SPECIFICATION

Tonkin+Taylor

Rotorua Lakefront Development - Stage 1 and Stage 1A

Technical Specification

Prepared for
Rotorua Lakes Council
Prepared by
Tonkin & Taylor Ltd
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Table of contents

1 Extent of Contract and General Clauses

1.1 Location

The work included in this contract and to which this Technical Specification and Drawings refer is located at the proposed Stage 1 and Stage 1A Rotorua Lakefront Development, Lakefront Drive, Rotorua.

1.2 Extent of contract

The Contract Works comprise construction of the civil works components of the Lakefront Development Stage 1 and Stage 1A areas in accordance with the Drawings and this Specification. The work includes but is not limited to:

- Earthworks
- Ground improvement
- Ground anchors / micro piles
- Concrete
- Pipework – gravity systems

1.3 Drawings

The following drawings (collectively known as the Drawings) listed in the drawing list on Drawing Number 1007467.3000-001 are included in and form part of the Contract Documents.

The Contractor shall inform the Engineer immediately of any apparent errors, inconsistencies or omissions in any of the Drawings. The Engineer will respond to any such information within two Days by means of clarification, confirmation or instruction.

The Contractor shall maintain one full set of drawings at the Site at all times specifically for recording As-Built locations and details. One full set of the marked up drawings shall be supplied to the Engineer by the Contractor on completion of construction with all as-built information marked up legibly in red.

1.4 Standard specifications

The standard specifications referred to herein shall be deemed to be incorporated in the Specification and shall apply to those sections of the Contract Works to which they are relevant. Reference to a standard specification refers to the latest edition at the date of closing of tenders and includes the latest amendments or revisions to that standard specification.

1.5 Vibration levels

The contractor shall ensure that vibrations generated from construction activities in close proximity to third party buildings or structures are less than the maximum limits recommended in the German standard diN 4150-3, Structural Vibration – Effects of Vibration on Structures. The maximum peak particle velocity (ppv) limits which are to be applied are summarised in table 1.2 or as otherwise required by the resource consent.

Vibration monitoring shall be undertaken by the contractor when:

- required by the resource consent, or
- In the opinion of the engineer, construction operations are likely to result in (or are likely resulting in) the vibration limits being exceeded at the site boundaries, or
- If any vibration complaints are received from neighbours.

Where required by the resource consent, the contractor shall prepare a construction vibration plan in accordance with this section.

In the first instance the contractor shall undertake vibration monitoring at the property boundary. Where vibration levels at the boundary exceed the specified limits or where complaints are received, additional monitoring inside the neighbouring property shall be undertaken (subject to access agreement with the neighbours). Such additional monitoring shall be undertaken either outside the building but connected to the building foundation or inside the building. Vibration measurement instruments shall be placed by the contractor at varying distances from the site to ensure that vibration is within the limits set out in the diN standard 4150-3, and to ensure that effects from vibration on occupants of adjacent structures are no more than minor and the occupants shall not be subject to unreasonable disturbance.

Where vibration monitoring is required, the contractor shall undertake monitoring when construction operations are most likely to result in maximum vibration at the site boundaries, or when construction plant or methodology changes that is likely to result in an exceedance of the above vibration limits. All vibration monitoring shall be undertaken by appropriately qualified and experienced personnel. The contractor shall note that building occupants may notice or be disturbed by vibrations at levels lower than those cited in diN 4150-3 (i.e. vibration is perceptible at levels above 0.3mm/s), which might give rise to complaints. This is often due to occupants concern over building damage, and can generally be mitigated through effective consultation regarding the project objectives and timeframes, vibration monitoring and demonstration to the occupant that vibration levels are within acceptable levels. The contractor and the engineer shall collectively take responsibility for liaison and discussions with neighbouring occupants to relay this information to them and to keep them informed of construction activities and programme. On request of the engineer, the contractor shall provide vibration records to the occupants. The contractor shall also take all practical steps to minimise disruption to the occupants as much as possible.

Type of Structure	**Maximum Permitted Vibration Velocity (mm/s)**			
	Intermittent Vibration (frequency)			**Continuous Vibration (all frequencies)**
	Less than 10 Hz	**10 to 50 Hz**	**50 to 100 Hz(1)**	
Sensitive Structures	3	3 to 8	8 to 10	3
Residential Structures	5	5 to 15	15 to 20	5
Commercial Structures	20	20 to 40	40 to 50	5

Note 1. For frequencies about 100Hz, at least the value specified in this column shall be applied.

1.6 Inspection and approval

In addition to the requirements for inspection contained elsewhere in the contract, the Contractor shall give the Engineer at least one (1) working days' notice that they wish to proceed to the following stages of the Works.

a commencement of work
b laying of pipelines
c laying of basecourse or filters
d cement stabilisation

e pouring of each precast concrete unit and allow inspection of reinforcing

f placing of any structural concrete

g covering of any structural concrete including installation of timber decking

h covering foundations including the placement of drainage and the backfilling behind retaining walls

i testing of any part of the Works as required under the Contract

j inspection for Practical Completion.

The Contractor shall not proceed to any stage of the Works until the Engineer has inspected, approved and where necessary measured the Works at the previous stage.

The Contractor shall be responsible for notifying the local council and arranging any Building Consent inspections by the council staff.

1.7 As-builts

One full set of the drawings shall be supplied to the Engineer by the Contractor on completion of construction with all as-built information marked up legibly in red.

In addition, the Contractor shall employ an independent Registered Surveyor to survey the "as-built works" and provide the Engineer with the following as-built plans within the timeframes specified in the General Conditions of Contract:

- subsoil drain layout and levels
- finished subgrade surface levels
- Finished filled surface levels
- Earthworks test locations
- Ground improvement test locations
- Plan showing extent and finished levels of ground improvement / cement stabilised foundation
- Plan showing extent of reinforced concrete pad foundations for the boardwalk and bridges
- Plan showing ground anchor / micro pile locations including a table of anchor depth below ground
- Any new pipework or services including lid levels, invert levels, bedding and connections.

All drawings shall be the same size – either A1 or A3 as appropriate to clearly show the required level of detail. All drawings shall clearly show the relevant extent of construction and shall:

- have an appropriate title, unique drawing number, and revision
- show the name of the company who prepared the drawing
- show the name (or initials) of the surveyor and the drawing preparer
- show the date of the survey (where appropriate)
- show a north point
- show coordinated grid lines and tick marks
- identify the coordinate system and level datums
- clearly identify all features and lines marked on the drawing
- clearly show the reduced level next to each survey point (where appropriate)
- show labelled unsmoothed contours at 0.2 m intervals (where appropriate).

The survey drawings shall be submitted to the Engineer for review. Once the drawings have been amended as marked up by the Engineer, they shall be certified by the Registered Surveyor as a true and correct record. Final drawings shall be submitted to the Engineer in hard copy form along with electronic copies in DXF or DWG format. Electronic drawings shall have all data in 3D.

2 Bulk Earthworks

2.1 Scope

This section covers all the works necessary to cut and fill the Site to the required levels, grades and standards, other than soil stabilisation work. Works covered include:

- Clearing the site
- Stripping topsoil
- Excavation of unsuitable materials to waste or designated areas
- Cut to waste or fill
- Controlled filling using on-site or imported approved materials
- Topsoil placing

2.2 Standard specifications, resource consents and guidelines

Construction work performed under this Section shall:

- Be carried out in accordance with New Zealand Transport Agency (NZTA) Specification TNZ F/1 : 1997 except where modified in this Specification
- Comply with the general requirements of the latest revisions of the following documents:

NZS 4402 : 1986	Methods of Testing Soils for Civil Engineering Purposes
BS 1377 : 1999	Methods of Test for Soils for Civil Engineering Purposes
NZS 3111 : 1986	Methods of Test for Water and Aggregate for Concrete
NZS 4407 : 1991	Methods of sampling and Testing Road Aggregate

- Comply with the general requirements of the latest revisions of all other Standards, specifications and Codes of Practice referenced in these Contract Documents
- Comply with the specific requirements of this Section and the Drawings
- Be carried out in full consideration of and in full compliance with the Resource Consents issued for this project by the *Bay of Plenty Regional Council* and of any associated management plans.
- Be carried out in accordance with the *"Erosion and Sediment Control Guidelines for Land Disturbing Activities", June 2010, issued by Environment Bay of Plenty* and with any subsequent and additional requirements of *the Bay of Plenty Regional Council.*

The Specification shall be read in conjunction with the above Standards and documents, which are deemed to form a part of this Specification. In the event of any requirements of this Specification being at variance with any of the above Standards and documents then the requirements of this Specification take precedence.

2.3 Definition of fill types

2.3.1 Bulk fill

Defines all general fill placed to form the required levels and to provide founding for the structures, access roads, services and similar. Bulk fill shall contain no unsuitable material, rubbish or topsoil. It shall be placed at slopes no steeper than 1V: 2H without reinforcement,

2.3.2 Landscape fill

Defines material that makes up the filling for landscaping areas and shall contain no unsuitable material, rubbish or topsoil. Landscape fill shall not be placed at steeper than 1V : 4H unless instructed otherwise by the Engineer.

2.3.3 Hardfill (GAP 65)

Defines a well graded aggregate, with slightly weathered to unweathered fragments of rock up to maximum 65 mm characteristic dimension and which is relatively free of fines and other mineral matter such that when compacted the rock fragments can achieve point-to-point contact. Rock is as defined in TNZ F/1, (i.e. any igneous, sedimentary, or metamorphic stone which is solidly bonded, or cemented together and which occurs in masses, ledges, seams, or layers).

2.3.3.1 Grading

The aggregate shall have a grading when tested according to NZS 4407:1991 Test 3.8.1 "The Particle Size Distribution – Preferred Method by Wet Sieving" which falls within the limits defined in Table 2.1 below:

Table 2.1: Grading envelope

Hardfill (GAP65)	Percentage passing	
Aperture size (mm)	**Lower limit**	**Upper limit**
65	100	100
37.5	80	90
19	50	70
9.5	30	55
4.75	20	40
2.36	15	30
1.18	10	22
0.6	6	18
0.3	4	14
0.15	2	10
0.075	0	7

2.3.4 Rockfill

Defines a well graded aggregate with slightly weathered to unweathered fragments of rock up to maximum 300 mm characteristic dimension (unless specified on Drawings) and which is relatively free of fines and other mineral matter such that when compacted the rock fragments can achieve point-to-point contact. Rock is as defined in TNZ F/1, (i.e. any igneous, sedimentary, or metamorphic stone which is solidly bonded, or cemented together and which occurs in masses, ledges, seams, or layers).

2.3.5 Pumiceous fill

Defines engineered fill material placed directly behind retaining walls and any other areas, as shown on the Drawings. Pumiceous fill shall contain no unsuitable material, rubbish or topsoil. The pumiceous fill shall comprise imported sandy pumice gravel which meets the specified grading, strength and bulk density criteria as defined in this specification.

2.3.6 Unsuitable material

Defines material that is either organic material, other than topsoil, within cuts or fill areas, or material which by its inherent nature cannot be satisfactorily reconditioned by wetting and drying for use as Rock, Structural, Buttress, Bulk or Landscape fill. Unsuitable materials shall be placed in areas designated on the Drawings, placed in on-site stockpiles, sent to an off-site disposal area or otherwise disposed of as instructed by the Engineer.

2.3.7 Rubbish

Rubbish is defined as inorganic material e.g. steel, concrete, plastic, refuse and other debris found during cut and fill operations and is categorised as Unsuitable Material unless otherwise approved by the Engineer.

2.3.8 Topsoil

Topsoil is defined as the layer of organic material immediately below the ground level that is unsuitable for use as Bulk, Hardfill, Rock, Pumiceous or Landscape fill, but which is considered by the Engineer to be suitable for re-spreading as a surface soil layer for establishing vegetation growth at the completion of the works.

2.4 General requirements

2.4.1 Earthworks management plan

In the event that a particular sequence of operation is required by the Principal or the Engineer, or because of the nature of the Works, the Contractor shall prepare an Earthworks Management Plan for [each area] of the earthworks. This Plan shall be subject to the approval of the Engineer and shall comply with any internal completion dates or order of carrying out the Works as set out in the Tender. The Earthworks Management Plan shall include:

- Programme(s) clearly showing the proposed earthworks sequence with identification of activities which affect earthworks operations and the effects of those activities on the programme.
- Details of compaction trials, including proposed optimum material handling and treatment, methods and procedures for conditioning of soils, and proposed construction plant requirements.
- An estimate of quantities of the various material types including the proportion of unsuitable and suitable material along the route. The Earthworks Management Plan shall include the location of cut and fill placement for each material type.
- Assessment and method of monitoring volumes of cut, fill, unsuitables and rubbish.
- Details of how the various Consent conditions and other environmental requirements are to be complied with.
- Procedures for monitoring earthworks compaction compliance, i.e. QC procedures, with timely reporting of results and procedures for identifying and rectifying non-compliances.
- The Earthworks Management Plan should be updated and revised as necessary as the works proceed, and in any case not less than once a month.

2.4.2 Drainage control and dewatering

All earthworks shall be carried out in fully drained conditions with no free water on the working surfaces. All preparatory excavation work and subsequent excavations in borrow areas or areas to be filled shall be kept effectively drained at all times. Cut and fill areas shall be sloped and graded adequately at all times so that they do not pond water or allow water to infiltrate. Temporary drains shall be installed or pumping carried out as necessary on a regular basis to remove or deflect water from the areas of operations, or to drain water as soon as it is seen to pond. If the Contractor considers it impracticable to maintain excavations or areas to be filled in a fully drained condition he shall propose, for the Engineer's approval, any measures to revise these drainage requirements.

Any fill or final excavation surface materials which have been allowed to become too wet or soft shall be removed and dried, or replaced. All fill surfaces shall be graded and rolled at the end of each day's work to prevent any ponding and erosion. Prior to commencement of the following day's filling operations, the previously graded and rolled surface shall be scarified by approved plant to remove any softened materials and to prevent the formation of sub-standard, or weak layers within the fill.

2.4.3 Erosion and sediment control

Earthworks shall be undertaken in a controlled manner so that erosion of disturbed areas is kept to a practical minimum and eroded material is confined on site as far as possible. Haul roads shall be treated as disturbed areas. Without exception, any stormwater from disturbed areas shall be directed to temporary silt ponds with erosion and sediment controlled in accordance with the Resource Consents and the Bay of Plenty Regional Council guidelines.

2.4.4 Dust control

Earthmoving shall be carried out and haul roads and cut/fill areas maintained so that dust is not raised near or blown over the working area and adjacent properties. The site shall be kept watered as necessary to meet this requirement until covered by dust-free materials, mulch, or established grass cover.

