required. Comments on excavatability for bored pile construction are provided earlier in this report.

7.8 Batter Slopes

For short-term construction periods, excavated unsurcharged faces up to 3 m in vertical height could be battered at 1V:1H or benched to an equivalent slope using benches not more than 1.5 m high in the residual soils and extremely weathered rock.

Near vertical excavations in the medium to high strength sandstone up to 3 m high should be stable in the short term subject to absence of adverse conditions (e.g. a combination of subvertical joints that could result in toppling wedges, or groundwater) and provided there are no surcharges on the crest areas. Specific assessment of slopes by an experienced geotechnical engineer would be required at time of construction.

Flatter batters or temporary support systems (e.g. 'shields' or 'shoring boxes' for trenches) may be required if significant groundwater seepage is encountered or if exposed faces are not protected from erosion by rainfall. Flatter batters may also be required where fissuring is observed within the clay soils. Shoring would be required to enable safe personnel entry into trenches (with vertical sides) deeper than 1.2 m.

7.9 Material Suitability for Re-use

Based on the subsurface conditions encountered, soil won from cut excavations is anticipated to generally be suitable for reuse as structural/embankment fill (e.g. such as Class A or B earth fill in zoned embankments as per MRTS04), provided it is free from deleterious material, oversize particles and organic matter. The site excavated material is unlikely to be suitable as select backfill for the reinforced earth and imported fill will be required.

Where clayey soils are proposed to be re-used in fill embankments, it is recommended that cohesive materials be placed at depth and granular materials be placed in the upper parts of the profile. This will reduce the effects of seasonal moisture change and foundation soil reactivity and will improve pavement subgrade conditions and surface trafficability during construction. Re-use of sands will require blending with clay to improve workability.

Any cohesive materials will require moisture conditioning as indicated in the following section of this report for compaction. Difficulties in achieving the required compaction density ratios and adverse long-term performance of the fill should be expected if adequate moisture conditioning is not undertaken.

7.10 Filling

Where filling is to be undertaken, the following recommendations should be adopted:

- Fill should be compacted at moisture contents within the range of -2% to +2% of optimum moisture content (OMC) for Standard Compaction. Adverse long-term performance of the fill and difficulties with handling, placement, and compaction may be encountered if the material is not at the approximate OMC for compaction, particularly if the materials are overly moist.
- Field density testing should be carried out to confirm the standard of compaction achieved and the placement moisture content (refer Table 5 below).
- Fill should have a maximum particle size of 100 mm for an uncompacted layer thickness of 200 mm and shall be compacted by repeated rolling to achieve the minimum dry density ratios (DDR) noted in Table 5 below.
- In accordance with good construction practice, fill embankments should be 'over built' then trimmed back to the well compacted material.

Table 5: Recommended minimum compaction criteria.

Location	Minimum Dry Density Ratio (%)
>0.3 m below pavement subgrade	95% (standard compaction) or 70% (Density Index)
≤0.3 m below pavement subgrade	97% (standard compaction) or 70% (Density Index)
Base Course	95% (modified compaction)
Subbase	98% (modified compaction)

Earthworks should be carried out in accordance with the Department of Transport and Main Roads Technical Specification 'MRTS04 General Earthworks'.

The settlement of well compacted fill is unlikely to exceed 0.5% to 1% of fill thickness over the design life for clay fill and less for granular fill.

7.11 Erosion Protection

Any excavations or disturbance of materials should be carefully monitored and managed during the construction phase to minimise potential for scour/erosion and sediment run off.

Based on the results of field and laboratory testing on the soils in Precinct A, it is assessed that there is a moderate to high potential for erosion if the subgrade soils are exposed during or following construction.

Exposed soil (i.e., on embankment matters and around the abutments) should be protected from erosion by appropriate measures such as re-vegetation and hydro-seeding, protective matting, or hard facings.

Temporary sediment control should be adopted during construction where surface water will flow offsite. Concentrating surface runoff should be avoided unless adequate erosion protection and flow dissipation is constructed.

7.12 Aggressivity of Soils

Chemical analysis can provide an indication of the potential for long term damage to foundations, buried pipelines, in-ground structures, services and other infrastructure. Soil texture is also important in this regard, as granular soils allow oxygen exchange (oxidation) to occur more readily and are also more permeable.

Australian Standard AS 2159-2009 *Piling - Design and installation* provides the range of exposure classification of the surface of steel piles and concrete piles based on the range of chemical conditions in the soil and the possibility of changes in groundwater levels.

The results of the soil aggressiveness testing indicate that the residual sandy clay/clayey sand materials are *Mild*, while the underlying sandstone is *Non-aggressive* towards concrete piles in soil in accordance with the definitions given in Australian Standard AS 2159-2009 Table 6.4.2(C) Exposure Classification for Concrete Piles – Piles in Soil.

8.0 LIMITATIONS

Should you require any further information please contact the undersigned. We draw your attention to the document, Limitations, which is included in Appendix D.

Core Consultants Pty Ltd



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Appendix A

Figure 1: Site Plan

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Legend

- Contour Lines (Department of Resources)
 - 1 m Contour
 - 5 m Contour
 - <all other values>
- Approximate Borehole Location (Core 2022)
- Approximate Borehole Location (Douglas Partners 2022)

Base plan drawn by Pitt & Sherry. Drawing Title - General Arrangement Sheet 1 of 5. Drawing No. - P22.0175.00-STR-DRG-002. Revision. - A. Dated 1 July 2022.
Aerial image sourced from Nearmap Pty Ltd, image dated 16 August 2022. Annotations by Core Consultants Pty Ltd.

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www.coreconsultants.com.au

CLIENT FRASERS PROPERTY NEW BEITH PTY LTD		PROJECT 'PRECINCT A' SUBDIVISION - MOUNTAIN RIDGE ROAD, NEW BEITH			
DRAWN BY MD	DATE 21/02/2023	DRAWING TITLE TEST LOCATION PLAN			
CHECKED BY SM	DATE 21/02/2023				
SCALE 1:500	SHEET SIZE A3	PROJECT No J001800	DOC No 003	DOC TYPE R	FIGURE No F001
		REVISION 1	SHEET 1 OF 1		

Plot Date: 26 February 2023 Time: 10:13 AM By: CAD Path: J:\2022 - J1479\J001800 - Frasers-Subdiv-New Beith\PHASE 1000\Figures Drawings\J001800-GIS-001\J001800.aprx

Appendix B

Reports of Boreholes, Core Photographs & Explanatory Notes

REPORT OF BOREHOLE: BHA

Client	Frasers Property New Beith Pty Ltd
Project	Proposed Rail Overpass Bridge
Location	Mountain Ridge Road, New Beith
Job No.	J001800

East	495938.0 m
North	6926534.0 m MGA94 Zone 56
Contractor	Drillsure Geotechnical
Drill Rig	Tracked Hydrapower Scout
Inclination	-90° Hole Dia. 100 mm

Sheet 1 OF 3
 Logged: MD
 Logged Date: 10/01/23
 Checked: SM
 Checked Date: 16/02/23

[illegible]

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Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495938.0 m
North 6926534.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 2 OF 3
Logged: MD
Logged Date: 10/01/23
Checked: SM
Checked Date: 16/02/23

Drilling					Field Material Description			Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations
				DEPTH RL			VL 0.1 L 0.3 M 0.3 H 1 VH 10 EH		Average Defect Spacing (mm)
				0					
				1.00		Continuation from non-cored borehole			
	100%	100	98			SANDSTONE: fine to medium grained, pale brown, massive, medium to high strength, moderately weathered.	MW		1.59-1.61: J, 0°, Clay In, PI, Ro, extremely closely spaced
				2.50		Blue grey.	SW		
	100%	100	100						
				4.90		Occasional coal veins.			
	100%	100	100						
				7.20		Coal clasts up to 60 mm in size.			7.84: DB, 0°, PI, Ro, Possible
	100%	100	100						
				8					

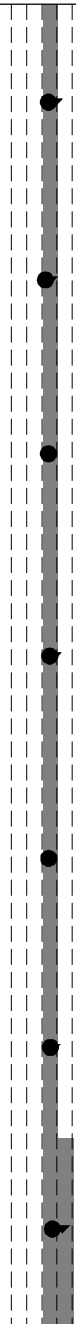
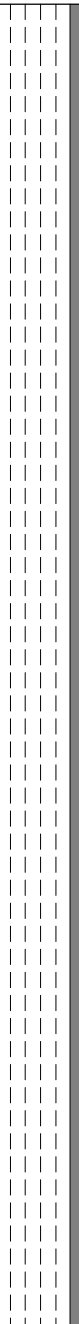
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REPORT OF BOREHOLE: BHA