2.4.5 Clearing and removal of vegetation and rubbish

The Contractor shall remove all vegetation from the area of earthworks, and shall clear all obstructions and rubbish from the area of the Works except those specifically identified by the Engineer as remaining. Clearing shall mean the removal of all growth, (other than grass and weeds), extraction of stumps, and rubbish. Extraction of stumps (if any) shall remove all roots greater than 25 mm diameter. The removal of grass and weeds shall be provided for under removal of topsoil.

Rubbish shall be removed offsite or placed in a location on-site approved by the Engineer.

Care must be exercised to prevent clearing of designated bush or trees and to restrict clearing to only those areas designated in the Drawings, or as approved by the Engineer.

2.4.6 Interpretation of Drawings

The Drawings generally show earthworks with batter intersecting original ground at a sharp change in direction. The Contractor shall round off all the tops of cuts and bottoms of fill batters and shall shape ends of cuts, fills and water table turnouts so as to blend into the surrounding topography. Particular care will be needed at the end of benches to meet any requirement for access whilst blending into the surrounding landform and providing control of stormwater runoff.

The Drawings show earthworks batter and benching only indicatively based on the intersection of cut and fill profiles with original ground levels interpolated from the available survey data. The

Contractor shall be responsible for setting out the earthworks to the required set-out given in the Drawings, or as otherwise instructed, and shall define on site, in advance of any stripping, clearing, or earthworks, the actual limits of the earthworks, clearing and stripping. Should the Contractor identify discrepancies between the Drawings, or as otherwise instructed, and the actual ground profiles when setting out the works then they shall immediately bring such discrepancies to the notice of the Engineer.

2.4.7 Over-excavation

The Contractor shall direct their operations to avoid excavating beyond designated profiles except where specifically instructed. Any excavation beyond these profiles carried out without the express instruction of the Engineer shall be made good, to the approval of the Engineer, with any necessary additional subsoil drainage and appropriately placed, compacted and structurally keyed fill of equal quality to the excavated profile. Such works shall be at the Contractor's own cost.

2.4.8 Preservation and maintenance

The Contractor shall preserve and maintain all earthworks, including partly completed earthworks, within their relevant specified standards, and shall make good, at his own cost, any earthworks which have deteriorated below the specified standards.

The Contractor shall carry out the works so as to minimise passage of construction plant over areas of fill or cut formed to final profiles. Areas of fill or cut that are softened or otherwise damaged due to repeated passage of construction plant shall be undercut and replaced. The Engineer shall inspect and approve the depth and extent of any such undercutting and the requirements for the replacement materials, which will be at the Contractor's own cost.

2.4.9 Tolerances

All earthworks shall be carried out to the lines, levels and grades shown on the Drawings or as otherwise instructed by the Engineer. The accuracy of surfaces to be overlain by metal courses or by concrete structures shall be such as to preserve the minimum thicknesses of the overlying layers. Tolerances shall otherwise be as follows:

• Batters	0 mm to +100 mm
• Other Surfaces	0 mm to + 75 mm

2.4.10 Inspections and approvals

Before cut is commenced or fill is placed in any area, including borrow areas, the Engineer shall be notified so that he can inspect the stripped surface and instruct whether further excavation and/or undercutting and backfilling is required or other works such as drainage are necessary. No cut or fill shall be undertaken in an area until such inspections of the stripped surface, and any other works that may be required below the stripped surface, have been made and the Engineer has approved the commencement of cut and/or fill.

The Contractor shall allow sufficient time for any subsurface and surface inspections and shall programme his operations and provide drainage, access and survey control so that any further works instructed prior to any filling can be carried out in an orderly manner without delay or damage to the works.

Where there is a delay of more than 24 hours between approval of a stripped area and placement of fill, or rainfall has occurred within the vicinity of the stripped area, the Contractor shall obtain a new approval of the surface finish from the Engineer. The surface shall be maintained in its approved condition until filled over.

2.4.11 Haul roads

Any haul roads proposed by the Contractor will be subject to the approval of the Engineer. The construction and use of haul roads shall not compromise the construction and future integrity of the permanent works.

In general, haul roads in cut areas shall be left at 1.5 m, or greater, above finished design levels in order to minimise damage to the permanent works.

2.4.12 Temporary stockpiles

In order to minimise the potential for slope instability the Contractor shall only place stockpiles in locations approved by the Engineer. Any stockpiles shall be constructed to be free draining with overall grades and profiles such as to avoid ponding and minimise erosion.

2.5 Excavation

2.5.1 General

Excavation includes removal of topsoil, excavation to form the cut profiles shown on the Drawings, excavation of fill from borrow areas, removal of unsuitable materials and rubbish, and preparation for drains, shear keys and foundations for structures.

2.5.2 Removal of topsoil

The Contractor shall strip all grass, weeds, turf, organic topsoil, roots and the like from the areas subject to earthworks before other operations commence in these areas. The stripped topsoil shall be separated from other stripped material and separately stockpiled for future reuse in locations shown on the Drawings or areas otherwise approved by the Engineer. The stockpiles shall have maximum heights of 2 m, with slopes not steeper than 1V:2H and all changes of grade rounded to conform generally with the surrounding landscape.

The Contractor shall use appropriate equipment and procedures so as to avoid contaminating or otherwise affecting the topsoil's potential for reuse. In particular the Contractor shall:

- Use excavators and dump trucks to strip the topsoil
- Not use scrapers without prior approval of the Engineer
- Place and spreading the topsoil using excavators from dump trucks and spread by low-ground-pressure tracked equipment.

The depth of topsoil stripping shall be sufficient to remove all organic material, turf and significant plant roots such as to expose soil containing an insignificant amount of organic material to the approval of the Engineer. Except where limited by boundaries, existing works or other limiting features, stripping shall extend 2 metres beyond the limits of areas subject to earthworks or construction. The Contractor shall determine the proposed stripping depth prior to starting operations, and shall avoid unnecessary over-excavation.

2.5.3 Excavation management

Cut areas shall be progressively excavated to form a uniformly graded surface within the batter limits. The Contractor shall form the excavations in a logical and orderly manner to minimise wastage and shall undertake continuous visual inspections of materials as they are excavated. Any unexpected variations in material types or properties, evidence of slip debris or slope instability or observations of buried vegetation, groundwater flows, or seepages should be immediately reported to the Engineer.

The earthworks shall be managed so the appropriate materials are used for the various fill types specified for use in the Contract Documents. The Contractor shall plan his earthworks carefully so as to optimise the use of the available fill materials. In particular the Contractor shall assess the volumes of different materials available, their locations relative to proposed fill areas and the degree of drying and conditioning required for the various materials.

All earthworks surfaces shall be sealed off with rubber tyre plant when rain is imminent to minimise erosion and protect exposed materials from strength loss due to increase in moisture content.

2.5.4 Cut to waste

No excess material shall be removed from the Site without the written approval of the Engineer. All surplus material shall be retained on the Site and shall be stockpiled at the locations shown on the Drawings or areas otherwise approved on Site by the Engineer. The stockpiles shall have maximum heights of 5 m, with slopes not steeper than 1V:2H and all changes of grade rounded to conform generally with the surrounding landscape.

If instructed by the Engineer, cut material, other than topsoil and that required for fill or backfill, shall be carted to an on-site dump area designated in the Contract Documents or approved by the Engineer, or removed from Site and disposed of. The dumped material shall be track-rolled and levelled to the level of the surrounding ground, or as instructed by the Engineer.

2.5.5 Cut to fill

Cut material may be suitable as landscape fill. The Contractor shall propose excavation methods and material handling procedures for the Engineer's approval so as to ensure that the proposed fill materials are not contaminated by organic material or other unsuitables.

2.5.6 General undercutting

The requirements for general undercutting, i.e. within gullies, for the fill foundations and below areas of cut, shall be as follows:

- All organic materials, other unsuitables, materials with an undrained shear strength of less than 80 kPa, or otherwise shown on the Drawings, or instructed by the Engineer, shall be undercut.
- The depth of the undercut in materials will be specified by the Engineer when the material at the subgrade level has been exposed and evaluated.
- On completion of the undercut, the surface shall be shaped, trimmed and compacted so as not to hold water. The compaction shall be as specified in Clause 2.8.
- The Contractor shall treat any excavated undercut material as unsuitables and dispose of accordingly, unless instructed otherwise by the Engineer.

2.5.7 Road and path pavement subgrade undercutting

2.5.7.1 General requirements

The requirements, if any, for subgrade undercutting will be as shown on the Contract Drawings/ as instructed by the Engineer/ will be dependent upon the CBR values of the subgrade material as set out in Table 2.2.

Table 2.2: Subgrade undercut treatment

Subgrade CBR	Treatment
Less than 5%	Undercut a minimum of 1 m, or as advised by the Engineer, backfill with adequately compacted suitable material.
Greater than or equal to 5%	No undercut or stabilisation required

2.5.7.2 Initial subgrade assessment

The Contractor shall inform the Engineer when he proposes to excavate to within 1 m of the road subgrade levels. The Contractor and Engineer shall inspect the cut surface, prior to excavating to subgrade level and shall assess the CBR of the exposed materials. Such assessment shall be carried out by means of visual inspection, Scala Penetrometer Tests, Falling Weight Deflectometer (FWD) testing or other method approved by the Engineer. The nature and extent of the testing shall be proposed by the Contractor and approved by the Engineer. Based on the above assessment the Engineer will confirm the requirements, if any, for subgrade undercutting. Cutting to subgrade can then take place.

2.5.7.3 Final subgrade assessment

The Contractor shall use appropriate plant and plan his work so that the condition of the subgrade is not compromised. After cutting to subgrade level, and immediately prior to any subgrade stabilisation, the CBR values of the exposed subgrade shall be established by similar tests at similar locations as prior to the first 1 m cut. The subgrade strength and any necessary subgrade treatment will then be agreed by the Contractor and the Engineer.

2.5.7.4 Protection of subgrade

The Contractor shall programme his work such that the subgrade is at all times protected from the effects of weather, construction plant or similar prior to any undercut treatment and/or placing or road pavement. If the surface of the subgrade becomes damaged or deteriorates prior to subgrade undercut treatment and/or pavement installation then the affected area should be scarified, reshaped, replaced, recompacted or otherwise treated and retested until the requirements for the subgrade are again obtained to the approval of the Engineer.

2.5.8 Undercut for shear keys, embankments and fill areas

Areas of expected undercut are shown on the Drawings and the Engineer may instruct additional or reduced areas of undercut as the works progress.

The work shall be carried out as separate operations to the cut-to-fill operations and shall be planned in conjunction with stream diversions, construction of culverts and other associated works as necessary. Sides of excavations shall be at slope angles compatible with slope heights to ensure stability and, if necessary, excavations shall be backfilled on the same day as excavation to ensure stability.

2.6 Subsoil drainage

Underdrainage (subsoil drainage) shall be installed as shown on the Drawings, or as instructed by the Engineer, and as specified in Specification Section 5.

The underdrains shall be located in the base of gulleys, and below sidling fills where cut grades exceed 1V : 4H. The outlets to all underdrains shall be carefully marked and protected as specified and their locations shown on the as-built drawings. The alignment, size and other details of all other drainage shall also be recorded on the as-built drawings.

2.7 Filling

2.7.1 General

Fill materials shall be sourced from areas of cut or from designated borrow areas. If the Contractor wishes to propose materials from alternative sources then he shall provide details of such sources for the Engineer's approval. Material types are required to be selected, handled and compacted to form zoned fills as shown on the Drawings or as instructed by the Engineer. The compaction standards are specified in Sections 2.8 and 2.9.

The Contractor shall take all precautions and maintain a tidy operation to minimize the presence of any loose, excavated materials that could become wet during rain. It is likely that considerable drying effort will be required for some of the materials if they become too wet to allow the required compaction criteria to be met. The Contractor shall also ensure that all fill is free of organic matter or other unsuitable materials.

2.7.2 Conditioning and spreading of fill

Before fill is placed in any area, the Contractor shall notify the Engineer that the fill foundation has been stripped, drained, including subsoil drains and prepared as required by the Drawings and Specification and is ready for the Engineer's inspection and approval.

Prior to compaction, the fill materials shall be spread uniformly in horizontal layers and, if necessary, conditioned to an appropriate water content by aeration and drying or wetting (as the case may be), and/or by blending and mixing "wet" and "dry" materials. When soil is to be dried, the Contractor shall disc the soil and allow it to dry uniformly to its full depth. When the soil is to be wetted, this shall be done with sprinkling equipment ensuring uniform and controlled distribution of water in conjunction with blading and discing. In all cases the fill shall be mixed and conditioned thoroughly so that immediately prior to compaction the material type and the water content of the fill is reasonably uniform within one area. The layers prior to compaction shall be less than 200 mm loose thickness with all fragments with less than 100 mm maximum dimension.

No new fill shall be placed over previously placed fill that has not achieved the required standard of compaction, or has become contaminated, or has deteriorated from the required fill standards, or requires testing and approval prior to placement of a new layer. Previously placed fill that does not comply shall be reworked by scarifying, conditioning and recompacting so as to meet the Specification or alternatively it shall be removed and replaced with complying material.

Positive and effective drainage shall be maintained during filling operations to minimize deterioration of material exposed in excavation areas and in the uppermost fill layers. Special care shall be taken to avoid hollows which could pond water. All fill surfaces shall be sealed off with rubber tyre plant when rain is imminent to minimise erosion and protect the fill from strength loss due to increase in moisture content.

Where fill is to be placed against sloping surfaces steeper than 1V: 2H, the sloping surface shall be excavated or "benched" such that horizontal benches at no greater than 0.5 m height intervals are formed. Adjacent areas of filling shall be carried out such that at no time shall fill levels be more than 0.5 m different between the adjacent areas, unless approved by the Engineer.

In order to ensure adequate compaction of the materials forming the final fill surface profile, all fill batter faces shall be overfilled as necessary and carefully trimmed back to the required design profile.

2.8 Compaction

2.8.1 General

The Contractor shall employ sufficient dedicated compaction plant so as to achieve the specified compaction. Equipment used in transportation and spreading will not be accepted as compaction plant. Compaction plant shall cover the entire area of each layer of fill and give each layer a uniform degree of compactive effort. The combined operations of spreading and compacting shall be undertaken using systematic and properly managed procedures, to the Engineer's approval, so as to ensure that each loose layer receives the required passes of the roller or other approved compaction equipment before further loose material is spread.

When soil is to be dried the Contractor shall disc the soil and allow it to dry uniformly to its full depth. When the soil is to be wetted, this shall be done with sprinkling equipment ensuring uniform and controlled distribution of water in conjunction with blading and discing.

Notwithstanding the requirements of Section 2.9 of this Specification, the Engineer may carry out check tests of compaction at any time. The Contractor shall stop or divert his machines as required by the Engineer to allow the tests to be carried out. Where field tests indicate that the specified standard of compaction has not been achieved, corrective action shall be taken to bring the fill to the required standard and as required by the Engineer. This may require the affected fill to be reworked by scarifying, conditioning and recompacting so as to meet the specification or alternatively it may need to be removed and replaced with complying material.

Competent and well-experienced Supervisors shall be provided by the Contractor to control procedures and shall carry out their duties primarily at the fill platform and not by delegation.

2.8.2 Compaction trials

Before placing any fill the Contractor shall carry out compaction trials to confirm to the Engineer the adequacy of the machinery and procedures which they propose to use for each of the fill types defined in Section 2.3 and in the Drawings. The trials shall be carried out using the same compaction equipment and on fill materials considered representative of those to be used for the permanent works. Separate trials will be necessary for each distinct fill material type and ongoing compaction trials may become necessary as the works proceed and/or where other fill types are accessed or identified. The Contractor shall keep the Engineer informed of fill material types being used or encountered during the course of the works and, depending on the materials encountered, the Engineer may instruct additional compaction trials be carried out.

Prior to commencing any compaction trials the programme and procedures shall be proposed by the Contractor for the Engineer's approval. The compaction trials shall comprise the spreading and compacting of a minimum of three superimposed 200 mm loose thickness layers of soil. The area of the compaction trial shall be sufficient to allow construction procedures and compactive effort to be representative of those proposed for the permanent works. For compaction trials of pumiceous fill the trial area should not be less than 25 m^2. Compaction standards of the trial areas shall be assessed using procedures and equipment as defined in Section 2.9 below.

Following completion of the compaction trials the procedures and construction plant shall be approved by the Designer and shall not be subsequently modified without the prior approval of the Designer.

The required standards of compaction shall be as defined in Section 2.9 for the various fill types. However during the compaction trials the Contractor may develop ad hoc tests which the Contractor may use as an approximate guide to the standard of compaction being achieved at any time, subject to their approval by the Designer.

2.8.3 Over-compaction of pumiceous fill

The Contractor shall manage the earthworks operations so as to minimise the potential for over compaction of the pumiceous fill. It is particularly noted that over-compaction of pumice materials can alter and degrade the geotechnical properties of the soil (including frictional properties, density and permeability), due to crushing of the pumice particles. The Contractor shall also attend to the water content of pumiceous structural fill to be placed within the works. This material may hold water, which when it is over-worked can acquire a slurry-like consistency.

Any areas of over-compacted structural fill identified by the Contractor shall be notified to the Engineer as soon as possible. The over-compacted material shall be undercut and removed from the works, and replaced by new structural fill at the direction of the Engineer.

2.9 Compaction standards and testing

2.9.1 General

The tests and testing frequency described and defined in Section 2.9 will be used to confirm that the placed fill materials meet the required Contract standards, design criteria and parameter values. At any time either prior to or during the course of construction, the Engineer may direct modifications to the compaction standards, frequencies and test methods defined in this Section with the object of ensuring that the design criteria and objectives for the particular materials and conditions encountered, are achieved.

Compaction and test requirements have been defined in this specification for the materials expected to be used for the construction of the works. Should alternative materials be proposed by the Contractor then they shall also propose appropriate Quality Control testing methods and procedures in order to demonstrate to the Engineer that the necessary design criteria can be achieved.

The tests to confirm the Contract requirements shall be undertaken using the fully specified methods set out herein, but the Engineer may approve more approximate and rapid methods on a day-to-day basis for preliminary assessment. Where an adequate correlation is established between the rapid and fully specified methods, the Engineer may rely on the results of the rapid methods. Where rapid methods are used, and there are discrepancies between the results obtained by rapid and fully specified methods the Engineer shall decide which tests apply. Results obtained by rapid methods shall not be used for final acceptance purposes unless approved by the Engineer.

The locations and levels of all in-situ tests shall be recorded within tolerances of 0.2 m horizontally and 0.1 m vertically.

All testing, both in-situ and laboratory, is to be carried out using an IANZ accredited testing organisation, with all equipment calibrated to relevant standards at the required frequency. Full details of the proposed testing organisation(s) shall be submitted to the Engineer for his approval

If the Engineer is satisfied that quality of materials is consistent and that the work is being carried out in a systematic and consistent manner, then may he instruct that the frequency of testing given in Table 2-5 can be reduced.

2.9.2 Fill test methods

The fill testing methods have been defined in Table 2.3 for the materials expected to be used for the construction of the works. Should alternative materials be proposed by the Contractor then he shall also propose appropriate quality control testing methods and procedures in order to demonstrate to the Engineer's approval that the necessary design criteria can be achieved.

Table 2.3: Fill test methods

Parameter	Test Description	Test Method
In-situ Density	"Rapid"	NZS 4407:1991, Test 4.2.1 (Nuclear Densometer Direct Mode) or NZS 4407:1991, Test 4.2.2 (Nuclear Densometer Backscatter Mode)
	"Fully Specified"	NZS 4402:1986, Test 5.1.1, 5.1.2, 5.1.3 (Sand replacement, balloon densometer or core cutter)
Maximum Dry Density & OMC determination	Standard Compaction	NZS 4402:1986, Test 4.1.1
	Heavy Compaction	NZS 4402:1986, Test 4.1.2
Strength	Scala Penetrometer	NZS 4402:1986, Test 6.5.2
	Pilcon Shear Vane	NZ Geotechnical Society Inc. "Guideline for hand held shear vane"
	Clegg Impact Test	ASTM D5874-95
Permeability	Laboratory Triaxial Permeability	Based on Head, Vol. 3, 1988, Section 20.4.2
Solid Density	Solid Density	NZS 4402:1986, Test 2.7.1
Moisture Content	Moisture Content	NZS 4402:1986, Test 2.1
Particle Size Distribution	PSD Wet Sieving	NZS 4402:1986, Test 2.8.1
	Hydrometer	NZS 4402:1986, Test 2.8.4

Note 1:

In the water content test the oven performance and forced ventilation requirements shall be waived provided that operating temperature range is verified and checked daily. Before the mass of a dried sample is accepted, it shall be dried for at least 14 hours, and be weighed at least twice at periods not less than four hours apart until the loss in mass between successive weighing is less than 0.1 grams per 100 grams.

Note 2:

In-situ Density: The air voids content of the compacted soil at any test location shall be taken as the mean of the air voids results from a set of density tests. A set of density tests shall comprise two or more individual tests made within an area of 0.5 m2.

Note 3:

The solid density test shall be performed using a sample at natural water content, not an oven-dried sample.

Note 4:

In-situ density tests may be replaced by the rapid method offered by a nuclear densometer (test method NZS4407 Test 4.2.1) provided that:

- appropriate tests are performed to correlate the bulk density obtained by the particular densometer being used to the bulk density obtained by the fully specified method;
- the correlation tests are performed initially at the rate of one (or more) correlation test for every 5 nuclear densometer tests until at least 10 satisfactory correlation tests have been performed (these 10 correlation test results can be obtained from previous work in the same material if the same nuclear densometer device is being used);
- subsequent correlations shall be performed at the rate of one (or more) correlation test for every 15 nuclear densometer tests;
- the correlation established by the 10 most recent relevant correlation tests shall be used to apply a correction to the air voids determined using the nuclear densometer;
- an air voids test result may be calculated immediately after testing, by assuming a value for the soil water content derived by applying a correction to the nuclear gauge water content, based on a correlation developed over 15 previous most recent relevant tests between the nuclear densometer water content and the oven water content – the water content correlation must be renewed each time the air voids correlation is renewed;
- a separate correlation shall be developed for each soil material type if this method is to be used.

Note 5:

Before a new shear vane is first used it should be calibrated to obtain values of torque versus spring deflection. It should be re calibrated at intervals of not more than 12 months.

2.9.3 Compaction standards

Fill materials shall be compacted so as to achieve the standards defined in Table 2-4 with the frequency of those tests as defined in Table 2-5 in Section 2.9.4. Note that not all of the tests listed in Table 2-4 are likely to be required for a particular project. Tests should be selected according to criteria such as design requirements, material (fill) performance requirements, ease of testing, availability of testing equipment/personnel etc.

Table 2-4: Compaction standards for fill

Material Type	**Parameter**	**Fill Type**		
		Landscape Fill	**Hardfill (GAP65)**	**Pumiceous Fill**
Cohesive	Average Minimum Shear Strength (1)	≥ 80 kPa	N/A	N/A
	Single Test Minimum Shear Strength (2)	> 70 kPa	N/A	N/A
	Average Maximum Air Voids (3)	≤ 10 %	N/A	N/A
	Single Test Maximum Air Voids	NA	N/A	N/A
Cohesionless	Maximum fines content (4)	N/A	15%	15%
	Minimum DCP	6 blow/300mm	10	10
	Single 50 mm interval minimum DCP	0.5 blow/50 mm	1 blow/50 mm	1 blow/50 mm
	In-situ Dry Density	> 75 % Solid Density	> 80 % Solid Density	> 80 % Solid Density
	In-situ Dry Density	> 90 % Maximum Dry Density	> 95 % Maximum Dry Density	> 95 % Maximum Dry Density

Note 1:

Corrected undrained shear strength determined by hand held Shear Vane. Average of 10 consecutive single readings within 1.0 m of each other.

Note 2:

Corrected undrained shear strength determined by hand held Shear Vane. Single reading. (Corrected means in accordance with the calibration of the instrument so that it represents the actual shear strength of the material and not just the dial reading of the instrument).

Note 3:

Average of 5 consecutive tests.

Note 4:

Percentage passing the 0.063 mm sieve

2.9.4 Frequency of testing

The frequency of testing shall be as described below in Table 2-5 and is the minimum considered acceptable. Additional tests and/or changes to the testing frequency may be instructed by the Engineer as the works proceed.

The Contractor shall control the earthworks operation so as to minimise the failure rate of any tests carried out as part of the Quality Control testing programme. Should any test result fail to meet the required design criteria the Contractor shall be required to propose remedial measures for the Engineer's approval. Such measures are expected to usually comprise the removal, replacement and satisfactory retesting of any fill within the agreed area of influence of the failed test location.

The Contractor shall rework and re-compact any area disturbed by any testing undertaken within the site, to the Engineer's approval.

Table 2-5: Minimum testing frequency

Fill Type	**Parameter**	**Test Type**	**Test Frequency**
Cohesive	Strength	Shear Vane	1 set per 200 m3 and 1 set per 0.3 m lift
	In-situ Density	"Rapid" with Moisture Content	1 set per 250 m3 and 1 set per 0.3 m lift and 1 set for each material type
Cohesionless	Strength	Scala Penetrometer	1 set per 100 m3 and 1 set per 0.3 m lift
	Particle Size Distribution	Wet sieving	1 initial test for each material type and then 1 test per 1,000 m3 for that particular material type
	In-situ Density	"Rapid" with Moisture Content	1 set per 250 m3 and 1 set per 0.3 m lift and 1 set for each material type
All	Maximum Dry Density, Solid Density and OMC	"Heavy" Compaction	1 initial test for each material type and then 1 test per 1,000 m3 for that particular material type

Note 1:

When In-Situ Density "Rapid" tests are carried out a set shall comprise 2 measurements using the same probe hole but oriented at 90° to each other.

Note 2:

The Contractor shall make every effort to ensure an even spread of test locations, both vertically and horizontally, through all fill areas. Spatial separation of tests within the completed fill areas shall be such that at least one set of tests is completed within any given continuous 0.3m thickness of fill. For the purposes of this clause a "fill area" is defined as the area or zone of continuous fill placed on a particular working day.

3 Cement Stabilisation Ground Improvement

3.1 Scope

The work specified in this Section covers the requirements for the supply of materials, construction and Quality Control testing of in-situ mixing ground improvement. The works shall be of the form and dimension shown on the Drawings, as specified in these Specifications or as instructed by the Engineer.

The main activities associated with the cement stabilisation construction include:

1. cement-stabilisation of natural ground to a defined depth (as shown on the Drawings) using in-situ mixing
2. verification testing and QA documentation
3. any necessary groundwater control and temporary support of excavations.

The depth of cement stabilised material and any overlying re-compacted soil fill material shall be as shown on the Drawings.

Construction shall be undertaken such that a homogeneous layer of stabilised materials is attained and no significant abrupt change in the in-situ density and/or stiffness is present within the as-built stabilised material. Care shall be taken to knit and overlap joints between panels or cells of in-situ mixed material which are constructed at different times to produce a homogeneous cement stabilised layer. Where it is necessary to construct the stabilised layers in separate sections, strips or panels, additional care is required at the vertical edge joins by cutting into the previously compacted zone to produce a homogeneous layer without compromising the compaction integrity across the joins.

The Contractor shall review the existing available geotechnical data and, undertake all necessary pre-construction inspections to assess the suitability of their construction methodology and plant to each material type to be utilised in the Works prior to the commencement of construction. The density and strength of the as-built cement stabilised ground improvement shall be verified via in-situ testing as specified in Section 3.8 of this Specification.

The cement stabilisation works will be undertaken adjoining a lakeshore area with the base of the cement mixing below the normal lake water level. The Contractor shall be responsible for all environmental controls to prevent contamination of the lake during construction works. It is anticipated that the Contractor will install some form of coffer dam arrangement to isolate the lake from the works area and utilise dewatering and water treatment measures as part of their environmental controls.

3.2 Definitions

The in-situ method involves mixing of cement into the undisturbed ground by way of a cement injector and some form of mixing paddles or rotovated to the full depth of the stabilised layer.

3.3 Set out and Tolerances

The cement stabilised material shall be placed so as to meet the following construction tolerances:

- Horizontal location: +100mm / - 0 mm
- Vertical extent (depth): +100mm / - 0 mm

3.4 Inspection, Review and Approval Hold Points

The Contractor's programme shall allow for inspection, review and approval hold points as detailed in Table 3.1 below and as described elsewhere in this Specification.

Table 3.1: Cement Stabilisation Inspection and Hold Points

HOLD POINT			
No.	**Name**	**Description**	**Responsibility**
1	Contractor's Laboratory testing methodology	The Contractor shall submit a detailed laboratory testing methodology including their proposed cement and additive mix, and proposed testing plan.	Engineer to review
2	Contractor's Field Trial methodology	The Contractor shall submit a detailed Field Testing plan detailing proposed mixing process and testing methodology. This should also include proposed environmental controls.	Engineer to review
3	Contractor's Anchor Test methodology	The Contractor shall submit a detailed methodology of their proposed ground anchor / micro pile testing within the cured Cement Stabilised Field Trial.	Engineer to review
4	Contractor's Methodology	The Contractor shall submit a detailed Construction Methodology to the Engineer, prior to mobilising any specialist machinery to the Site. The methodology shall provide a detailed report of the results of the field trial and proposed their cement dosing rate and details of any additional additives. This should also include proposed environmental controls.	Engineer to review
5	Cement Stabilisation Construction General	The construction of the cement stabilisation shall be observed by the Engineer on a regular basis during the construction period to check compliance with the Specification and Drawings. Such observations should generally be done together with the Contractor.	Engineer to observe
6	Quality Assurance Tests	The Contractor shall submit for the ongoing review of the Engineer, copies of all imported fill source and Site suitability laboratory testing and on-going in-situ field test results.	Engineer to review
7	Completion of Cement Stabilisation	The surface of the completed cement stabilised area shall be inspected and approved by the Engineer after completion.	Engineer to inspect and approve

3.5 Cement dosage

3.5.1 General

The Contractor shall use a Sulphate Resistant cement and include other additives such as Fly Ash or Microsilica to provide sulphate resistance to the stabilised material complying with NZS 3122. Cement shall be stored under cover and protected from dampness until used. Any cement containing lumps shall not be used in the Works.