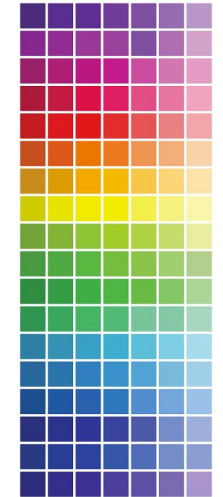
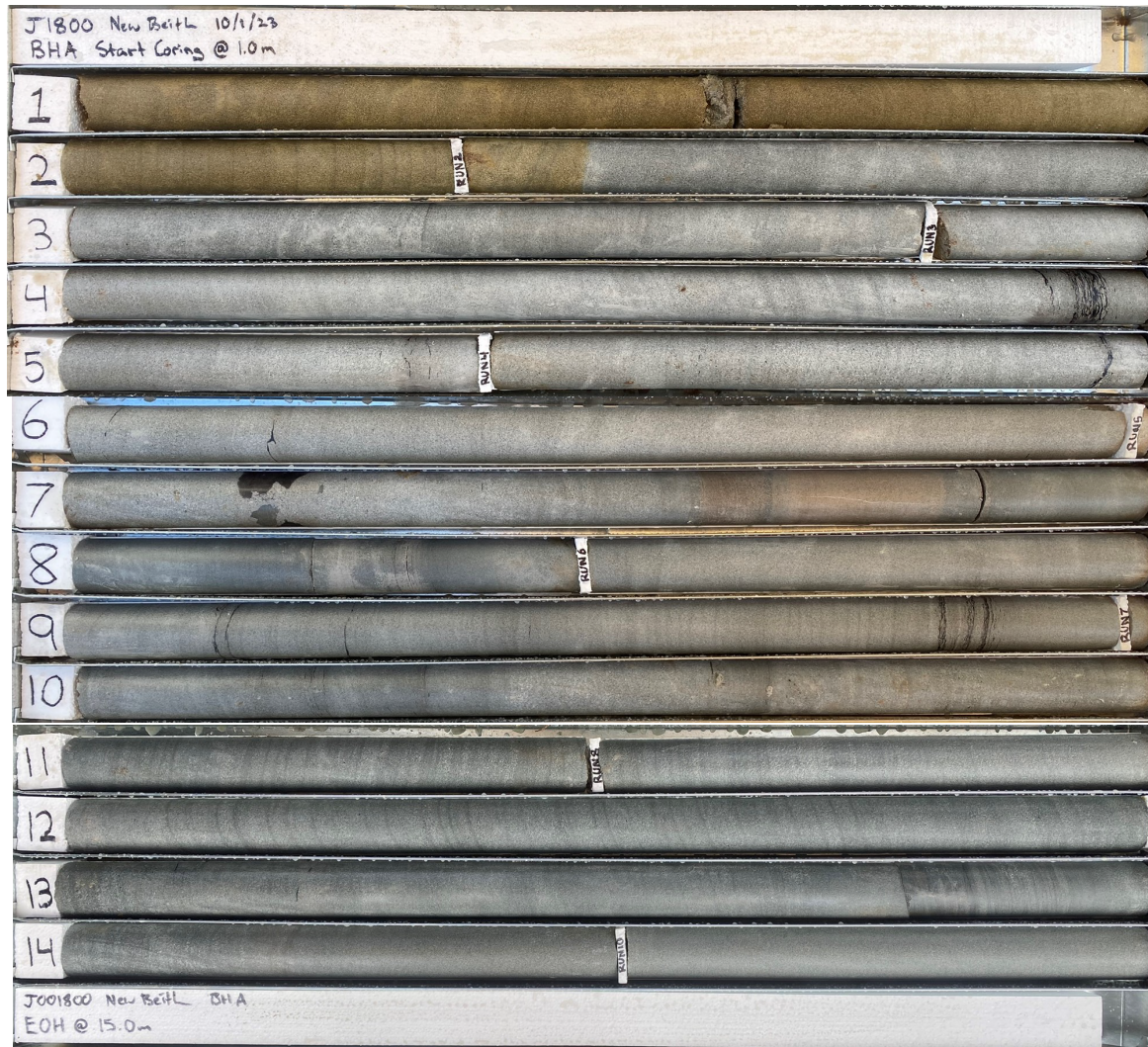
Client	Frasers Property New Beith Pty Ltd
Project	Proposed Rail Overpass Bridge
Location	Mountain Ridge Road, New Beith
Job No.	J001800

East	495938.0 m
North	6926534.0 m MGA94 Zone 56
Contractor	Drillsure Geotechnical
Drill Rig	Tracked Hydrapower Scout
Inclination	-90° Hole Dia. 100 mm

Sheet 3 OF 3
 Logged: MD
 Logged Date: 10/01/23
 Checked: SM
 Checked Date: 16/02/23

Drilling					Field Material Description			Defect Information									
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)						
									V_L 0.1 L 0.3 W 1 H 1 V_{x3} 3 V_{x10} 10 E_H		30 100 300 1000 3000						
NMLC	100%	100	100	8			SANDSTONE: fine to medium grained, pale brown, massive, medium to high strength, moderately weathered.	SW		8.22: HB							
	100%	100	100	9													
	100%	100	100	10													
	100%	100	100	11													
	100%	100	100	12													
	100%	100	100	13													
	100%	100	100	14													
	100%	100	100	15													
				15.00									Hole Terminated at 15.00 m Target depth Backfilled				

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PointID : BHA Depth Range: 1.0 - 15.0 m



TITLE

PROPOSED RAIL OVERPASS BRIDGE,
MOUNTAIN RIDGE ROAD, NEW BEITH

BHA
Core Photo

DRAWN

KB

DATE

14/02/2023

CHECKED

SM

DATE

22/02/2023

SCALE

Not To Scale

A4

PROJECT No

J001800

FIGURE No

1/1

REPORT OF BOREHOLE: BHB

Client	Frasers Property New Beith Pty Ltd
Project	Proposed Rail Overpass Bridge
Location	Mountain Ridge Road, New Beith
Job No.	J001800

East	495951.0 m
North	6926529.0 m MGA94 Zone 56
Contractor	Drillsure Geotechnical
Drill Rig	Tracked Hydrapower Scout
Inclination	-90° Hole Dia. 100 mm

Sheet 1 OF 3
 Logged: MD
 Logged Date: 11/01/23
 Checked: SM
 Checked Date: 16/02/23

[illegible]

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Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495951.0 m
North 6926529.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 2 OF 3
Logged: MD
Logged Date: 11/01/23
Checked: SM
Checked Date: 16/02/23

Drilling						Field Material Description				Defect Information		
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)	
								VL L M H VH EH			30 100 300 1000 3000	
				0								
				1								
				1.60			Continuation from non-cored borehole					
				1.70			SANDSTONE: fine to medium grained, pale brown and brown, massive, very low strength, highly weathered.	HW				
				1.90			Silty Sandy CLAY: medium plasticity, pale brown and brown; sand fine to medium grained.	XW				
				2			SANDSTONE: fine to medium grained, pale brown and orange brown, massive. Blue grey, occasional coal veins.	HW		2.02-2.04: J, 0°, Clay In, PI, Ro, extremely closely spaced		
				2.30				MW		2.33: J, 5°, Cn, PI-Un, Ro		
								SW		2.56-2.82: J, 5°, Cn, PI-Un, Ro, closely spaced		
				3						3.20: HB		
				4								
				5								
				6								
				7								
				8								

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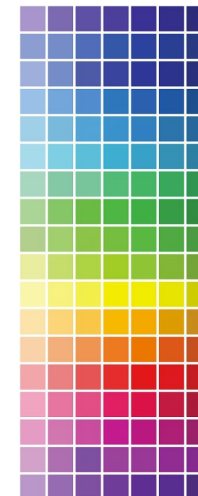
Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495951.0 m
North 6926529.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 3 OF 3
Logged: MD
Logged Date: 11/01/23
Checked: SM
Checked Date: 16/02/23

Drilling					Field Material Description				Defect Information			
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)	
								VL 0.1 L 0.3 M 0.3 H 1 VH 3 EH 10			30 100 300 1000 3000	
NMLC	100%	100	100	8			SANDSTONE: fine to medium grained, pale brown and orange brown, massive.	SW		9.18: J, 15°, Coal Ct, Pl, Ro 		

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PointID : BHB Depth Range: 1.6 - 15.0 m

core
consultants pty ltd

TITLE

PROPOSED RAIL OVERPASS BRIDGE,
MOUNTAIN RIDGE ROAD, NEW BEITH

BHB
Core Photo

DRAWN

KB

DATE

14/02/2023

CHECKED

SM

DATE

22/02/2023

SCALE

Not To Scale

A4

PROJECT No

J001800

FIGURE No

1/1

Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495994.0 m
North 6926494.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 1 OF 3
Logged: MD
Logged Date: 09/01/23
Checked: SM
Checked Date: 16/02/23