The Contractor shall determine the cement dosage and requirements for other additives from their Laboratory Testing and Field Trial programme to achieve the following standards:

Table 3.2: Cement Stabilised Ground Improvements - Minimum requirements

Parameter	Requirement	Test Method
Unconfined Compressive Strength	Average > 2 MPa No test lower than 1.8 MPa	NZS 4402:Test 6.3.1
Shear Strength	Average > 500 kPa No test lower than 300 kPa	BS1377 Part 7 Test 9 (a)
Bulk Density	Greater than 18 kN/m^3	NZS4402 Test 5.1.4

The Contractor shall decide on the percentage of cement to be added in order to meet the minimum test criteria required by this specification. The dosage rates may need to be adjusted by the contractor during construction in order to achieve the required strength criteria. Any costs associated with rework or increase in the cement dosage rate, to achieve minimum test criteria, shall be borne by the Contractor.

3.5.2 Laboratory testing

The Contractor shall undertake the material reference and suitability tests listed below and shall submit the required tests for acceptance by the Engineer prior to the start of the ground improvement construction.

3.5.3 Field Trial

The Contractor shall undertake a field trial to verify the cement dosage and additives in their proposed cement mix design can achieve the requirements in Table 3.2.

The field trial shall comprise a minimum of 3 test cells each of a minimum dimension of 4.5 m wide by 4.5 m long by 3.5 m deep. Each test cell area shall have a different cement dosage and shall be undertaken to verify the minimum properties can be achieved, and to look to optimise the cement dosage percentage.

Verification testing shall be undertaken within each test cell. The verification testing of each test cell shall comprise as a minimum the following:

- 2 number continuously cored boreholes drilled from the surface to a depth of 4 m. One borehole shall be drilled at 7 days following mixing. The second borehole shall be drilled 28 days following mixing.
- The borehole shall be a minimum of 100 mm diameter to facilitate laboratory testing of recovered samples.
- The temperature shall be recorded in the base of each borehole.
- The boreholes shall be spaced a minimum of 2.5 m apart.
- The boreholes shall be reinstated with 20 MPa micro concrete.
- The recovered core not used in testing shall be stored on site to facilitate inspection by the Engineer.
- Additional boreholes and samples recovery shall be undertaken upon the instruction of the Engineer.

Laboratory testing of the recovered samples shall be undertaken. The laboratory testing shall comprise the following

Table 3-3: Minimum stabilised material field trial testing schedule

Parameter	Test Description	Test Method	Minimum Test Frequency
Strength – Laboratory Testing	CBR	RUU TR7 Test 8 in 100mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	3 samples per cell(1)
	UCS	NZS 4402:Test 6.3.1 in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	3 samples per cell(1)
	Triaxial	BS1377: Part 7 Test 9 (a) in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	1 sample per cell(1). Additional samples may be required at the discretion of the Engineer
In-situ Sampling and Laboratory Testing	Borehole samples	100mm continuously borehole for full depth of the stabilised material. Samples stored for laboratory testing	2 boreholes per cell
	UCS	NZS 4402:Test 6.3.1 in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	2 tests per borehole
	Triaxial	BS1377: Part 7 Test 9 (a) in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	2 tests per borehole
	Temperature	To be agreed with Engineer	1 test in base of borehole
	Bulk Density	NZS 4402 Test 5.1.4	2 tests per borehole

Notes:

1 Samples to be taken from the treated material and placed into 4 x 100 mm diameter test cylinders within 1 hour of cement mixing. One test cylinder is to be selected for initial testing. The remaining cylinders shall be used for additional testing after additional curing if the first test fails the minimum strength criteria.

Ground anchors will be tested to failure (proof load) within the field trial cells once they have cured. The test cells shall not be undertaken in an area where the boardwalk is constructed over water and cannot be incorporated into the permanent works. The field trials could be undertaken over areas where retaining walls or boardwalk over land will be constructed. Alternatively, the test cells could be undertaken on the landward section of the 3.0m boardwalk.

3.6 Soil-cement blending

The Contractor shall treat all soil material which lies within the ground improvement zone with an appropriate dose of cement and shall thoroughly mix the cement into the soil so as to achieve the minimum density and strength testing criteria. Thorough mixing of cement into the soil is a key aspect in achieving the minimum strengths requirements. The Contractor shall detail in their Construction Management Plan the machinery and equipment that will be used and the QA procedure that will be implemented in order to provide assurance that the cement is thoroughly mixed into the soil. This shall include details of how the location of each panel or area that is stabilised will be surveyed and sequencing of panel construction in order avoid disturbing recently stabilised material and ensuring sufficient overlap and interlocking of panel joints.

The Engineer may request that inspection test pits are undertaken within the soil-cement stabilised material to verify that panel joints are sufficiently overlapped and that there is sufficient mixing of cement, to provide a homogeneous layer.

3.7 QA records

During mixing the following records shall be kept for each day's projection, or part thereof and submitted to the Engineer the day following work:

1. Date and weather conditions
2. Details of the time cement was added to the soil and when mixing was completed
3. Cement-stabilised soil panel / cell number, including location and depth
4. Total volume and /or mass of cement added to the above total mass of soil for each panel/cell.

Records of testing locations and results shall be submitted to the Engineer one day following undertaking in situ testing or receiving laboratory results.

It is expected that the Contractor will use GPS positioning controls to record the location (horizontally and vertically) and cement dosage to ensure accuracy and consistency of the treated materials in the defined stabilised cell area.

3.8 Permanent works testing requirements

3.8.1 Verification test methods and frequency

The type and frequency of testing which is to be completed by the Contractor is listed below in Table 3-4. Additional tests and /or changes to the testing frequency may be instructed by the Engineer as the Works proceed.

Test methods which leave a hole size greater than 25 mm shall be backfilled with cement grout.

Table 3-4: Minimum stabilised material verification testing schedule

Parameter	Test Description	Test Method	Minimum Test Frequency
Strength – Laboratory Testing	CBR	RUU TR7 Test 8 in 100mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	1 sample per 100m^3 material treated(1)
	UCS	NZS 4402:Test 6.3.1 in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	1 sample per 100 m^3 material treated(1)
	Triaxial	BS1377: Part 7 Test 9 (a) in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	1 sample only(1). Additional samples may be required at the discretion of the Engineer
	Bulk density	NZS 4402 Test 5.1.4	1 sample per 100 m^3 material treated(1)
In-situ Sampling and Laboratory Testing	Borehole samples	100mm continuously borehole for full depth of the stabilised material. Samples stored for laboratory testing	1 borehole per 500 m^3
	UCS	NZS 4402:Test 6.3.1 in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	2 per borehole
	Triaxial	BS1377: Part 7 Test 9 (a) in 100 mm diameter test cylinders cured for 7 days (increased to 28 days at the discretion of the Engineer)	2 per borehole
	Temperature	To be agreed with Engineer	1 test in base of borehole
	Bulk Density	NZS 4402 Test 5.1.4	2 per borehole

Notes:

1 Samples to be taken from the treated material and placed into 4 x 100 mm diameter test cylinders within 1 hour of cement mixing. One test cylinder is to be selected for initial testing. The remaining cylinders shall be used for additional testing after additional curing if the first test fails the minimum strength criteria.

3.8.2 Verification Test Criteria

All cement stabilised material shall meet the minimum criteria stipulated in Table 3.2. The minimum strength requirement is expected to be able to be achieved within seven days of placement. Where this minimum strength is not achieved within seven days the Contractor may choose to rework the material or to wait for additional strength gain to be obtained and to then retest the material.

Where the minimum strength has not been achieved within 28 days, the Contractor shall remediate the non-confirming material. Any such delays or rework shall be at the Contractor's expense.

4 Ground Anchors – Bars

4.1 Scope

The work specified in this Section covers the requirements for the supply of materials, construction and Quality Control testing of bar ground anchors for use as micro piles. The works shall be of the form and dimension shown on the Drawings, as specified in these Specifications or as instructed by the Engineer.

4.2 Standard specifications

Construction work performed under this Section shall comply with:

- The general requirements of the latest revisions of the following documents:
  - BS 8081 : 1989 Code of Practice for Ground Anchorages
  - BS EN 1537 : 2000 Execution of special geotechnical work - ground anchors
  - BS 5896 : 1980 Specification for high tensile steel wire strand for the prestressing of concrete
  - NZS 3109 : 1997 Concrete construction
  - NZS 3112 : 1986 Testing Concrete
- The general requirements of the latest revisions of all other Standards, Specifications and Codes of Practice referenced in these Contract Documents
- The specific requirements of this Section and the Drawings.

This Specification shall be read in conjunction with the above referred Standards and documents, which are deemed to form a part of this Specification. In the event of any requirements of this Specification being at variance with any of the above Standards and documents then the requirements of this Specification take precedence.

4.3 Design of ground anchors

The ground anchors shall be single-bar anchors with permanent single-corrosion protection complying with Clause 4.5 of this Specification. That standard of corrosion protection applies to all parts of the ground anchor, including the bond length, free length and the anchor headworks.

The anchors will be "Type A" straight shaft gravity grouted anchors, as defined by BS 8081 : 1989 except where specified otherwise or if instructed by the Engineer.

The required anchor capacities and dimensions are specified on the Drawings and summarised in Table 4-1 below, although others may be instructed by the Engineer as the works proceed.

Table 4-1: Anchor Capacities and Dimensions

Parameter	Anchor Working Load
	30 kN
Prestress Anchor Load (kN)	0
Nominal Length (m)	3
Bond Length (m)	3
Anchor Diameter (mm)	150
Horizontal Spacing (m)	1.0m or as shown on the drawings

4.4 Contractors Method Statement

Prior to the commencement of any anchor works the Contractor shall provide a Method Statement for the Engineer's approval, setting out the Contractor's proposed fabrication, installation and testing procedures so as to construct anchors able to satisfy the minimum design criteria contained in the Drawings and Specification. The Method Statement shall include as a minimum:

- Description of proposed installation procedures, materials and equipment
  - Method and details of proposed grouting procedures
  - Grout and concrete details
  - Proposed drilling method and equipment, including anticipated casing requirements
  - Proposed installation method
  - Details of supporting QA documentation as necessary
  - Proposed testing method and equipment, including methods for measuring load and deformation. Valid calibration certificates shall be provided for pressure gauges, dial gauges and the like
- Details of reporting format for drilling, installation, testing
- Details of stressing and testing procedures
- Details of health and safety procedures over and above those generally required for the works and to meet statutory requirements

The Engineer will review the Method Statement within 5 Working Days of its receipt. The Engineer will then discuss the Method Statement with the Contractor and advise of any amendments which the Engineer considers are necessary in order to ensure the Method Statement complies with the specified requirements. The revised Method Statement shall then be submitted to the Engineer for approval.

4.5 Materials

4.5.1 Anchor components

4.5.1.1 Bars

Anchor bar shall be Grade 500 ReidBar, complying to the requirements of BS 8081 and BS 5896. Mill certificates and other supporting documentation shall be submitted to the Engineer for each batch of bar showing the ultimate load, the yield, the percentage elongation at yield load and the modulus of elasticity. The bar shall be hot dip galvanised with a minimum coating application thickness of 600 gm/m2.

4.5.1.2 Other

Other components of the anchor fabrication shall comply with BS 8081 and other relevant standards. Any HDPE or PVC tubing or "Densotape" or similar approved fitted over the bar free length shall be such as to allow elongation during stressing. Materials for accessories such as end caps, grouting tubes etc shall be equivalent to the plastic tubing.

All materials used to fabricate the anchor shall be free of water soluble chlorides and other ingredients which might cause corrosion, hydrogen embrittlement or stress corrosion of the bar. All plastics which will form a permanent part of the anchor shall be non-reactive with grout, concrete and their ingredients.

4.5.2 Grout

Anchor grout shall comprise fresh Portland Cement complying with NZS3122 : 1995, water and approved additives. The proportions of cement, water and additives shall conform to NZS 3109 to produce a cement-rich grout having a standard-cured compressive strength of not less than 20 MPa at 7 days and 35 MPa at 28 days when tested in accordance with NZS 3112. Sand or other materials shall not be used unless approved by the Engineer. The water/cement ratio shall be not less than 0.4.

4.5.3 Anchor headworks

Materials for the anchor headworks shall be as specified on the Drawings or as specified in the approved Contractor's Method Statement.

4.5.4 Corrosion protection

All parts of the ground anchor, including the bond length, free length and the anchor headworks shall have a permanent single protection standard of corrosion protection, as defined in BS8081:1989, developed to provide 50 years design life and as specified on the Drawings or the approved Contractor's Method Statement.

4.6 Construction

4.6.1 Drilling

The Contractor shall use plant, equipment and techniques appropriate for the terrain, access requirements, and ground and groundwater conditions on the site. The Contractor shall be responsible for employing methods and equipment required to adequately support, (including using temporary casing if necessary), the drill hole during the construction and installation of the anchor and to prevent excess air, drill fluid or grout loss. The drilling techniques and equipment used shall be in accordance with the approved Contractor's Method Statement.

Completed drillholes shall be not less than 150 mm diameter over their full length. Drilling fluids other than fresh water or air shall not be used at any time during the drilling of ground anchor holes unless approved by the Engineer. Under no circumstances is the Contractor to use a polymer drilling fluid in combination with bentonite. The Contractor shall use appropriate shields to reduce noise and to control dust to acceptable levels, and shall fully comply with all environmental requirements with respect to the containment and disposal of drilling flush and cuttings.

Drillholes shall be carried out at the locations shown on the Drawings or as instructed by the Engineer. The drillholes shall be within 2° of their specified alignment over their full length.

The Contractor shall provide an individual drilling log for each anchor drill hole, which shall be provided to the Engineer within 1 Working Day of completion of drilling of that hole. The information provided shall be as listed in the Contractor's Method Statement but as a minimum shall not be less than specified in BS 8081.

4.6.2 Installation

Not more than 2 hours prior to anchor installation the drillhole shall be cleaned out with air or water. Installation of an anchor will not be permitted until the relevant drillhole log has been approved by the Engineer. Anchors shall be carefully installed in accordance with the fabricator's recommendations and the Contractor's Method Statement so as to ensure that the fixed length is located within the specified anchorage zone. All equipment for handling and installing the anchor

shall be such that it will not damage the anchor bars or its corrosion protection. Prior to installation all grout tubes shall be checked with water or compressed air to ensure that they are clear.

Centralisers shall be provided as necessary in order to ensure that the installed anchor is located centrally within the drillhole.

The Contractor shall keep individual anchor records detailing the date and time of installation and any difficulties or problems that were encountered.

4.6.3 Grouting

The grouting equipment shall be capable of continuous mechanical mixing to produce a homogenous grout free of lumps and undispersed cement. The pumping system shall have a series of valves and calibrated pressure gauges so as to permit continuous circulation and pumping of grout with an accuracy of +/- 0.1 MPa grout pressure.

The grouting procedures shall be as per the Contractor's approved Method Statement and shall conform to the requirements of BS 8081, but as a minimum shall include the following provisions:

- The primary grout shall be pumped into the anchor drillholes through a grout pipe provided for that purpose until the grout flows from the primary grout vent.
- The grout shall always be injected at the lowest point on the fixed length. The level of the top of the primary grout shall be checked in order to ensure that the grout completely fills the drillhole over the fixed bond length.
- The free stressing length shall be flushed out through specially provided flushing tubes to remove any excess grout above the bond length.
- After grouting the anchors shall remain in an undisturbed condition until the specified grout strengths have been achieved.
- The free lengths of the anchors shall be grouted after final stressing.
- Grout samples shall be taken and tested at an approved laboratory. The Contractor shall propose the extent of such testing in the Contractor's Method Statement with a minimum requirement of:
  - Two sets of 4 test samples each day that grouting is proposed, with not less than 2 sets of 4 test samples for the first batch of grout mixed and used; and
  - Not less than 1 set of 4 test samples for each additional batch of grout mixed
- Required (primary) grout volumes shall be calculated for each ground anchor and correlated against grout volumes achieved for each anchor together with any known reasons for disparity.
- If the grout volume is significantly less or greater than the calculated nominal volume required, the Contractor shall notify the Engineer immediately together with proposed remedial actions.
- The Contractor shall keep records of all grouting works, which shall be provided to the Engineer within 1 Working Day of completion of each and every any stage of grouting. The information provided shall be as listed in the Contractor's Method Statement but as a minimum shall not be less than specified in BS 8081.

4.6.4 Anchor headworks

The anchor headworks shall be constructed as specified on the Drawings or as specified in the approved Contractor's Method Statement.

4.7 Testing

4.7.1 General

Two types of anchor tests are required as outlined below:

a Proof Load Test

b Acceptance Load Tests

The individual test requirements are set out in the following Sections:

4.7.2 Records

The Contractor shall keep records of all anchor stressing and testing. The stressing records and test results for any particular anchor shall be provided to the Engineer within 5 Working Days of completion of stressing and testing for that anchor. The information provided shall be as listed in the Contractor's Method Statement but as a minimum shall not be less than specified in BS 8081.

4.7.3 Proof load tests

4.7.3.1 Objectives

The objectives of the Proof Load Tests are to prove the anchor system meets design criteria with respect to load and deformation behaviour. The tests shall be carried out on 4 No. anchors installed in each of the three trial cells specified in Section 3.5.3.

4.7.3.2 Procedures

The anchors are to be stressed over three loading cycles to the percentage of Working Load specified in Table 4-2 below:

Table 4-2: Proof Load Test Loading Cycles

Load Increment (% of Working Load)			**Minimum Period of Observation (minutes)**
First Load Cycle	**Second Load Cycle**	**Third Load Cycle**	
10	10	10	1
50	50	50	1
100	100	100	1
125	125	150	15
100	100	100	1
50	50	50	1
10	10	10	1

Notes:

a) The initial load (10 % of Working Load) shall be applied to "bed in" the anchor and testing system and allow the jack to stabilise. Once the jack has stabilised, all displacements and load values shall be recorded and the specified loading and unloading increments applied.

b) The applied load shall be held steady at each increment/decrement with a variation of no more than 2 kN from the specified load.

c) Displacements at load increments other than 125% and 150% of Working Load shall be recorded initially on reaching that load and then after 1 minute, immediately prior to changing to the next load increment.

d) Displacements at 125% and 150% of Working Load shall be recorded initially on reaching that load and then at 5 minute intervals.

4.7.3.3 Proof load test acceptance criteria

Failure is deemed to have occurred if the displacement of the anchor head relative to the measuring datum exceeds the following criteria for 125% or 150% Working Load, as specified in Table 4-3 below:

Table 4-3: Proof Load Test Acceptance Criteria

Period of Observation (minutes)	**Maximum Permissible Displacement (% of elastic extension of anchor bars at 125 % and 150 % of Working Load)**
5	1
15	2

4.7.4 Acceptance load tests

4.7.4.1 Objectives

The objectives of the Acceptance Load Tests are to take any "slack" out of the installed anchor system and to demonstrate that installed anchors are able to provide at least the design working load capacity. The tests shall be carried out on a minimum of 50% of the installed anchors.

4.7.4.2 Procedures

The anchors are to be stressed over two loading cycles to the percentage of Working Load specified in Table 4-4 below:

Table 4-4: Acceptance Load Test Loading Cycles

Load Increment (% of Working Load)		**Minimum Period of Observation (minutes)**
First Load Cycle	**Second Load Cycle**	
10	10	1
50	50	1
100	100	50
50	55	1
10	10	1

Notes:

1 The initial load (10 % of Working Load) shall be applied to "bed in" the anchor and testing system and allow the jack to stabilise. Once the jack has stabilised then all displacements and load values shall be recorded and the specified loading and unloading increments applied.

2 The applied load shall be held steady at each increment/decrement with a variation of no more than 2 kN from the specified load.

3 Displacements at load increments other than 100% of Working Load shall be recorded initially on reaching that load and then after 1 minute, immediately prior to changing to the next load increment.

4 Displacements at 100% of Working Load shall be recorded initially on reaching that load and then at 5 minute intervals.

4.7.4.3 Acceptance load test acceptance criteria

Failure is deemed to have occurred if the displacement of the anchor head relative to the measuring datum exceeds the criteria given in Table 4-5 for 100% Working Load:

Table 4-5: Acceptance Load Test Acceptance Criteria

Period of Observation (minutes)	Maximum Permissible Displacement (% of elastic extension of anchor at 100% of Working Load)
5	1
20	2

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5 Pipes and Associated Works – Gravity Systems

5.1 Scope

This section details the requirements for pipe and associated works for gravity systems, the extent of which is shown on the Drawings.

Requirements for trenching, bedding and backfilling are given in the specification for trenching, bedding and backfilling for underground services.

5.2 Compliance

Construction work performed under this Section shall comply with the general requirements of the following documents and the specific requirements of this Section:

AS 1463 - Polyethylene pipe extrusion compounds

AS 1650 - Hot-dipped galvanized coatings on ferrous articles

AS 1741 - Vitrified clay pipes and fittings with flexible joints - Sewer quality

AS 2032 - Installation of uPVC Pipe Systems

AS 2129 - Flanges for pipes, valves and fittings

AS 4087 - Metallic flanges for waterworks purposes

AS/NZS 1254 - PVC pipes and fittings for stormwater and surface water applications

AS/NZS 1260 - PVC-U pipes and fittings for drain, waste and vent application

AS/NZS 2033 - Installation of polyethylene pipe systems

AS/NZS 2280 - Ductile Iron Pressure Pipes and Fittings

AS/NZS 2566 - Buried flexible pipelines

AS/NZS 4058 - precast concrete pipes (pressure and non-pressure)

AS/NZS 4130 - Polyethylene pipes for pressure applications

AS/NZS 4131 - Polyethylene (PE) compounds for pressure pipes and fittings

AS/NZS 5065 - Polyethylene (PE) and polypropylene pipes and fittings for drainage and sewerage applications

BS EN 295 - Vitrified clay pipes and fittings and pipe joints for drains and sewers

NZS 3109 - Concrete Construction

NZS 3114 - Specification for concrete surface finishes

NZS 3302 - Specification for ceramic pipes, fittings and joints

NZS 4452 - Code of practice for the construction of underground pipe sewers and drains

NZS 7643 - Code of practice for the installation of unplasticised pvc pipe systems

SNZ NZS/BS 2494 - Elastomeric seals for Joints in Pipework and Pipelines

- Local Authority Standards

5.3 Materials

5.3.1 General

Materials shall be stored, handled, and distributed on Site with care to avoid damage and in accordance with the manufacturer's recommendations. All materials to be used in the works shall be in new condition.

5.3.2 Concrete pipes

Concrete drainage pipes shall be manufactured in accordance with the requirements of AS/NZS 4058. The pipe class shall be as shown on the drawings.

5.3.3 Vitrified clay pipes and fittings

Vitrified clay pipes and fittings shall be rubber ring jointed the drawings and comply with NZS 3302.

5.3.4 PE pipes and fittings

PE pipes and fittings shall comply with AS/NZS 5065, AS/NZS 4129 and AS/NZS 4130 and shall be manufactured from PE 80B or PE 100 compound. Pipe SDR ratings shall be as shown on the Drawings.

PE pipes and fittings greater than 100 mm diameter shall be joined by fusion butt welding, electrofusion couplings or stub flange plus steel ring connections unless otherwise specified or shown on the Drawings. Pipes and fittings less than 100 mm diameter may be joined with mechanical fittings approved by the Engineer.

Fittings shall have a wall thickness not less than the pipe wall thickness. Fittings with a thicker wall require the approval of the Engineer.

Bend and junction dimensions shall comply with the pipe manufacturer's requirements. All PE pipes and fittings shall be UV protected for above ground installations. Long radius bends in the pipe may be made on site at a radius greater than the manufacturer's minimum allowable radius.

5.3.5 uPVC pipes and fittings

uPVC pipes and fittings shall comply with AS/NZS 1260. Connections shall be rubber ring jointed or solvent welded as shown on the Drawings.

5.3.6 Elastomeric rings

Elastomeric rings for pipe joints shall be manufactured in accordance with NZS/BS 2494. Rings shall be stored away from direct sunlight and shall be kept free of grease, oil, paint and other substances deleterious to rubber. Rings showing any signs of damage or fault shall be removed from the Site and replaced at the Contractor's expense.

5.3.7 Fittings

Fittings shall be to the standards specified on the Drawings and in this Specification. Cast iron fittings and specials shall be manufactured from high quality grey iron and coated with a proven bituminous compound. When no standard is specified on the Drawings, fittings shall be to the Local Authority standards. All bends and fittings shall be suitable for jointing with gibault and rubber ring joints unless otherwise indicated on the Drawings.

Fittings such as gibault joints, tapping straps and saddles not covered by standard specifications shall be approved types. Bolts used in these fittings shall be hot dip galvanised to AS 1650 after

threading. Nuts shall be tapped after galvanising up to 0.4 mm oversize to enable them to mate with the bolts and the threads shall be oiled. The bolts shall be liberally coated with an approved anti-seize lubricant prior to assembly. Following assembly of the unit the nut and exposed thread on the end of the bolt shall be securely wrapped with at least two layers of Denso tape to form an impermeable cover.

Grade 316 stainless steel bolts and nuts can be used as an alternative to galvanized fasteners unless stated otherwise on the Drawings or by the Engineer. Neoprene washers shall be used to prevent the nuts and bolts making contact with other metals.

5.3.8 Manholes and catchpits

5.3.8.1 General

The requirements for manholes and catchpits are shown generally on the Drawings. Where particular items are not detailed they shall be to the Local Authority standards.

Manholes including covers and frames shall be constructed in accordance with the Drawings and the standards and drawings of the relevant Local Authority.

The manholes shall be watertight from the ground surface to the bottom of the manhole when the inlet and outlet pipes are plugged.

Manhole risers shall comply with AS/NZS 4058.

Manhole cover frames shall be haunched with 30 MPa concrete and either plastered or trowelled off to a smooth surface. All manhole covers shall be set flush with and have the same cross-fall as the finished surfaces of the road, footway or ground, unless detailed otherwise. In sloping ground, the manhole access and cover shall be on the lower side, unless detailed otherwise.

At pipe connections to manholes, two flexible joints shall be provided in the pipe at a distance not greater than one metre from the wall of the manhole, unless detailed otherwise on the Drawings or in Local Authority standards.

5.3.8.2 Manhole foundation

The manhole structure shall be constructed or placed on a levelling course placed on top of the subgrade material.

The suitability of the subgrade material shall be confirmed by the Engineer Where the material is not suitable for the manhole foundation, the Contractor shall upon receipt of an order from the Engineer over-excavate the material as necessary and backfill with an approved compacted granular bedding material.

The levelling course shall comprise a 100 mm minimum layer of compacted approved AP20 granular material or concrete.

5.3.8.3 Testing

The Engineer may require a watertightness test to be carried out at manholes.

The watertightness test shall be carried out after pre-soaking the manhole filled with water for one hour. The watertightness test shall comprise the filling of the manhole to its lid level after plugging manhole pipe connections and checking the water volume drop in the manhole for a period of one hour. The manhole shall be considered watertight if the drop does not exceed 0.2 litre per square metre of internal cross-section area per metre depth of the manhole.

5.3.9 Catchpits

5.3.9.1 General

Catchpits including surface componentry shall be constructed in accordance with the Drawings and the standards and drawings of the relevant Local Authority.

The catchpits shall be watertight from the ground surface to the bottom of the catchpit when the outlet pipe is plugged.

Precast manufactured catchpits shall comply with NZS 3109. Surface finishes shall comply with NZS 3114 and shall be F4 and U2 for formed and trowelled surfaces respectively.

At pipe connections to catchpits, two flexible joints shall be provided in the pipe at a distance not greater than one metre from the wall of the catchpit, unless detailed otherwise on the Drawings or in Local Authority standards.

Unless otherwise specified or shown on the Drawings and Local Authority standards, the invert of the catchpit lead outlet shall be 450 mm minimum above the invert of the catchpit.

5.3.9.2 Catchpit foundation

The catchpit shall be constructed or placed on a levelling course placed on top of the subgrade material.

The suitability of the subgrade material shall be confirmed by the Engineer. Where the material is not suitable for the catchpit foundation, the Contractor shall upon receipt of an order from the Engineer over-excavate the material as necessary and backfill with an approved compacted granular bedding material.

The levelling course shall comprise a 100 mm minimum layer of compacted approved AP20 granular material or concrete.

5.3.9.3 Testing

The Engineer may require a watertightness test to be carried out at catchpits.

The watertightness test shall be carried out after pre-soaking the catchpit filled with water for one hour. The watertightness test shall comprise the filling of the catchpit to its lid level after plugging catchpit pipe connections and checking the water volume drop in the catchpit for a period of one hour. The catchpit shall be considered watertight if the drop does not exceed 0.2 litre per square metre of internal cross-section area of the catchpit.