Drilling				Sampling		Field Material Description														
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	GROUP SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY	DENSITY	DCP TEST Blows per 100 mm							
													0	5	10	15	20	25		
ADT	L-M		0	0.20	DS 0.40-0.80 m				FILL Silty Sandy GRAVEL: fine to coarse grained, sub-angular to angular, blue grey and pale brown; sand fine to coarse grained. Silty SAND (POSSIBLE RESIDUAL SOIL): fine to medium grained, pale brown. Sandy CLAY (RESIDUAL SOIL) with silt: medium plasticity, grey with mottled red brown; sand fine to medium grained. Tending to silty sand and extremely weathered material.	D	VD									
			D																	
	M		0.35	DS 1.20-1.50 m									SANDSTONE: fine to medium grained, pale brown, massive, very low strength, highly weathered.	St / VSt	VSt					
			w < PL		VSt															
	H		1.00		SPT 1.50-1.74 m 14,30/90mm N=R				Very low to low strength.					D	H					
			1.20																	
			1.65																	
			1.74																	
					2						Continued as Cored Borehole									
					3															
			4																	
			5																	
			6																	
			7																	

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Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495994.0 m
North 6926494.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 2 OF 3
Logged: MD
Logged Date: 09/01/23
Checked: SM
Checked Date: 16/02/23

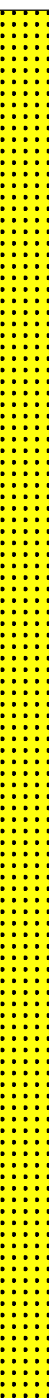
Drilling					Field Material Description			Defect Information		
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)
							VL 0.1 L 0.3 M 0.3 H 1 VH 10 EH			30 100 300 1000 3000
				0						
				1						
				1.74		Continuation from non-cored borehole				
				2		SANDSTONE: fine to medium grained, pale orange brown and grey, massive.	HW		2.07-2.16: J, 10°, Clay Ct, Pl, Ro, closely spaced	
				2.60		Pale brown with grey.			2.31-2.41: J, 10°, Cn, Pl, Ro, very closely spaced 2.32-2.62: J, 80°, Clay Ct, Un, Ro	
				3					2.90-2.98: J, 5°, Ve, Pl, Ro, closely spaced 3.05-3.07: J, 5°, Ve, Pl, very closely spaced	
				4		Pale brown.	XW HW MW XW HW MW XW HW MW		3.26-3.31: EWS, 5 - 10°, Clay, Pl 3.46-3.59: J, 0 - 5°, Ve, Pl, Ro, very closely spaced 3.60-3.63: EWS, 5°, Clay, Pl 3.76: J, 0°, Ve, Pl, Ro 3.90-3.92: EWS, 5°, Clay, Pl	
				5		Blue grey.	MW		4.54-4.60: J, 5°, Sn, Pl, Ro, very closely spaced 4.89-4.95: J, 8°, Ve, Pl, Ro, very closely spaced	
				5.60		Grey brown.			5.83-5.91: J, 15°, Ve, Pl, Ro, very closely spaced 6.04-6.18: J, 10°, Ve, Pl, Ro, very closely spaced	
				6					6.26: J, 45°, Ve, Pl, Ro 6.48: J, 5°, Ve, Pl, Ro	
				6.62		Blue grey.	HW			
				7						
				8						

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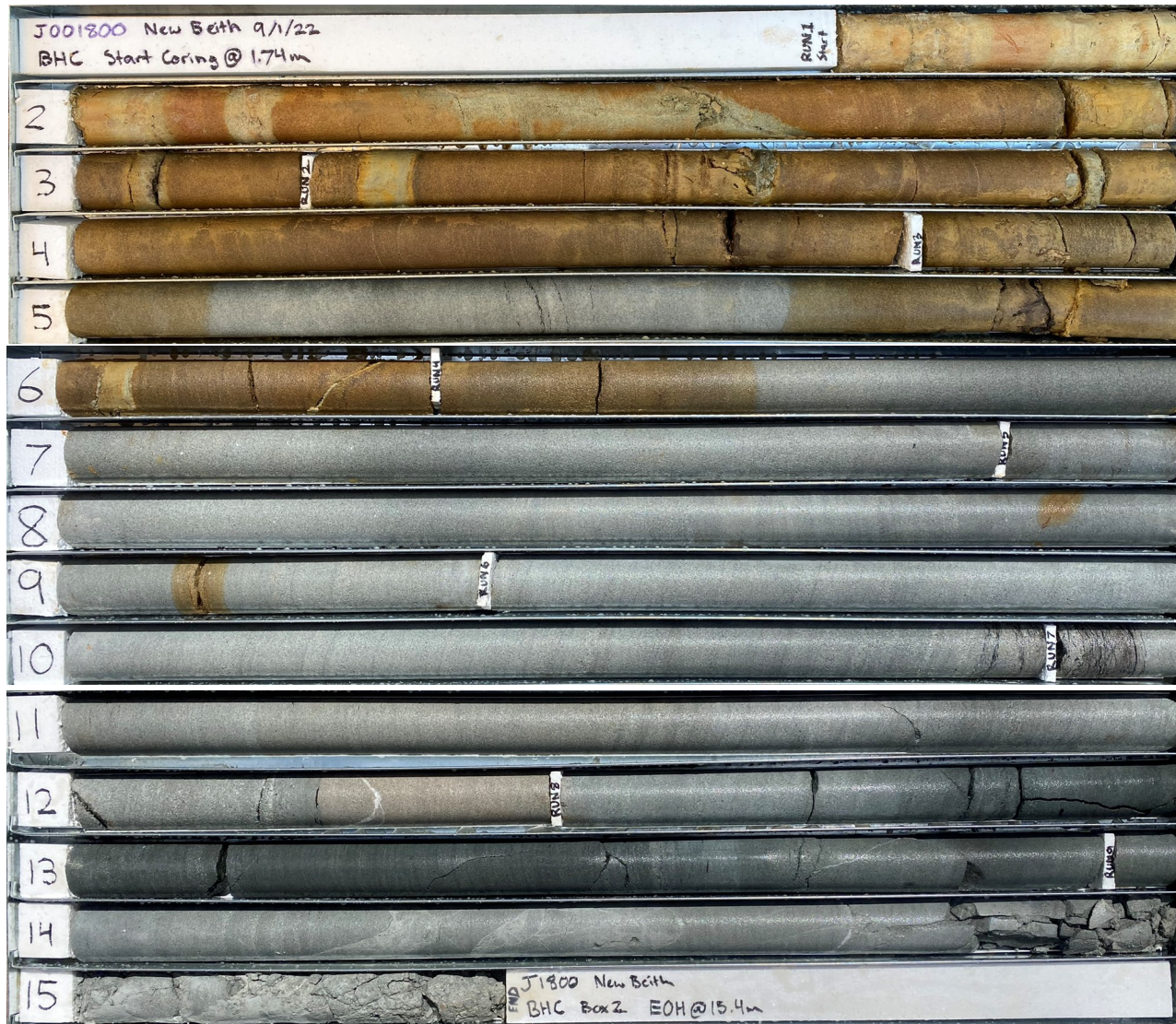
Client Frasers Property New Beith Pty Ltd
Project Proposed Rail Overpass Bridge
Location Mountain Ridge Road, New Beith
Job No. J001800

East 495994.0 m
North 6926494.0 m MGA94 Zone 56
Contractor Drillsure Geotechnical
Drill Rig Tracked Hydrapower Scout
Inclination -90° Hole Dia. 100 mm

Sheet 3 OF 3
Logged: MD
Logged Date: 09/01/23
Checked: SM
Checked Date: 16/02/23

Drilling						Field Material Description			Defect Information			
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations		Average Defect Spacing (mm)
								VL L M H VH EH				30 100 300 1000 3000
NMLC			100	100	8		SANDSTONE: fine to medium grained, pale orange brown and grey, massive.	SW				
					9					9.13: J, 5°, Ve, Pl, Ro		
			100	100	10							
					10.80			Zones of fine to coarse grained.				
			100	100	11					11.74: J, 45°, Cn, Pl, Ro		
					12					12.00: J, 30°, Cn, Pl, Ro 12.18: J, 10°, Cn, Pl, Ro		
					12.23			Fine to medium grained, occasional quartz veins.				
			100	53	13					12.67: J, 5°, Cn, Pl, Ro 12.81-12.85: J, 5°, Cn, Pl, Ro 12.85-13.06: J, 80 - 90°, Cn, Un, Ro		
					14					13.40: J, 75°, Cn, Pl, Ro 13.50-13.67: J, 15°, Cn, Pl-Un, Ro		
			100	59	15				MW		13.78: J, 45°, Cn, Pl, Ro	
				15.40						15.01-15.40: FZ, 0 - 90°, Cn, Pl, Ro		
							Hole Terminated at 15.40 m Target depth Backfilled					