5.4 Construction

5.4.1 Setting out

The Contractor shall provide all equipment necessary to accurately set out pipelines in both the horizontal and vertical planes. The methods that are proposed to be used shall be submitted to the Engineer for approval prior to the commencement of the works.

5.4.2 Transport, handling and storage of materials

The Contractor shall be responsible for the transport (unless noted otherwise), handling, storage and security of all pipe and fittings which shall be handled and stacked in accordance with the manufacturer's recommendations. uPVC pipe shall be handled and stored strictly in accordance with the requirements of NZS 7643.

Materials will be inspected by the Engineer at his discretion upon delivery to Site or as soon thereafter as practicable. The Contractor shall provide every assistance, including lifting and rotating pipes, to enable these inspections to be undertaken efficiently.

Damaged or unsatisfactory materials noted at that time will be marked and the Contractor shall either replace the item or if the Engineer permits, repair the defect in an approved manner. The costs of repair or replacement of damaged, unsatisfactory or repaired materials used or unused in the works shall be borne by the Contractor, whether the Engineer has inspected and approved the materials or not.

5.4.3 Pipe laying and jointing

5.4.3.1 General

Pipes and fittings shall be thoroughly cleaned before lowering into the trench, and shall be kept clean throughout the jointing and testing procedures. Whenever work is discontinued or whenever there is any likelihood of entry of foreign matter, the open ends of the laid pipes shall be closed with suitable caps. If the excavations are likely to be flooded by stormwater, these end caps shall be watertight and effective precautions taken to prevent the pipeline from floating.

The jointing and laying of all pipes shall be carried out in accordance with the manufacturer's written instructions.

Notwithstanding any tolerance given in this Section pipes shall be laid so that deviations at joints do not exceed 66% of the manufacturer's recommended maximum deviations.

All pipes and fittings shall be thoroughly cleaned before placing, and all scale, burrs, sand, slag and other obstructions shall be removed. All open ends of pipework shall be properly sealed by a metal or plastic cap at the end of each day's work or at the end of each section of work. The Contractor shall be liable for costs due to any damage caused by debris in pipes.

5.4.3.2 Concrete pipe laying

Concrete pipes shall be laid with their collars pointing upstream.

5.4.3.3 PE pipe laying and welding

The Contractor shall note the higher thermal expansion and contraction movements of PE pipe compared with other pipe materials. Pipe joint connections and pipe connections into structures shall be undertaken in accordance with the requirements of the Drawings and other specifications and after pipe thermal movements have been minimised.

Fusion butt welding of PE pipes shall be carried out by an approved certified welder with a recognised New Zealand certificate in accordance with NZS/AS 2033.

Pipes of dissimilar PE compound shall not be welded together. Pipes of dissimilar wall thickness shall not be welded unless otherwise indicated on the Drawings or directed by the Engineer. In this case the Contractor shall chamfer the inside of the thicker walled pipe so the ends of the pipes to be welded have the same thickness. The chamfer shall commence 50 mm in from the end of the pipe.

5.4.3.4 Gravity lines

Each separate pipe shall be individually set to line and within 10 mm of the invert levels shown on the Drawings provided that the deviation from a string line extending over two pipe lengths shall not exceed 10 mm. Each joint shall be completed before the adjoining pipe is laid.

5.4.3.5 Protection of buried bolts, couplings and valves

Couplings and flanges shall be cleaned to remove any loose dirt or contamination. A coat of Denso Multipurpose primer shall be applied over the whole assembly, working it in to obtain a full film cover on all parts of the assembly.

The assembly shall be contoured to a smooth profile with Denso mastic of minimum 5mm thickness, ensuring that the tape will not bridge in any areas or become perforated in service.

Denso Petrolatum tape shall be spirally wound, without stretching, over the whole of the area to be wrapped, with the heavily compounded side of the tape towards the work. Apply the tape with 55% overlap to achieve a consistent double thickness. Smooth the tape as wrapping proceeds to exclude any voids and ensure intimate contact with the mastic and the pipe protective wrapping.

Finally wrap the whole assembly (over the Denso Petrolatum tape) with Denso MP/HD tape in spiral winding, applying firmly and tight but without stretching with a 55% overlap to achieve a consistent double thickness.

The Denso Petrolatum tape shall overlap the pipe protective coating by a minimum of 100mm. The Denso MP/HD final tape shall overlap the Denso Petrolatum tape onto the pipe protective coating by a minimum of a further 50mm.

5.5 Pipe testing

5.5.1 General

Prior to the commencement of testing the Contractor shall submit, for the Engineer's approval, details of the Contractor's proposed methods, including details of test equipment and programme for testing and shall arrange for all tests to be witnessed by the Engineer.

The Contractor shall provide all equipment and temporary works necessary for the flushing of all debris from pipelines and to carry out the specified tests.

If any pipeline or manhole fails to meet the specified test acceptance criteria, the Contractor shall locate the faults, perform all necessary remedial work and retest the pipeline or manhole until the acceptance criteria are satisfied. This work shall be performed at no additional cost to the Principal.

5.5.2 Testing drainage lines

The drain line shall be checked by means of a mirror and lamp to ensure there are no obstructions in the barrel and to ensure the pipes are laid straight.

Pressure testing of the pipe systems shall be carried out in accordance with subsection 1.5, but it is strongly recommended the Contractor carry out its own intermittent testing during its drainlaying operations to ensure the final test requirements are satisfied. The Contractor shall submit for approval the method and equipment it proposes to use for any such intermittent testing. Care shall be taken when releasing water to avoid damage to the pipes and trench.

Prior to testing, the pipe system shall be completely cleaned out of all silt, rubbish and debris.

5.5.3 Testing of drainage pipe system (non PE pipes)

All pipelines, (excluding subsoil drainage lines) shall be filled with water and tested to a minimum head of 1.8 m head at the upstream end (except where this would produce a head in excess of 6 m at the downstream end). The test head shall be maintained for 30 minutes and the make-up water volume measured. The make-up water volume shall not exceed 0.5 litre per hour per linear metre of pipe per metre of internal diameter.

An air test may be acceptable, in exceptional circumstances, subject to the approval of the Engineer.

5.5.4 Testing of drainage pipe system (PE pipes)

Testing of PE non-pressure pipelines shall be undertaken in accordance with the requirements of section 7.3 of AS/NZS 2033, Method 1 – Hydrostatic test, unless otherwise specified or directed by the Engineer.

Where pressure rated pipes are used for drainage purposes, pressure pipe system testing may be required as specified or directed by the Engineer. Such testing shall be undertaken in accordance with the Basic pressure test (Visual) of section 7.2 of AS/NZS 2033 unless otherwise specified or directed by the Engineer.

5.5.5 Leaks, cleaning and maintenance

Should any leaks develop in any pipes, catchpits or manholes, such portions shall be taken up and relaid and the cost of any repairs or damage done by breakages or otherwise in so doing shall be borne by the Contractor.

Foreign matter shall not be permitted to enter any existing drainage systems.

6 Trenching, Bedding and Backfilling for Underground Services

6.1 Scope

This Section details the requirements for trenching, bedding and backfilling for pipes and underground services of the form and dimensions shown on the Drawings, and as specified in these Specifications.

Where the words pipe, pipelaying, pipework, etc., are used in this Section to determine phases of work, those words shall include duct, duct laying, duct work, etc., as applicable.

6.2 Compliance

Construction work performed under this Section shall comply with the general requirements of the following documents and the specific requirements of this Section:

- AS/NZS 2032:2006 Installation of PVC pipe systems.
- AS/NZS 2033:2008 Installation of polythene pipe systems.
- AS/NZS 3725:2007 Design for installation of buried concrete pipes.
- The Health and Safety in Employment Act and Regulations.
- Approved Code of Practice for Safety in Excavations and Shafts for Foundations.
- Guidelines for the Provision of Facilities and General Safety in the Construction Industry.
- Excavation Guide, Safety in Construction No. 5 to the extent that it does not conflict with any of the above.
- Local Authority standards.

6.3 Materials

6.3.1 Bedding and backfill

The requirements for bedding and backfill are specified on the Drawings and in this Section.

Where excavated material from the site does not meet the requirements for backfill, suitable material meeting these requirements and being compatible with adjacent materials shall be imported.

6.3.2 Geotextiles

Separation geotextiles shall be non-woven polymeric fabric, formed from a plastic yarn of a long-chain synthetic polymer composed of at least 100% by weight of propylene and shall contain stabilisers and/or inhibitors added to the base plastic to make the filaments resistant to deterioration due to ultraviolet and heat exposure. The fibres shall be rot proof, chemically stable and have low water absorbency.

The filter network shall be dimensionally stable (i.e. fibres shall maintain their relative position with respect to each other) and be resistant to delamination. The fabric shall be free of defects or flaws which significantly affect its physical and/or filtering properties.

Storage and handling of fabric shall be in accordance with the manufacturer's recommendations except that in no case shall the fabric be exposed to heat or direct sunlight to the extent that its strength or toughness are diminished. Fabric which is not to be installed immediately shall not be stored in direct sunlight.

Fabric shall not be stored directly on the ground. The storage area shall be such that the cloth is protected from mud, soil, dust, and debris.

Torn or punctured fabric shall not be used.

The separation geotextile shall meet the following minimum requirements:

- Minimum grab tensile strength (ASTM D4632) - 0.8kN
- Minimum puncture strength (ASTM D4833) - 2.2 kN
- UV resistance at 500 hours (ASTM D4355) - minimum 70 % of strength retained.

Refer to Section 8.3.2 for geotextiles associated with rock armouring.

6.3.3 Drainage aggregate

The drainage aggregate when sieved in accordance with NZS 4402 Part 1, test 9B shall comply with the following gradings:

<u>Standard Sieve</u>

Aperture Size	Percentage Passing
26.5 mm	100
9.5 mm	10 max.

6.4 Construction

6.4.1 Trenching

6.4.1.1 Excavation

Trenches shall be excavated in accordance with the longitudinal sections and the typical cross sections shown on the Drawings, and as specified below.

For nominal pipe diameters less than 700 mm trench sides shall be vertical from the pipe invert to at least 150 mm above the pipe crown.

For nominal pipe diameters between 700 mm and 1000 mm the maximum trench width at the crown of the pipe shall be O.D. + 500 mm.

Trench excavation shall be carried out expeditiously, and subject to all specific requirements of the contract. All hard surfaces (asphalt, concrete, etc.) shall be neatly sawcut prior to the commencement of excavation.

Excavated material shall be stacked well clear of the edge of the excavation and the size of the stockpile shall be limited to avoid any danger to the stability of the trench or adjacent services and facilities. Any surplus material shall be disposed of off-site.

The Contractor shall not open the trench more than 50 metres ahead of the pipelayers and, where soft ground is encountered, not further than approved by the Engineer. The open trench length may also be limited by the Engineer for excavations in residential areas or close to buildings and structures and when there is a risk of wet weather disruption.

If the Contractor by its own negligence over-excavates any area, then it may be necessary to carry out remedial filling with an approved material to achieve the original in-situ density, at no additional cost to the Principal.

Unsuitable excavated material shall be loaded out and removed immediately from the Site.

The trench invert shall be tested for strength to determine if it has sufficient strength or if over-excavation and material replacement is required. A hand held Shear Vane or Scala Penetrometer shall be used depending on the material type and depth of testing required. A Clegg Hammer may be used if the foundation material is non-cohesive. The test results shall be used to derive equivalent California Bearing Ratio (CBR) strength values using the correlations in Table 6-1 below.

The test frequency shall be 20 m maximum for trenches in non-road areas and 5 m maximum for trenches in permanent or temporary road areas.

Table 6-1: Material Strength Test Correlations

Estimated CBR value	Scala Penetrometer Number of blows per 50 mm penetration	Approximate minimum Shear Vane soil strength (kPa)	Approximate minimum Clegg Impact Hammer value (CIV)
3.5	1	50	7
5	1.5	75	8
8	2	100	10
10	2.5	150	12
13	3	200	14

6.4.1.2 Shoring

Shoring shall be provided as necessary to ensure the security of the work and safety of workmen, and to comply with the Health and Safety in Employment Act and its related Regulations and Codes of Practice. If, for any reason, the sides of a trench collapse, the Contractor shall at its own expense carry out all necessary remedial work.

Excavations with a depth greater than the width and exceeding 1.5 m depth shall be shored unless the excavated sides are battered to a stable slope.

The shoring shall protect existing buildings and other structures in the vicinity of the excavation from settlement and damage. Any repair work required to such structures shall be rectified at the Contactor's expense.

Before commencement of works the Contractor shall supply details of its proposed shoring method and appropriate calculations for the shoring certified by a Chartered Professional Engineer. Acceptance of the proposed method of shoring and calculations shall not relieve the Contractor of responsibility for the adequacy of shoring.

The Contractor shall provide the necessary notifications to the Department of Labour when excavation deeper than 1.5 m is required.

6.4.1.3 Water in trenches

Should water be present in a trench, the level shall be kept below the level of the top of the bedding until each joint has been made and backfilled.

The Contractor shall provide adequate plant as required to remove and dispose of water without interfering with pipelaying work.

The Contractor shall take precautions to prevent flotation of pipes in locations where open trench excavations may become flooded. The precautions may include partial backfilling of the trench leaving pipe joints exposed while awaiting testing of the joints.

The Contractor shall not permit any flooding of property, footpaths, or roadways to result from pumping operations. All water shall be disposed of at the nearest adequate and approved drain point. Pumping of sand and silt from excavations shall be avoided by providing a suitable trap to prevent such material being uplifted by pumping equipment. Any material inadvertently deposited in adjacent services or surface areas shall be removed immediately.

6.4.1.4 Unsuitable foundation

Where the material in the trench bottom is not suitable for a pipe foundation, the Contractor shall, upon receipt of an order from the Engineer, over-excavate the trench as necessary and backfill with an approved compacted granular replacement material.

Table 6-2: Trench Invert Strength

Trench situation	Minimum required CBR value for trench invert
1 In non-road areas	5
2 In permanent road areas with pipe cover to road subgrade less than 0.3 m	10
3 As in 2. but with pipe cover between 0.3 m and 1.0 m	8
4 As in 2. but with pipe cover greater than 1.0 m	5
5 Beneath temporary construction haul roads	10

Over-excavation shall be carried out if the strength of the trench invert is less than the CBR values in Table 6-2. The depth of over-excavation shall extend to foundation material with the above strength values or as directed by the Engineer.

Replacement material shall be compacted in layers not exceeding 150 mm thickness after compaction. The replacement material shall be tested for strength using a Clegg Hammer or Scala Penetrometer and shall achieve CBR strength values at least those given in Table 6-2. The test frequency shall be at 10 m maximum centres.

If the natural bottom of the trench becomes weakened or disturbed by the Contractor's own operations, then the Contractor shall undertake all necessary remedial action to strengthen the foundation with approved compacted granular backfill at its own cost.

6.4.1.5 Maintenance of trenches

Where trenches are excavated for pipes or services to be supplied and laid by others, the open trenches shall be maintained by the Contractor.

6.4.2 Pipe bedding and support

Pipe bedding is the layer on which the pipe is laid. Pipe support is the material above the bedding layer and surrounding the pipe up to 150 mm above the top of the pipe.

Bedding shall not commence until the base of the trench has been inspected and approved by the Engineer.

The types of pipe bedding and support required including the degree of compaction are defined on the Drawings.

Bedding shall be spread evenly on the trench invert and well compacted. The bedding shall be shaped to receive the pipe which shall bear firmly on the bedding along the full length of the pipe barrel. Socket holes shall be provided beneath each pipe joint to permit jointing of the pipe and allow the barrel to rest firmly on the bedding. Where unreinforced concrete bedding is required the details are shown on the Drawings. A construction joint shall be provided at each pipe joint using a softboard spacer or any other method approved by the Engineer.

The pipe bedding and support layers shall be tested to demonstrate that the compaction requirements defined on the Drawings have been met. Compaction compliance testing shall be conducted as follows:

Pipe bedding layer

If compaction compliance is in terms of density, a Nuclear Densometer shall be used.

If compaction compliance is not in terms of density, a Scala Penetrometer, a Clegg Hammer or other approved device shall be used.

The test frequency shall be at 20 m maximum centres for each placed layer.

Pipe support layers

The pipe support layers shall be carefully placed in 150 mm thick layers and compacted using hand-held rammers around and over the pipe to the depth shown in the Drawings.

If compaction compliance for any of the layers is in terms of density, a Nuclear Densometer shall be used.

If compaction compliance is not in terms of density, a Scala Penetrometer, Clegg Hammer or other approved device shall be used.

The test frequency shall be at 20 m maximum centres for each placed layer.

6.4.3 Backfilling

Backfilling above the pipe support layers and surface reinstatement of the trenches shall commence as soon as possible after laying, inspection and testing (if applicable) of each section of pipeline. Where applicable each trench shall be backfilled with material similar to that excavated from the trench and/or selected or supplied to comply with the requirements for backfill as shown on the Drawings and in this Specification.

The backfill up to the topsoil level in grassed surfaces and gardens or up to subgrade level in driveways, paved areas and roads shall be carefully placed in layers not thicker than 150 mm compacted thickness and compacted by hand operated mechanical equipment. Compacted backfill strength shall be at least equal to that of the adjacent ground as measured by a hand held Shear Vane.

For granular backfill layers, the degree of compaction required shall be in terms of density and strength as shown in Table 6-3 below.

Table 6-3: Granular Backfill Compaction

Trench situation	Density requirement for granular backfill (% of maximum dry density)	Strength requirement at road subgrade (CBR value)
1 In non-road areas	85	n/a
2 In permanent road areas with pipe cover to road subgrade less than 0.3 m	93	10
3 As in 2. but with pipe cover 0.3 m to 1.0 m	93 (top layer) 90 (lower layers)	10
4 As in 2. But with pipe cover greater than 1.0 m	93 (top layer) 90 (lower layers)	10
5 Beneath temporary construction haul roads	93 (top layer) 90 (lower layers)	10

Trench backfill testing shall be carried out at a frequency of one test per backfill layer at 20 m maximum centres.

Backfilling around pipes and structures shall be carried out in a manner which does not cause displacement of or excessive stresses in the buried structure. In general the backfill level on one side of the structure shall be within 300 mm of the level of the other side. The compaction techniques employed shall not overstress the structures.

A minimum cover of 0.6 m of compacted material complying with this Specification and the Drawings shall be provided over pipes before heavy vibratory compaction equipment is used in the backfilling process.

6.4.4 Subsoil drains

The pipes shall be laid as shown on the Drawings.

Unless shown otherwise on the Drawings, trenches shall have a minimum gradient of 1 in 100, and shall be excavated with vertical sides from the base to a minimum of 300 mm above the top of the pipe.

Subsoil drain filter material shall be clean durable stone, and when wet sieved in accordance with NZS 4402 Part 1: Test 9B, shall comply with the following gradings:

Standard Sieve Aperture Size	Percentage Passing
26.5 mm	100
13.2 mm	85 – 100
9.3 mm	80 – 95
4.75 mm	65- 85
2.36 mm	50 – 70
1.18 mm	35 – 55
600 µm	18 – 40
300 µm	3 – 25
150 µm	0 – 8
75 µm	0

The trench shall be backfilled with filter material placed in layers not exceeding 150 mm loose depth and shall be compacted to a density at least equivalent to that of the surrounding material.

6.4.5 Reinstatement

Ground disturbed by works constructed under this Section shall be reinstated to its pre-contract condition, or to such other condition as the Engineer may instruct.

All permanent surface reinstatement shall be as defined on the Drawings. Road resurfacing shall be similar in type, quality, texture, skid resistance and strength to the surrounding materials. Any traffic markings present prior to the Contract Works shall be reinstated on completion of the works.

The reinstated finished surface shall match the surrounding surface level. Trench or other excavation over-break will require fresh surface cutting to maintain straight lines and a tidy appearance to the surface finish.

Topsoil layers and surface layers in driveways, paved areas and roads shall be reinstated to match the surrounding surfaces or as shown on the Drawings.

All excess excavated material and any other material not used in the works shall be disposed of offsite unless instructed otherwise.

6.4.6 Testing – general

Copies of compaction test results shall be provided to the Engineer for approval prior to construction of any final reinstatement surfacing.

Tests shall be carried out at points varying across the width of the trench. Tests shall also be carried out at excavations around structures such as manholes and catchpits. Along trenches, individual tests shall meet the specified compaction requirements. The Engineer will instruct the Contractor to conduct further tests at the Contractor's expense should any tests fail. Any remedial work required by the Contractor to provide complying test results shall be done at the Contractor's expense.

A minimum of four tests shall be carried out in backfill at manholes and catchpits.

7 Concrete Work

7.1 General

The work covered by this section includes the supply of all labour, materials and plant for the mixing and placing of all concrete, together with all other associated trade work, complete as shown, all as described in the Drawings and/or Specification. It shall apply to components precast off site as well as those cast on site.

The Contractor shall do all preparation necessary to receive or adjoin other work, and shall be responsible for the installation of embedments in concrete, and shall form all penetrations, chases, nibs, etc. necessary for the work of other trades as shown in the Drawings unless directed otherwise by the Engineer.

7.2 Standard specifications

The requirements of the following standard specifications and any Standards quoted therein shall apply except where modified by this Specification. Where this Specification differs from any provision of the Standards the requirements of this Specification shall take precedence. Reference to any Standard shall include any amendments and any Standard in substitution thereof. Materials and workmanship shall comply with these Standards unless expressly noted otherwise.

NZS3101	The Design of Concrete Structures
NZS 3104	Specification for Concrete Production – normal and special mix grades
NZS 3109*	Concrete Construction (and the Standards to which it refers)
AS/NZS 4671	Steel Reinforcing Materials

* Construction Reviewer as defined in this Standard shall mean the Engineer.

7.3 Materials

7.3.1 General concrete requirements

The Contractor's attention is drawn to the important requirement for dense, durable, concrete. To minimise concrete shrinkage, mix designs shall provide concretes with the minimum possible water cement ratios and minimum cement contents which are consistent with a cohesive concrete of adequate workability, density, surface finish and required strength. Refer to Isthmus Specification for surface finish requirements for individual structures.

To ensure uniform concrete quality and compliance with the above requirements, all materials and concrete will be subject to detailed inspection at the construction site by the Engineer and all concreting operations shall be within effective quality assurance procedures.

Any subcontractor supply from off-site shall be under a quality plan approved by the Engineer which assures meeting of the requirements of this Specification. Notwithstanding this clause, the Contractor shall be responsible for supply of all materials.

The following concrete mixes as specified below shall be used in different parts of the Works as shown on the Drawings:

Table 7.1: Concrete Mixes

Grade Designation	Maximum Aggregate Size (mm)	Specified Compressive Strength (MPa)	Method of Placement	Specified Slump (mm)	Maximum water cementitious ratio	Minimum binder content (kg)	Additional Data
50 (special mix)	19	50	NBC	100	0.45	370	Binder must contain combinations of cement and Supplementary cementitious material (30% fly ash, 65% slag or 8% amorphous silica)

\+ NBC denotes nominated by Contractor

++ Shrinkage testing shall be in accordance with AS 1012.13

Prescribed mixes will only be permitted on small remote projects outside operating areas of ready-mix plants, and where concrete strength is less than 25 MPa.

Normal and special mix concrete shall be supplied from an off-site plant approved by the N.Z. Ready Mix Concrete Association to produce normal grade and special grade concrete, and satisfying the requirements of this Specification. Plant including transit trucks shall have sufficient capacity to meet comfortably the requirements of the largest daily pour required to be undertaken continuously between authorised concrete joints.

Concrete shall be delivered to where it will be placed and be discharged either directly into formwork or concrete buckets. Transport or placement by concrete pump will not be permitted without the approval of the Engineer.

The concrete shall be composed of the following materials: ordinary Portland cement, coarse and fine aggregates, water and material specified in Table 1.1. An air entraining admixture and a retarder may be used. No other ingredients shall be added to the concrete unless requested and permitted in writing by the Engineer.

7.3.2 Aggregates

Fine and coarse aggregates shall comply with the requirements of NZS 3121 and NZS 3111. The maximum size of coarse aggregate shall be 19 mm. Refer to Isthmus specification for aggregate type and size requirements for individual structures. The Contractor shall provide the Engineer with details of the types and source of supply of the aggregates, a minimum of two weeks before work is to commence. In addition he shall provide confirmation from the supplier that the proposed aggregates are non-reactive as defined in clause 6.1 of the CCANZ report no. TR3, Alkali Silica Reaction: Minimising the Risk of Damage to Concrete : Guidance Notes and Recommended Practice (2 no. edn) 2003. Once approval has been obtained for the aggregates to be used, neither the quality nor source shall vary without the prior approval of the Engineer.

7.3.3 Additives and admixtures

Unless specified otherwise, chemical admixtures will generally not be permitted in concrete except as provided for in NZS 3104 for High Grade and Special Grade and Special Grade Mass concrete or when special circumstances clearly warrant their use and for their use in special-class concrete.

All approved chemical admixtures and their use shall comply with AS 1478 and shall be used in accordance with the manufacturer's recommendation.

Technical details including data of all admixtures proposed to be used shall be forwarded to the Engineer for approval including written indication from the manufacturer(s) of its compatibility to other chemical admixtures and concrete ingredients.

Any admixture used shall not adversely affect the concrete or any reinforcement including protective coatings and shall be used only in locations and in such quantities as approved by the Engineer.

The chloride content of any products used in a mix shall be reported and included in the calculation of the total chloride ion content referred to in Table 3.11 of NZS 3101:2006. Calcium chloride as an admixture will not be permitted.

The total alkali contribution levels (measured as Na20 equivalent) of all admixtures shall be reported and included in the calculation of the total alkali content.

Admixtures shall not be mixed together prior to introduction to the mix.

Requirements for the use of shrinkage limiting admixtures in order to achieve the specified shrinkage test limits shall be determined by trial mixes and subsequent testing.

7.3.4 Superplasticers

If Superplasticisers are used, dosage rates shall be in accordance with manufacturer's recommendations. The proposed dosage shall be submitted to the Engineer for review. Superplasticisers shall be added at the site of the work and not at the batch plant unless suitable provision is made for the testing and documentation of pre-superplasticed slump, volume of superplasticiser addition and that adequate slump and workability retention is achieved to allow efficient placement and compaction of the concrete when delivered to site.

7.3.5 Air entrainment

Concrete containing fly ash shall not be air-entrained, unless the Contractor supplies proof from tests on trial mixes or previous production that the amount of air-entrained can be controlled within specified limits and that the compressive strength of the concrete will be satisfactory.

The use of air-entraining agents shall not be permitted in concrete, which is to be subsequently pre-stressed.

7.3.6 Corrosion inhibitors

Where the use of Corrosion Inhibiting admixtures is specified or where the Contractor requires its use to achieve required performance, the Contractor shall submit the following data / information to the Engineer for review and approval:

i Corrosion inhibitor type and manufacturer's printed technical information including the manufacturer's recommendations for application dosage and usage method.

ii Proposed dosage and procedures for introducing of corrosion inhibitor with evidence that the fresh and hardened concrete properties will not be adversely affected and the corrosion inhibitor is uniformly distributed.

iii Technical data on long-term protection capability of the nominated corrosion inhibitor.

7.3.7 Shrinkage reducing admixtures

If a shrinkage-reducing admixture is proposed by the Contractor, product details, manufacturer's printed technical information, intended dose rate and other relevant technical data on short and long term performance of the concrete shall be submitted to the Engineer for review and approval.

Any such admixtures shall be included in trial mixes for the works. They shall not be detrimental to the achievement of the strength and durability requirements of the Specification.

7.3.8 Reinforcing steel

Reinforcing bars shall be higher ductility plain carbon steel Grade 300E or 500E as shown on the Drawings, and shall comply with AS/NZS 4671. Grade 500E bars shall be produced by the Micro-Alloy process unless otherwise permitted by the Engineer. Reinforcing mesh shall be Super Ductile Grade 500E or Grade 300E. Where permitted by the Engineer the use of low ductility mesh may be used in structures where ductile behaviour is not critical.

7.3.9 Mortar

The mortar mix proportions shall be such as to meet the following requirements:

Compressive strength at 28 days – fc (50 mm dia x 100 mm cylinders tested in a similar manner to the concrete compression test) shall be 20 MPa, unless specified elsewhere on the Drawings.

Volumetric proportions of cement including fly ash where approved to moist sand with 3 to 5% moisture content (or equivalent proportions if weigh-batched) shall be not less than one part of cement to three parts of sand.

Admixtures or fly ash containing chlorides, nitrates or any other compounds likely to adversely affect the strength or durability of the mortar shall not be used.

The mortar shall be mixed to a uniform consistency before application.

Epoxy resin mortar shall be an approved sand filled epoxy resin prepared and placed in accordance with the manufacturer's recommendations. Concrete surfaces which are to receive epoxy resin mortar shall be cleaned of all laitance immediately prior to covering with an epoxy resin tack coat.

The curing of epoxy mortar shall be in accordance with the manufacturer's recommendations. Unused epoxy resin, either filled or unfilled, shall be discarded when it loses its plastic condition. Excess or spilled resin shall be cleaned up while in a plastic state.

7.3.10 Cement

Cement shall be type GP – general purpose Portland cement and/or blended cements complying with AS 3972 / NZS 3122 and the following requirements governing supply, quality and storage.

The intended source and brand to be used shall be advised as soon as possible and be to the Engineer's approval and, once approved, the cement shall not be changed without the written consent of the Engineer.