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PointID : BHC Depth Range: 1.74 -15.4 m

core
consultants pty ltd

TITLE

PROPOSED RAIL OVERPASS BRIDGE,
MOUNTAIN RIDGE ROAD, NEW BEITH

BHC
Core Photo

DRAWN	KB	DATE	14/02/2023
CHECKED	SM	DATE	22/02/2023
SCALE	Not To Scale		A4
PROJECT No	J001800	FIGURE No	1/1

EXPLANATION OF NOTES, ABBREVIATIONS & TERMS USED ON BOREHOLE AND TEST PIT REPORTS

DRILLING/EXCAVATION METHOD

AS	Auger Screwing	RD	Rotary blade or drag bit	NQ	Diamond Core - 47 mm
AD	Auger Drilling	RT	Rotary Tricone bit	NMLC	Diamond Core - 52 mm
*V	V - Bit	RAB	Rotary Air Blast	HQ	Diamond Core - 63 mm
T	TC - Bit, e.g. ADT	RC	Reverse Circulation	HMLC	Diamond Core – 63mm
HA	Hand Auger	PT	Push Tube	BH	Tractor Mounted Backhoe
ADH	Hollow Auger	CT	Cable Tool Rig	EX	Tracked Hydraulic Excavator
DTC	Diatubre Coring	JET	Jetting	EE	Existing Excavation
WB	Washbore or Bailer	NDD	Non-destructive digging	HAND	Excavated by Hand Methods

PENETRATION/EXCAVATION RESISTANCE

L	Low resistance . Rapid penetration possible with little effort from the equipment used
M	Medium resistance . Excavation possible at an acceptable rate with moderate effort from equipment used
H	High resistance to penetration/excavation . Further penetration is possible at a slow rate
R	Refusal or Practical Refusal . No further progress possible without the risk of damage or unacceptable wear to the digging implement or machine.

These assessments are subjective and are dependent on many factors including the equipment power, weight, condition of excavation or drilling tools, and the experience of the operator.

WATER

	Water level shown at date		Partial water loss
	Water inflow		Complete water loss
GROUNDWATER NOT OBSERVED	The observation of groundwater whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.		
GROUND WATER NOT ENCOUNTERED	The borehole/test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/test pit been left open for a longer period.		

SAMPLING AND TESTING

SPT	Standard Penetration Test to AS1289.6.3.1-2004	
4,7,11 N=18	4,7,11 = Blows per 150mm	N = Blows per 300mm penetration following 150mm seating
30/80mm	Where practical refusal occurs, the blows and penetration for that interval are reported	
RW	Penetration occurred under the rod weight only	
HW	Penetration occurred under the hammer and rod weight only	
HB	Hammer double bouncing on anvil	
DS	Disturbed Sample	
BDS	Bulk disturbed sample	
G	Gas Sample	
W	Water sample	
FP	Field permeability test over section noted	
FV	Field vane shear test expressed as uncorrected shear strength (sv = peak value)	
PID	Photoionisation Detector reading in ppm	
PM	Pressuremeter test over section noted	
PP	Pocket penetrometer test expressed as instrument reading in kPa	
U63	Thin walled tube sample - number indicates nominal sample diameter in millimetres	
WPT	Water pressure tests	
DCP	Dynamic cone penetration test	
CPT	Dynamic cone penetration test	
CPTu	Static cone penetration test with pore pressure (u) measurement	

ROCK CORE RECOVERY

TCR = Total Core Recovery (%)

SCR = Solid Core Recovery (%)

RQD = Rock Quantity Designation (%)

$$= \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100$$

$$= \frac{\sum \text{Length of cylindrical core recovered}}{\text{Length of core run}} \times 100$$

$$= \frac{\sum \text{Axial lengths of core} > 100 \text{ mm}}{\text{Length of core run}} \times 100$$

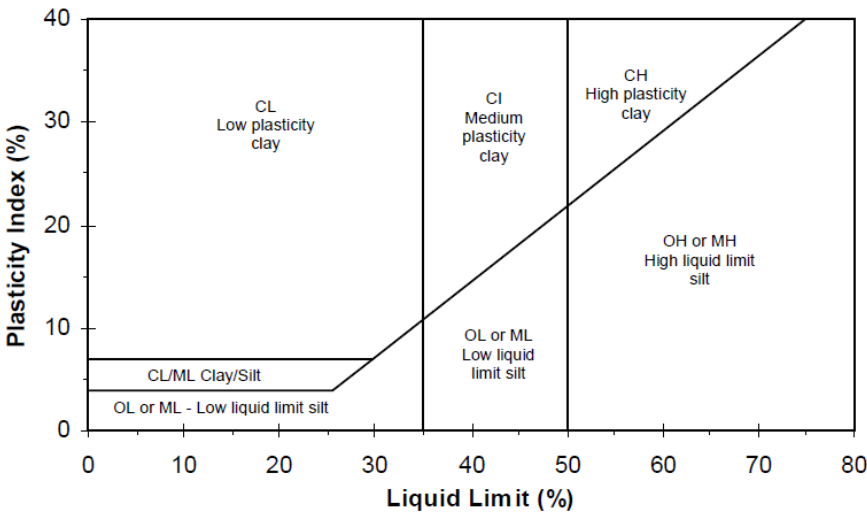
METHOD OF SOIL DESCRIPTION USED ON BOREHOLE AND TEST PIT REPORTS

	FILL		CLAY (CL, CI, or CH)
	GRAVEL (GP or SW)		ORGANIC SOILS (OL or OH or Pt)
	SAND (SP or SW)		COBBLES or BOULDERS
	SILT (ML or MH)		

Combinations of these basic symbols may be used to indicate mixed materials such as sandy clay.

CLASSIFICATION AND INFERRED STRATIGRAPHY

Soil and Rock is classified and described in Reports of Boreholes and Test Pits using the preferred method given in AS 1726 - 2017. The material properties are assessed in the field by visual/tactile methods.

PARTICLE SIZE			PLASTIC PROPERTIES
Major Division	Sub Division	Particle Size	
Boulders		>200 mm	
Cobbles		63 - 200 mm	
Gravel	Coarse	20 - 63 mm	
Gravel	Medium	6.7 - 20 mm	
Gravel	Fine	2.36 - 6.7 mm	
Sand	Coarse	0.6 - 2.36 mm	
Sand	Medium	0.21 - 0.6 mm	
Sand	Fine	0.075 - 0.21 mm	
Silt		0.002 - 0.075 mm	
Clay		<0.002 mm	

MOISTURE CONDITION FOR COARSE GRAINED SOIL AS 1726 - 2017

Symbol	Term	Description
D	Dry	Non-cohesive and free running
M	Moist	Soil feels cool, darkened in colour, tends to stick together
W	Wet	Soil feels cool, darkened in colour, soil sticks together, free water forms when handling

MOISTURE CONDITION FOR FINE GRAINED SOIL AS1726 - 2017

Symbol	Term	Description
W<PL	Moist dry of liquid limit	Hard and friable or powdery
W = PL	Moist near plastic limit	Soils can be molded at a moisture condition approximately equal to the plastic limit
W >PL	Moist, wet of plastic limit	Soils usually weakened and free water forms on hands when handling
W = LL	Wet near plastic limit	
W > LL	Wet, wet of liquid limit	

CONSISTENCY TERMS FOR COHESIVE SOILS		AS1726—2017	RELATIVE DENSITY OF COARSE GRAINED SOILS			AS1726—2017
Symbol	Term	Undrained Shear Strength	Symbol	Term	Density Index %	SPT 'N' #
VS	Very Soft	0 to 12 kPa	VL	Very Loose	Less than 15	0 to 4
S	Soft	12 to 25 kPa	L	Loose	15 to 35	4 to 10
F	Firm	25 to 50 kPa	MD	Medium Dense	35 to 65	10 to 30
St	Stiff	50 to 100 kPa	D	Dense	65 to 85	30 to 50
VSt	Very Stiff	100 to 200 kPa	VD	Very Dense	Above 85	Above 50
H	Hard	Above 200 kPa	In the absence of test results, consistency and density may be assessed from correlations with the observed behaviour of the material.			

TERMS FOR ROCK MATERIAL STRENGTH & WEATHERING AND ABBREVIATIONS FOR DEFECT DESCRIPTIONS

ROCK MATERIAL STRENGTH CLASSIFICATION				AS1726—2017
Symbol	Term	Uniaxial Compressive Strength (MPa)	Point Load Strength I_s (50) (MPa)	Field Guide
VL	Very Low Strength	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick. Pieces up to 30 mm thick can be broken with finger pressure.
L	Low Strength	2 to 6	0.1 to 0.3	Easily scored with knife. Indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point. A piece of core 150 mm by 50 mm may be broken by hand. Sharp edges of core are friable and break during handling.
M	Medium Strength	6 to 20	0.3 to 1	Readily scored with a knife. A piece of core 150 mm by 50 mm can be broken by hand with difficulty.
H	High Strength	20 to 60	1 to 3	A piece of core 150 mm by 50 mm cannot be broken by hand but can be broken by a pick with a single firm blow. Rock rings under hammer.
VH	Very High Strength	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow. Rock rings under hammer.
EH	Extremely High Strength	Above 200	Above 10	Specimen requires many blows with geological pick to break through intact material. Rock rings under hammer.

● = Diametral Point Load Test ▼ = Axial Point Load Test

CLASSIFICATION OF MATERIAL WEATHERING		AS1726—2017
Symbol	Term	Field Guide
RS	Residual Soil (<i>Note 1</i>)	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible but the soil has not been significantly transported.
XW	Extremely Weathered (<i>Note 1</i>)	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
HW	Highly Weathered (<i>Note 2</i>)	The whole rock mass is discoloured, usually by iron staining or beaching to the extent that the colour of the original rock is not recognizable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
MH	Moderately Weathered (<i>Note 2</i>)	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognizable, but shows little or no change in strength from fresh rock.
SW	Slightly Weathered	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
FR	Fresh	Rock shows no signs of decomposition of individual minerals or colour change.
Note 1	The term 'Extremely Weathered rock' is misleading as the material has soil properties. The word 'rock' should be replaced with the name of the original rock of the word 'material', eg. Extremely Weathered granite or Extremely Weathered material.	
Note 2	Where it is not possible to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock the term 'Distinctly Weathered' may be used.	

DEFECT TYPE/DESCRIPTION				DEFECT PROFILE		DEFECT ROUGHNESS	
				Symbol	Description	DESCRIPTION	
B	Bedding Parting	V	Vein	PL	Planar	Symbol	Description
J	Joint	HB/DB	Handling/Drilling Break	St	Stepped		
EW	Extremely Weathered Seam	C	Contact	Un	Undulating		
FZ	Fracture Zone	L	Cleavage	DEFECT INFILL DESCRIPTION		Vertical Boreholes - The dip (inclination from horizontal) for the defect is given. Inclined Boreholes - The inclination is measured as the acute angle to the core axis.	
CZ/S	Crushed Zone/Seam	X	Foliation	Symbol	Description		
IS	Infilled Seam	S	Schistosity	Cn	Clean: No visible coating		
SZ/S	Sheared Zone/Seam			Sn	Stain: Coated 1 to 3 mm		
				Vr	Veneer: < 1 mm		
				Ct	Coating: 1 to 3 mm		

Appendix C

Geotechnical Laboratory Test Certificates

Material Test Report

Report Number: 202937.00-143
Issue Number: 1
Date Issued: 16/02/2023
Client: Core Consultants Pty Ltd
Unit 1/18 Lysaght Street, Coolumb Beach QLD 4573
Contact: Simon Maggiora
Project Number: 202937.00
Project Name: Proposed Development
Project Location: Material Testing, Maroochydore QLD
Client Reference: J001800
Work Request: 21956
Sample Number: SS-21956I
Date Sampled: 13/12/2022
Dates Tested: 30/01/2023 - 14/02/2023
Sampling Method: Sampled by Client
The results apply to the sample as received
Sample Location: BHB, Depth: 0.9 - 1.1 m



Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Shae Harry

Laboratory Manager

Laboratory Accreditation Number: 828

Particle Size Distribution (AS1289 3.6.1)			
Sieve	Passed %	Passing Limits	
0.075 mm	45		

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	40		
Plastic Limit (%)	18		
Plasticity Index (%)	22		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	10.0		
Cracking Crumbling Curling	None		

Core Consultants Pty Ltd
18 Lysaght St
Coolum Beach
QLD 4573



NATA Accredited
Accreditation Number 1261
Site Number 1254

Accredited for compliance with ISO/IEC 17025 – Testing
NATA is a signatory to the ILAC Mutual Recognition
Arrangement for the mutual recognition of the
equivalence of testing, medical testing, calibration,
inspection, proficiency testing scheme providers and
reference materials producers reports and certificates.

Attention: **Simon Maggiora**

Report **966066-S**
Project name **PROPOSED BRIDGE**
Project ID **J001800**
Received Date **Feb 22, 2023**

Client Sample ID			BHC_SPT (1.5 - 1.74m)
Sample Matrix			Soil
Eurofins Sample No.			B23-Fe0052235
Date Sampled			Jan 09, 2023
Test/Reference	LOR	Unit	
Chloride	5	mg/kg	300
Conductivity (1:5 aqueous extract at 25 °C as rec.)	10	uS/cm	180
pH (1:5 Aqueous extract at 25 °C as rec.)	0.1	pH Units	7.8
Resistivity*	0.5	ohm.m	56
Sulphate (as SO4)	30	mg/kg	98
Sample Properties			
% Moisture	1	%	29

Sample History

Where samples are submitted/analysed over several days, the last date of extraction is reported.

If the date and time of sampling are not provided, the Laboratory will not be responsible for compromised results should testing be performed outside the recommended holding time.

Description	Testing Site	Extracted	Holding Time
Chloride - Method: LTM-INO-4090 Chloride by Discrete Analyser	Melbourne	Feb 25, 2023	28 Days
Conductivity (1:5 aqueous extract at 25 °C as rec.) - Method: LTM-INO-4030 Conductivity	Melbourne	Feb 25, 2023	7 Days
pH (1:5 Aqueous extract at 25 °C as rec.) - Method: LTM-GEN-7090 pH in soil by ISE	Melbourne	Feb 25, 2023	7 Days
Sulphate (as SO ₄) - Method: LTM-INO-4110 Sulfate by Discrete Analyser	Melbourne	Feb 25, 2023	28 Days
% Moisture - Method: LTM-GEN-7080 Moisture	Melbourne	Feb 23, 2023	14 Days

Company Name: Core Consultants Pty Ltd
Address: 18 Lysaght St
Coolum Beach
QLD 4573

Project Name: PROPOSED BRIDGE
Project ID: J001800

Order No.:
Report #: 966066
Phone: 07 5475 5900
Fax:

Received: Feb 22, 2023 5:35 PM
Due: Mar 2, 2023
Priority: 5 Day
Contact Name: Simon Maggiora

Eurofins Analytical Services Manager : Alana Wadsworth

Sample Detail						Aggressivity Soil Set	Moisture Set
Melbourne Laboratory - NATA # 1261 Site # 1254						X	X
External Laboratory							
No	Sample ID	Sample Date	Sampling Time	Matrix	LAB ID		
1	BHC_SPT (1.5 - 1.74m)	Jan 09, 2023		Soil	B23-Fe0052235	X	X
Test Counts						1	1

Internal Quality Control Review and Glossary

General

- Laboratory QC results for Method Blanks, Duplicates, Matrix Spikes, and Laboratory Control Samples follows guidelines delineated in the National Environment Protection (Assessment of Site Contamination) Measure 1999, as amended May 2013 and are included in this QC report where applicable. Additional QC data may be available on request.
- All soil/sediment/solid results are reported on a dry basis, unless otherwise stated.
- All biota/food results are reported on a wet weight basis on the edible portion, unless otherwise stated.
- Actual LORs are matrix dependant. Quoted LORs may be raised where sample extracts are diluted due to interferences.
- Results are uncorrected for matrix spikes or surrogate recoveries except for PFAS compounds.
- SVOC analysis on waters are performed on homogenised, unfiltered samples, unless noted otherwise.
- Samples were analysed on an 'as received' basis.
- Information identified on this report with blue colour, indicates data provided by customer that may have an impact on the results.
- This report replaces any interim results previously issued.

Holding Times

Please refer to 'Sample Preservation and Container Guide' for holding times (QS3001).

For samples received on the last day of holding time, notification of testing requirements should have been received at least 6 hours prior to sample receipt deadlines as stated on the SRA.

If the Laboratory did not receive the information in the required timeframe, and regardless of any other integrity issues, suitably qualified results may still be reported.

Holding times apply from the date of sampling, therefore compliance to these may be outside the laboratory's control.

For VOCs containing vinyl chloride, styrene and 2-chloroethyl vinyl ether the holding time is 7 days however for all other VOCs such as BTEX or C6-10 TRH then the holding time is 14 days.

Units

mg/kg: milligrams per kilogram	mg/L: milligrams per litre	µg/L: micrograms per litre
ppm: parts per million	ppb: parts per billion	%: Percentage
org/100 mL: Organisms per 100 millilitres	NTU: Nephelometric Turbidity Units	MPN/100 mL: Most Probable Number of organisms per 100 millilitres
CFU: Colony forming unit		

Terms

APHA	American Public Health Association
COC	Chain of Custody
CP	Client Parent - QC was performed on samples pertaining to this report
CRM	Certified Reference Material (ISO17034) - reported as percent recovery.
Dry	Where a moisture has been determined on a solid sample the result is expressed on a dry basis.
Duplicate	A second piece of analysis from the same sample and reported in the same units as the result to show comparison.
LOR	Limit of Reporting.
LCS	Laboratory Control Sample - reported as percent recovery.
Method Blank	In the case of solid samples these are performed on laboratory certified clean sands and in the case of water samples these are performed on de-ionised water.
NCP	Non-Client Parent - QC performed on samples not pertaining to this report, QC is representative of the sequence or batch that client samples were analysed within.
RPD	Relative Percent Difference between two Duplicate pieces of analysis.
SPIKE	Addition of the analyte to the sample and reported as percentage recovery.
SRA	Sample Receipt Advice
Surr - Surrogate	The addition of a like compound to the analyte target and reported as percentage recovery.
TBTO	Tributyltin oxide (<i>bis</i> -tributyltin oxide) - individual tributyltin compounds cannot be identified separately in the environment however free tributyltin was measured and its values were converted stoichiometrically into tributyltin oxide for comparison with regulatory limits.
TCLP	Toxicity Characteristic Leaching Procedure
TEQ	Toxic Equivalency Quotient or Total Equivalence
QSM	US Department of Defense Quality Systems Manual Version 5.4
US EPA	United States Environmental Protection Agency
WA DWER	Sum of PFBA, PFPeA, PFHxA, PFHpA, PFOA, PFBS, PFHxS, PFOS, 6:2 FTSA, 8:2 FTSA

QC - Acceptance Criteria

The acceptance criteria should be used as a guide only and may be different when site specific Sampling Analysis and Quality Plan (SAQP) have been implemented

RPD Duplicates: Global RPD Duplicates Acceptance Criteria is 30% however the following acceptance guidelines are equally applicable:

Results <10 times the LOR: No Limit

Results between 10-20 times the LOR: RPD must lie between 0-50%

Results >20 times the LOR : RPD must lie between 0-30%

NOTE: pH duplicates are reported as a range not as RPD

Surrogate Recoveries: Recoveries must lie between 20-130% for Speciated Phenols & 50-150% for PFAS

PFAS field samples that contain surrogate recoveries in excess of the QC limit designated in QSM 5.4 where no positive PFAS results have been reported have been reviewed and no data was affected.

QC Data General Comments

- Where a result is reported as a less than (<), higher than the nominated LOR, this is due to either matrix interference, extract dilution required due to interferences or contaminant levels within the sample, high moisture content or insufficient sample provided.
- Duplicate data shown within this report that states the word "BATCH" is a Batch Duplicate from outside of your sample batch, but within the laboratory sample batch at a 1:10 ratio. The Parent and Duplicate data shown is not data from your samples.
- pH and Free Chlorine analysed in the laboratory - Analysis on this test must begin within 30 minutes of sampling. Therefore, laboratory analysis is unlikely to be completed within holding time. Analysis will begin as soon as possible after sample receipt.
- Recovery Data (Spikes & Surrogates) - where chromatographic interference does not allow the determination of recovery the term "INT" appears against that analyte.
- For Matrix Spikes and LCS results a dash "-" in the report means that the specific analyte was not added to the QC sample.
- Duplicate RPDs are calculated from raw analytical data thus it is possible to have two sets of data.

Quality Control Results

Test				Units	Result 1			Acceptance Limits	Pass Limits	Qualifying Code
Method Blank										
Chloride				mg/kg	< 5			5	Pass	
Sulphate (as SO4)				mg/kg	< 30			30	Pass	
LCS - % Recovery										
Chloride				%	104			70-130	Pass	
Sulphate (as SO4)				%	101			70-130	Pass	
Test	Lab Sample ID	QA Source		Units	Result 1			Acceptance Limits	Pass Limits	Qualifying Code
Spike - % Recovery										
					Result 1					
Chloride	M23-Fe0062703	NCP		%	2.6			70-130	Fail	Q08
Test	Lab Sample ID	QA Source		Units	Result 1			Acceptance Limits	Pass Limits	Qualifying Code
Duplicate										
					Result 1	Result 2	RPD			
Conductivity (1:5 aqueous extract at 25 °C as rec.)	B23-Fe0057938	NCP		uS/cm	130	110	22	30%	Pass	
pH (1:5 Aqueous extract at 25 °C as rec.)	B23-Fe0057938	NCP		pH Units	6.9	6.7	pass	30%	Pass	
Resistivity*	B23-Fe0057938	NCP		ohm.m	76	95	22	30%	Pass	
Sulphate (as SO4)	M23-Fe0059201	NCP		mg/kg	69	72	3.0	30%	Pass	
Duplicate										
Sample Properties					Result 1	Result 2	RPD			
% Moisture	S23-Fe0051510	NCP		%	21	22	3.5	30%	Pass	

Comments

Sample Integrity

Custody Seals Intact (if used)	N/A
Attempt to Chill was evident	Yes
Sample correctly preserved	Yes
Appropriate sample containers have been used	Yes
Sample containers for volatile analysis received with minimal headspace	Yes
Samples received within HoldingTime	Yes
Some samples have been subcontracted	No

Qualifier Codes/Comments

Code	Description
Q08	The matrix spike recovery is outside of the recommended acceptance criteria. An acceptable recovery was obtained for the laboratory control sample indicating a sample matrix interference.

Authorised by:

Zoe Flynn	Analytical Services Manager
Linda Chouman	Senior Analyst-Sample Properties
Scott Beddoes	Senior Analyst-Inorganic



Glenn Jackson
General Manager

Final Report – this report replaces any previously issued Report

- Indicates Not Requested

* Indicates NATA accreditation does not cover the performance of this service

Measurement uncertainty of test data is available on request or please [click here](#).

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AXIAL TEST

CLIENT : Frasers Property New Beith Pty Ltd

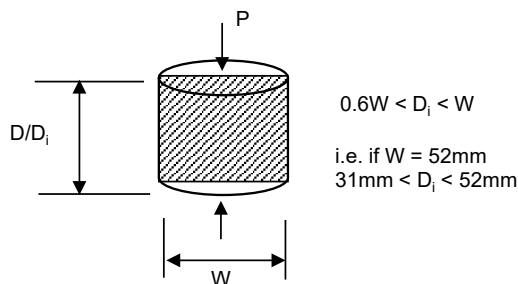
PROJECT : Proposed Rail Bridge Crossing

LOCATION : Mountain Ridge Road, New Beith

JOB NO : J001800

Point Load Testing Device EL No. : 611

Sample Moisture Condition and Storage History : Dry



Borehole Number	Depth (m)	Width (W) (mm)	Length (D _i) /Width (W) Ratio	Platen Sep. at Failure (D) (mm)	Gauge Load (P) (kN)	D _e ² (mm ²)	I _s (MPa)	F	I _{s(50)} (MPa)	Strength Classification, Failure Type*, Rock Tested, Comment	
BHC	6.42	41.00	1.27	36.0	4.03	1879.3	2.14	0.94	2.01	H	
BHC	7.4	35.0	1.5	32.0	2.58	1426.03	1.81	0.88	1.59	H	
BHC	8.5	33.0	1.6	28.0	2.20	1176.47	1.87	0.84	1.58	H	
BHC	9.3	36.0	1.4	33.0	2.63	1512.61	1.74	0.89	1.55	H	
BHC	10.3	42.0	1.2	39.0	3.51	2085.57	1.68	0.96	1.62	H	
BHC	11.1	33.0	1.6	32.0	3.95	1344.54	2.94	0.87	2.56	H	
BHC	12.2	47.0	1.1	41.0	3.92	2453.53	1.60	1.00	1.59	H	
BHC	14.1	40.0	1.3	34.0	4.86	1731.61	2.81	0.92	2.58	H	
BHC	15.1	47.0	1.1	40.0	1.21	2393.69	0.51	0.99	0.50	M	
BHB	2.7	33.0	1.6	30.0	2.82	1260.51	2.24	0.86	1.92	H	
BHB	3.7	35.0	1.5	32.0	1.90	1426.03	1.33	0.88	1.17	H	
BHB	4.7	40.0	1.3	37.0	2.84	1884.39	1.51	0.94	1.41	H	
BHB	5.6	40.0	1.3	39.0	2.02	1986.25	1.02	0.95	0.97	M	
BHB	6.7	35.0	1.5	33.0	2.70	1470.59	1.84	0.89	1.63	H	
BHB	7.4	48.0	1.1	44.0	8.71	2689.08	3.24	1.02	3.29	VH	
BHB	8.6	34.0	1.5	30.0	3.04	1298.70	2.34	0.86	2.02	H	
BHB	9.5	50.0	1.0	48.0	3.02	3055.77	0.99	1.05	1.03	H	
BHB	10.5	41.0	1.3	37.0	3.93	1931.50	2.03	0.94	1.92	H	
BHB	11.6	40.0	1.3	35.0	5.02	1782.54	2.82	0.93	2.61	H	
BHB	12.5	45.0	1.2	40.0	4.12	2291.83	1.80	0.98	1.76	H	
BHB	13.6	43.0	1.2	40.0	3.57	2189.97	1.63	0.97	1.58	H	
BHB	14.6	44.0	1.2	39.0	7.94	2184.88	3.63	0.97	3.53	VH	
BHA	1.6	45.0	1.2	40.0	2.28	2291.83	0.99	0.98	0.98	M	
BHA	2.4	48.0	1.1	44.0	2.59	2689.08	0.96	1.02	0.98	M	
BHA	3.5	50.0	1.0	47.0	3.10	2992.11	1.04	1.04	1.08	H	
BHA	4.5	40.0	1.3	38.0	2.93	1935.32	1.51	0.94	1.43	H	
BHA	5.4	43.0	1.2	40.0	4.29	2189.97	1.96	0.97	1.90	H	
BHA	6.5	45.0	1.2	40.0	2.96	2291.83	1.29	0.98	1.27	H	
BHA	8.5	38.0	1.4	33.0	4.44	1596.64	2.78	0.90	2.51	H	
BHA	9.5	49.0	1.1	45.0	4.81	2807.49	1.71	1.03	1.76	H	
BHA	10.4	50.0	1.0	48.0	4.62	3055.77	1.51	1.05	1.58	H	
BHA	11.5	40.0	1.3	36.0	4.48	1833.46	2.44	0.93	2.28	H	
BHA	12.5	44.0	1.2	39.0	3.51	2184.88	1.61	0.97	1.56	H	
BHA	13.5	41.0	1.3	39.0	4.55	2035.91	2.23	0.95	2.13	H	
BHA	14.5	45.0	1.2	27.0	7.61	1546.99	4.92	0.90	4.42	VH	

Borehole Number	Depth (m)	Width (W) (mm)	Length (D _l) /Width (W) Ratio	Platen Sep. at Failure (D) (mm)	Gauge Load (P) (kN)	D _e ² (mm ²)	I _s (MPa)	F	I _{s(50)} (MPa)	Strength Classification, Failure Type*, Rock Tested, Comment																					
NOTES :																															
* 1 = Fracture through fabric of specimen oblique to bedding, not influenced by weak planes.						Sampled By / Date : MD 11/01/2023 - 13/01/2023																									
2 = Fracture along bedding.						Tested By / Date : KB 1/02/2023																									
3 = Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.						Computed By / Date : KB 1/02/2023																									
4 = Chip or partial fracture.						Checked By / Date : SM 16/02/2023																									
Diametral Test FORMULA																															
Equivalent Core Diameter (D _e)		$D_e^2 = D^2$				<table><tr><th>Symbol</th><th>Term</th><th>I_{s(50)} (MPa)</th></tr><tr><td>VL</td><td>Very Low</td><td>>0.03 to 0.10</td></tr><tr><td>L</td><td>Low</td><td>>0.10 to 0.30</td></tr><tr><td>M</td><td>Medium</td><td>>0.30 to 1</td></tr><tr><td>H</td><td>High</td><td>>1 to 3</td></tr><tr><td>VH</td><td>Very High</td><td>>3 to 10</td></tr><tr><td>EH</td><td>Extremely High</td><td>> 10</td></tr></table>					Symbol	Term	I _{s(50)} (MPa)	VL	Very Low	>0.03 to 0.10	L	Low	>0.10 to 0.30	M	Medium	>0.30 to 1	H	High	>1 to 3	VH	Very High	>3 to 10	EH	Extremely High	> 10
Symbol	Term	I _{s(50)} (MPa)																													
VL	Very Low	>0.03 to 0.10																													
L	Low	>0.10 to 0.30																													
M	Medium	>0.30 to 1																													
H	High	>1 to 3																													
VH	Very High	>3 to 10																													
EH	Extremely High	> 10																													
Uncorrected Point Load Strength (I _s)		$I_s = \frac{1000 P}{D_e^2} (MPa)$																													
Size Correction Factor (F)		$F = \left(\frac{D_e}{50} \right)^{0.45}$																													
Point Load Strength Index (I _{s(50)})		$I_{s(50)} = I_s F$																													

DIAMETRAL TEST

CLIENT : Frasers Property New Beith Pty Ltd

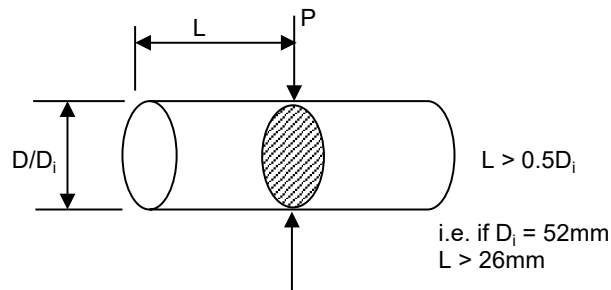
PROJECT : Proposed Rail Bridge Crossing

LOCATION : Mountain Ridge Road, New Beith

JOB NO : J001800

Point Load Testing Device EL No. : 611

Sample Moisture Condition and Storage History : Dry



Borehole Number	Depth (m)	Length (L) (mm)	Length (L)/Dia (D _i) Ratio	Platen Sep at Failure (D) (mm)	Gauge Load (P) (kN)	I _s (MPa)	F	I _{s(50)} (MPa)	Strength Classification, Failure Type*, Rock Tested, Comment	
BHC	6.42	150.00	0.35	47.0	4.65	2.11	0.97	2.05	H	
BHC	7.4	80.0	0.7	49.0	2.18	0.91	0.99	0.90	M	
BHC	8.5	160.0	0.3	49.0	3.76	1.57	0.99	1.55	H	
BHC	9.3	110.0	0.5	50.0	2.69	1.08	1.00	1.08	H	
BHC	10.3	90.0	0.6	49.0	3.11	1.30	0.99	1.28	H	
BHC	11.1	180.0	0.3	49.0	4.13	1.72	0.99	1.70	H	
BHC	12.2	100.0	0.5	50.0	4.00	1.60	1.00	1.60	H	
BHC	14.1	160.0	0.3	49.0	4.63	1.93	0.99	1.91	H	
BHC	15.1	120.0	0.4	47.0	6.84	3.10	0.97	3.01	VH	
BHB	2.7	70.0	0.7	50.0	4.67	1.87	1.00	1.87	H	
BHB	3.7	80.0	0.7	50.0	1.60	0.64	1.00	0.64	M	
BHB	4.7	110.0	0.5	49.0	4.25	1.77	0.99	1.75	H	
BHB	5.6	95.0	0.5	50.0	2.10	0.84	1.00	0.84	M	
BHB	6.7	90.0	0.6	50.0	1.82	0.73	1.00	0.73	M	
BHB	7.4	90.0	0.6	50.0	2.70	1.08	1.00	1.08	H	
BHB	8.6	80.0	0.7	50.0	3.06	1.22	1.00	1.22	H	
BHB	9.5	110.0	0.5	50.0	2.12	0.85	1.00	0.85	M	
BHB	10.5	85.0	0.6	50.0	2.61	1.04	1.00	1.04	H	
BHB	11.6	80.0	0.7	47.0	4.39	1.99	0.97	1.93	H	
BHB	12.5	120.0	0.4	50.0	2.85	1.14	1.00	1.14	H	
BHB	13.6	85.0	0.6	50.0	2.71	1.08	1.00	1.08	H	
BHB	14.6	100.0	0.5	48.0	5.38	2.34	0.98	2.29	H	
BHA	1.6	95.0	0.5	50.0	2.20	0.88	1.00	0.88	M	
BHA	2.4	95.0	0.5	50.0	2.22	0.89	1.00	0.89	M	
BHA	3.5	100.0	0.5	48.0	2.64	1.15	0.98	1.12	H	
BHA	4.5	80.0	0.7	49.0	2.27	0.95	0.99	0.94	M	
BHA	5.4	100.0	0.5	49.0	3.19	1.33	0.99	1.32	H	
BHA	6.5	85.0	0.6	50.0	2.18	0.87	1.00	0.87	M	
BHA	8.5	105.0	0.5	49.0	3.80	1.58	0.99	1.57	H	
BHA	9.5	110.0	0.5	50.0	3.12	1.25	1.00	1.25	H	

Borehole Number	Depth (m)	Length (L) (mm)	Length (L)/ Dia (D _i) Ratio	Platen Sep at Failure (D) (mm)	Gauge Load (P) (kN)	I _s (MPa)	F	I _{s(50)} (MPa)	Strength Classification, Failure Type*, Rock Tested, Comment
BHA	10.4	110.0	0.5	50.0	3.96	1.58	1.00	1.58	H
BHA	11.5	90.0	0.6	50.0	4.32	1.73	1.00	1.73	H
BHA	12.5	100.0	0.5	49.0	3.91	1.63	0.99	1.61	H
BHA	13.5	110.0	0.5	49.0	4.44	1.85	0.99	1.83	H
BHA	14.5	110.0	0.5	50.0	5.36	2.14	1.00	2.14	H

NOTES :

- * 1 = Fracture through fabric of specimen oblique to bedding, not influenced by weak plane
 2 = Fracture along bedding.
 3 = Fracture influenced by pre-existing plane, microfracture, vein or chemical alteration.
 4 = Chip or partial fracture.

Sampled By / Date : MD 11/01/2023 - 13/01/2023

Tested By / Date : KB 1/02/2023

Computed By / Date : KB 1/02/2023

Checked By / Date : SM 16/02/2023

Diametral Test FORMULA

$$D_e^2 = D^2$$

Equivalent Core Diameter (D_e)

$$I_s = \frac{1000 P}{D_e^2} (MPa)$$

Uncorrected Point Load Strength (I_s)

$$F = \left(\frac{D_e}{50} \right)^{0.45}$$

Size Correction Factor (F)

$$I_{s(50)} = I_s F$$

Point Load Strength Index (I_{s(50)})

Symbol	Term	I _{s(50)} (MPa)
VL	Very Low	>0.03 to 0.10
L	Low	>0.10 to 0.30
M	Medium	>0.30 to 1
H	High	>1 to 3
VH	Very High	>3 to 10
EH	Extremely High	> 10

UNIAXIAL COMPRESSIVE STRENGTH & DEFORMATION TEST REPORT

Test Method: AS 4133.4.3.1

Client	Core Consultants	Report No.	23020182-MOD
Address	1/12 Lysaght Street, Coolum Beach QLD 4573	Workorder No.	0010672
Project	J001800 - Proposed Bridge	Test Date	9/02/2023
Client ID	BHC - NMLC	Report Date	15/02/2023
Description	-	Depth (m)	7.00-7.25
Sample Type	Single Individual Rock Core Specimen		
Uniaxial Compressive Strength 19.9 MPa			

Young's Modulus

Tangent 3.62 GPa

Secant 2.31 GPa

Poisson Ratio

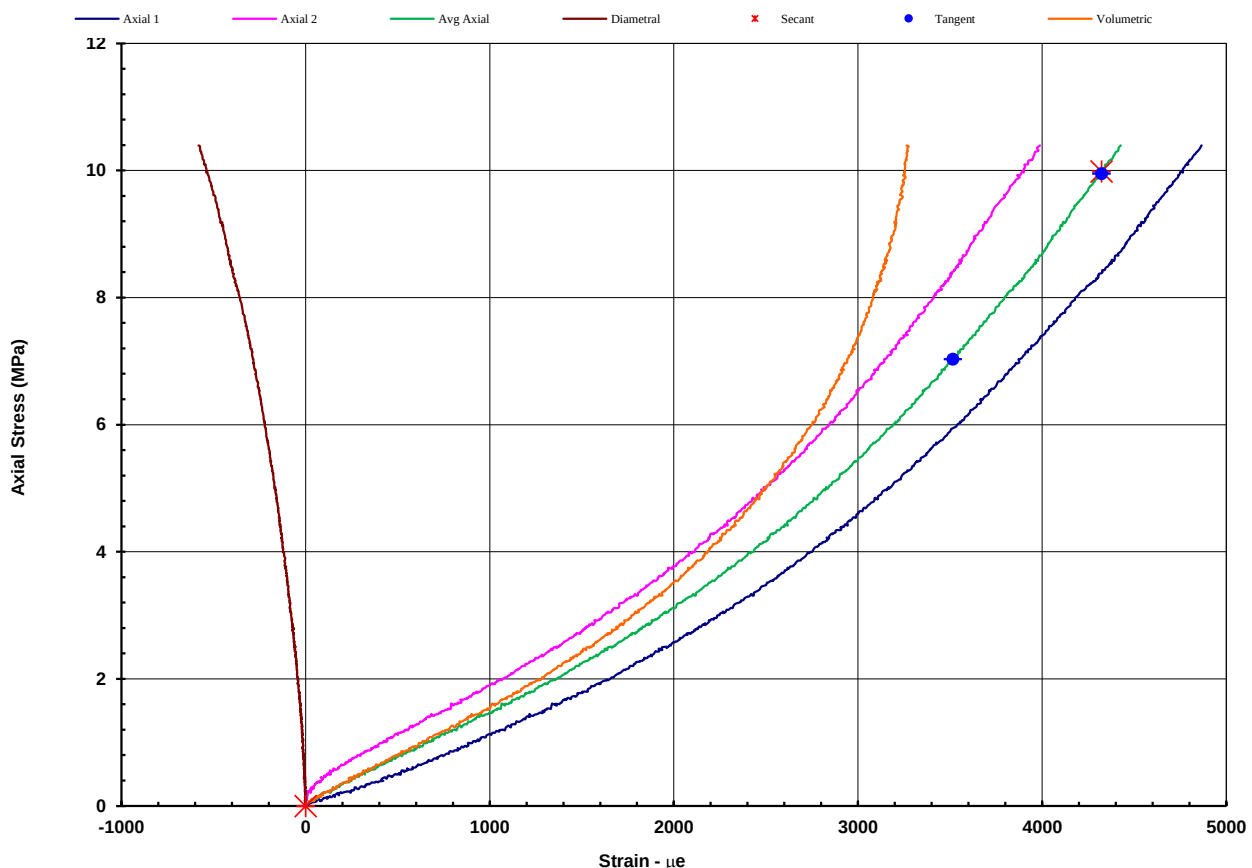
0.309

0.124

from 35 % to 50 % of Max UCS

from 0 % to 50 % of Max UCS

Axial Stress vs Strain Plots



Notes/Remarks:

Sample/s supplied by client

Graph not to scale

Tested as received.

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Accredited for compliance with ISO/IEC 17025 - Testing

*NATA is a signatory to the ILAC Mutual Recognition Arrangement for the mutual recognition of the equivalence of testing, medical testing, calibration, inspection, proficiency testing scheme providers and reference materials producers reports and certificates

Tested at Trilab Brisbane Laboratory

Authorised Signatory

J. Rasmussen



Laboratory No. 9926

The results of calibrations and tests performed apply only to the specific instrument or sample at the time of test unless otherwise clearly stated.

Reference should be made to Trilab's "Standard Terms and Conditions of Business" for further details.

Trilab Pty Ltd

ABN 25 065 630 506

ACCURATE QUALITY RESULTS FOR TOMORROW'S ENGINEERING

UNIAXIAL COMPRESSIVE STRENGTH & DEFORMATION TEST REPORT

Test Method: AS 4133.4.3.1

Client	Core Consultants	Report No.	23020182-MOD
Average Sample Diameter (mm)	52.1	Moisture Content (%)	4.9
Sample Height (mm)	138.4	Wet Density (t/m ³)	2.38
Duration of Test (min)	8:56	Dry Density (t/m ³)	2.27
Rate of Loading (MPa/min)	2.23	Bedding (°)	Nil
Mode of Failure	Conical	Test Apparatus	Kelba 1000kN Load Cell
Rupture Angle (°)	60		

CLIENT:	Core Consultants
PROJECT:	J001800 - Proposed Bridge
LAB SAMPLE No.	23020182
BOREHOLE:	BHC - NMLC
	BEFORE TEST
	DATE: 6-2-23
	DEPTH: 7.00-7.25



Notes/Remarks:

Sample/s supplied by client

Graph not to scale

Tested as received.

Page 2 of 2 REP03603

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ACCURATE QUALITY RESULTS FOR TOMORROW'S ENGINEERING

Appendix D

Limitations

LIMITATIONS

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