The Contractor shall arrange for cement manufacturer's test certificates to be forwarded to the Engineer. The certificate shall be provided prior to use of any cement from that batch to which the certificate applies.

The temperature of the cement shall not exceed 30 deg. C prior to mixing.

The Contractor may seek approval for the use of Portland Pozzolan cement complying with NZS 3123 or Portland Limestone Filler cement complying with NZS 3125 where necessary.

7.3.11 Water

The water shall not exceed 30°C temperature prior to mixing. Water used for making concrete shall be free from visible contamination by oil or organic matter, and shall comply with the requirements of NZS 3121. Where it is intended to use recycled water from within the batch plant for the production of concrete its effect on the total alkali content of the concrete shall be considered when determining the susceptibility for AAR. Where water is used for concrete curing the water used shall not stain the concrete.

7.3.12 Proprietary grout

Proprietary grout shall be provided in a pre-mixed form. 10 mm aggregate may be added where permitted by the manufacturer and where the thickness of voids to be grouted permits.

7.4 Concrete mix design

7.4.1 Concrete from an audited NZRMCA plant

The Contractor shall advise the Engineer of his mix details, concrete supplier and provide all supporting documentation if requested to do so to confirm the audit status of the ready mix plant.

7.5 Concrete supply

7.5.1 Production

Concrete shall be produced by one of the following:

a A ready mix concrete plant having a current grading as determined by the Qualification Committee of the N.Z. Ready Mix Concrete Association.

b A concrete plant complying fully with the requirements of NZS 3109 and the appropriate standard for factory or site mixed concrete.

7.5.2 Delivery records

Records shall be kept at the batching plant for each batch including the following:

i Batch number and docket number which can be referred back to the batch plant

ii Specified slump

iii Mix designation (minimum strength, aggregate size and admixtures)

iv Specified strength

v Date and time of mixing

vi Quantity delivered

vii Actual weight and type of cement, fine and coarse aggregate, weight of free water and hence the free water / cement ratio.

If concrete is delivered by truck from an off-site batching plant, delivery dockets containing this information shall accompany each truck. These dockets shall be made out in triplicate by the batch plant operator. The original shall be retained by the batch plant. The remaining two copies shall be given to the truck driver who shall retain one copy and give the other copy to the Engineer at the work area.

If delivery records do not agree with the approved mix design, the Engineer may reject the concrete for placement in the Works.

7.5.3 Discharge time

Discharge of concrete into the structure or formwork shall be completed within the following period of time after the first introduction of the cement to the aggregates or of mixing water to the cement and aggregates:

For concrete without an approved retarder	45 minutes
For concrete containing an approved retarder	100 minutes

In the event that the Contractor can measure concrete placing temperatures the discharge placement time limits may be modified by Table 7.2 below.

Table 7.2: Concrete Discharge Time Limits

Concrete Temperature at Time of Placement	Maximum Elapsed Time After Introduction of Water
Less than 10°C	Not permitted
10°C to 24°C	1.5 hours
24°C to 27°C	1 hour 15 minutes
27°C to 30°C	1 hour
30°C to 32°C	45 minutes
Greater than 32°C	Not permitted

7.5.4 Mixing and transport equipment

All materials and concrete shall be fully discharged from any mixing and transporting equipment used before it is charged with a new mix. The drum of the mixer and the trucks shall be thoroughly cleaned of all adhering concrete at frequent intervals during continuous operation; the mixers which have been out of use for more than 30 minutes shall be cleaned before any fresh concrete is mixed or introduced. Any water in mixing or transporting equipment shall be removed before concrete or materials are discharged into.

7.5.5 Additional water

After discharge from the batching plant, no water shall be added to special mix concrete in the transit mixer trucks, for any reason whatsoever.

For normal mix concrete water may be added on site subject to the requirements of clause 9.4.2.1 of NZS 3109. The quantity of added water shall be recorded on the delivery docket. Under no circumstances shall the amount of added water exceed 10 litres per m^3 of concrete.

7.5.6 Large concrete pours

For large concrete pours where the lesser dimension of concrete to be placed exceeds 500 mm the Contractor shall ensure early age thermal crack control is implemented by providing and installing temperature sensing devices of type and locations to the approval of the Engineer. The Contractor shall address the following variables for controlling temperature rise:-

- section thickness
- formwork and insulation
- ambient conditions
- concrete placing temperature

- concrete mix proportions
- controlling rate and lift height of concrete placement
- types and sources of cementitious materials
- adding flaked or chipped ice to mix water
- use of internal cooling pipes.

Temperature of the concrete at placement shall not exceed 25°C. Maximum temperature of the concrete shall not be more than 70°C and temperature differential between the core and outer surfaces of the concrete shall not exceed 20°C.

7.6 Testing of concrete

7.6.1 General

All equipment for the tests as required in NZS 3112 shall be provided by the Contractor. The Contractor shall supply all necessary labour to assist with sampling of the concrete and supply all necessary concrete for the testing.

The Contractor shall supply all equipment and labour necessary to prepare the samples and perform on-site tests, which shall be carried out under the Engineer's observation. All laboratory tests shall be performed by a registered testing laboratory approved by the Engineer and the Contractor shall pay all costs and incidental expenses incurred for testing services.

Where stripping times are required to be reduced because of the construction timetabling or for other reasons deemed valid by the Engineer, the number of specimens shall be increased. This number will be determined by the Engineer but will not normally be fewer than six compressive strength test specimens.

7.6.2 Slump tests

The results of slump tests taken on samples of concrete supervised by the Engineer at the point of delivery shall be the only basis for defining the slump of the mixes supplied.

The slump for cast in situ concrete shall be as specified unless authorised by the Engineer as a variation. Tolerances for slump shall be in accordance with NZS 3109, Table 9.1. Where a test sample at the time of placing fails to come within these tolerances the concrete is liable for rejection by the Engineer. Slump tests shall be carried out in accordance with NZS 3112: Part 1. At least one slump test shall be performed on each batch of concrete.

7.6.3 Compressive strength test samples

Each day during concrete placement on site a total of three compressive strength test specimens shall be moulded for each 75 m3 of concrete placed unless the Engineer directs otherwise. The specimens shall be prepared, stored and cured in accordance with NZS 3112: Part 2. If a specimen is required for testing for early strength indication at 7 days, a fourth sample shall be made.

Three specimens shall be tested at 28 days and the other specimen (if required) shall be tested for early age strength at 7 days. The strength of the concrete will be determined from the cylinder test strengths with tolerances as set out in NZS 3109. If concrete is supplied from more than one plant then this requirement shall apply to the concrete from each plant.

7.6.4 Core sampling

If after 28 days the compressive test specimens do not reach the required strength, the Engineer may at his discretion order the concrete represented by the failed tests to be removed from the work. The Contractor may elect, at his expense, to take core samples of the concrete to confirm the in situ strength of the concrete. The Contractor shall notify the Engineer before such samples are taken.

Sampling of cores shall be performed in accordance with BS EN 12504-1 and core sample testing is to be performed in accordance with NZS 3112 Part 2, Section 9. Sampling and testing shall be performed by an independent agency.

7.6.5 Air entrainment tests

Air entrainment tests shall be carried out in accordance with NZS 3104 clause 2.15.3.1 and percentage of air entrained determined in accordance with NZS 3112 Part 1. Air entrainment values for the work shall be between 3% and 6% for concrete with nominal aggregate size greater than 20 mm, and 4 to 8% for 10 to 20 mm nominal aggregate size.

7.7 Execution of work

7.7.1 Excavation, filling and compaction

All excavation and filling, including sub-base, basecourse and compaction, necessary for the construction of the concrete works, shall comply with the relevant sections of this Specification detailing bulk earthworks and unbound granular pavement layer.

Filling behind retaining structures shall not commence until one week after the concrete pour. Specified compaction behind retaining structures shall be achieved using hand-held compaction equipment and the Contractor shall not allow heavy compaction equipment within 1.5 m of retaining structures.

7.7.2 Formwork

7.7.2.1 General

Formwork shall comply in all respects with AS3610 and NZS 3109 Chapter 5. The Contractor shall be responsible for the design of all formwork. The Contractor shall ensure thorough checking of the formwork design as part of quality assurance. Where the Contractor uses proprietary materials, these shall be used in accordance with the manufacturer's recommendations and designed in accordance with the relevant New Zealand Standard.

Off the form surface finish shall comply with NZS 3114 and the Drawings.

7.7.2.2 Tolerances

The Contractor shall provide final concrete tolerances as set out on the Drawings and in accordance with NZS 3109.

7.7.2.3 Striking of formwork

The striking of formwork shall be carried out in accordance with NZS 3109. Where formwork is required to be removed early, the Contractor shall obtain the written approval of the Engineer. The Engineer will only give this approval if results of compressive tests show that the concrete is up to the strength that would normally be obtained at the minimum stripping time as given in Table 5.3 of NZS 3109.

7.7.2.4 Re-use of formwork

Where formwork is to be re-used, the forms shall be clean and free of any laitance that may exist on the surface. Oil or other approved formwork release agents shall be applied to the surface of the form before it is re-used.

7.7.2.5 Support of formwork

Formwork shall be designed and constructed to provide the necessary rigidity and strength to support the loads to be carried. Bracing shall be provided both transversely and longitudinally and provision shall be made by means of wedges or jacks for adjusting the formwork.

7.7.2.6 Embedded items

Where metal accessories are used they shall be constructed to be wholly or partially removable without damage to the concrete. Any embedded portion shall terminate not less than twice its minimum dimension from the concrete surface but in no case less than minimum cover requirements.

Resulting cavities shall be filled with cement mortar or epoxy mortar and the surface left sound, smooth, even and uniform in colour.

7.7.3 Steel reinforcing

7.7.3.1 General

Reinforcing materials shall be higher ductility plain carbon steel complying with AS/NZS 4671. The placing of steel reinforcing shall comply with NZS 3109, unless otherwise specified.

7.7.3.2 Stacking and protection

Reinforcing steel shall be stacked clear of the ground at all times. The steel shall be protected from damage at all times, and any grease, mud, spilled concrete or other coating on the steel shall be removed before it is concreted in the final work.

7.7.3.3 Bending, threading and welding of steel reinforcing

All bars shall be bent cold unless otherwise permitted by the Engineer. No bar shall be bent twice in the same place. Bends shall be as shown on the Drawings, and there shall be no kinks or bends on straight sections of reinforcement.

Grade 500E reinforcement produced by the Quenched and Self Tempered (QT) process may only be bent once and shall not be rebent on site. Grade 300E QT reinforcement may be rebent once only in accordance with the requirements of NZS3109.

Grade 500E reinforcement of 16mm bar diameter or less produced by the Micro-Alloying (MA) process may be rebent on site. The rebending process shall comply with the requirements of NZS3109 (i.e. heating to cherry red heat). Grade 500E MA reinforcement greater than 16mm diameter may only be bent once, and shall not be rebent on site. Grade 300E MA reinforcement may be rebent once only in accordance with the requirements of NZS3109.

Grade 500E QT and grade 300E QT reinforcement shall not be threaded or welded.

Welding of reinforcement, including tack welding, shall be carried out in accordance with an approved method statement and welding procedure. Proposed weld locations and details shall also be approved by the Engineer.

All welding procedures shall be in accordance with AS/NZS1554. Detailed welding procedures including proposed welding techniques and electrodes, with drawings and schedules as required, shall be prepared prior to welding commencing on site. Only welders qualified to AS/NZS1554.3 (Clause 10.2) shall be used for permanent works welding. Proof of welder's proficiency and qualification shall be made available on request.

Under no circumstances shall quenched and tempered reinforcement be welded.

All welds shall be category SP in accordance with AS/NZ1554.

All welding work shall be protected from the weather.

The standard for interpretation of non-destructive testing shall be AS/NZS1554. The extent of non-destructive examination (NDE) is set out below:

Extent of Non-Destructive Testing				
Weld Category	**Visual Means**		**Other Means**	
	Visual Scanning (Note 1)	**Visual Examination (Note 2)**	**Magnetic Particle or Liquid Penetrant**	**Radiography or Ultrasonics (Note 3)**
SP fillet welds	100%	10%	NA	NA
SP Butt welds	100%	10%	NA	Refer Note 3

Notes:

1. Visual scanning shall determine that no welds called for in the drawings or specification are omitted. Visual scanning shall also detect gross welding defects.
2. Visual examination shall determine whether the required weld quality (in accordance with Table 6.2.2 of AS/NZS1554.1) has been achieved. Visual examination shall be carried out by a suitably qualified and experienced welding inspector. Should the welding inspector have concerns with the weld quality, other NDE may be requested.
3. The extent of radiography testing for full butt splices of reinforcement will be determined on a case by case basis, taking into account the size and number of bars affected and their position in the structure.

Where the proportion of non-destructive examination (either visual or other means) called for above is less than 100%, the programme of testing shall be agreed prior to commencement of any welding. The programme shall involve full testing of the first 5% of welds, in order to pick up and correct the cause of any major defect at the commencement of welding. Once compliance is established, the frequency of subsequent testing may then be progressively reduced to achieve the overall level of testing specified above. In the event of a non-compliant test result, a return to full testing of the next 5% of welds will be required.

If the tests of any weld do no conform to the specified requirements, two additional specimens from the same weld shall be tested. In the case of failure of one or both of these additional tests, the length of weld covered by the tests shall be rejected.

Weld test results shall be cross-referenced to the weld location and shall be maintained on site as part of the as-built record.

7.7.3.4 Placing and fixing

The Contractor shall provide all labour and materials for the support of the reinforcing steel to maintain its correct position during the placing and compaction of concrete. Such supports are not shown on the Drawings. The minimum number of rigid supports to be provided by the Contractor shall be:

slab steel - two approved supports per square metre

wall steel - one approved support per two square metres

beam and column steel - one line of support per metre length

The Contractor shall be responsible for ensuring that the reinforcing is be held in position with the aid of approved spacers or supports. When access is required across light reinforcing bars or mesh, the Contractor shall make use of walking boards to avoid the need for workmen to walk on the reinforcing steel. Tie wire using annealed iron not smaller than 1.25 mm diameter or by approved clips shall be used to secure reinforcement at intersections. The ends of wire ties shall be bent away from the nearby faces of forms and shall not project into the concrete cover.

Plastic spacers or plastic footed steel high chairs are preferred. Concrete blocks used to fix the steel from the forms shall be at least as strong as the adjacent placed concrete and firmly wired to the reinforcement using wires cast into the blocks but may only be used where the concrete surface will not be exposed or the concrete surface will be plastered.

Starter bars shall be secured to prevent damage including restraint against swaying in the wind.

Where the height of bars extending above freshly placed concrete exceeds 1.5 m, the bars shall be fixed to a timber plate or similar to ensure no movement at the bar / concrete interface before the concrete sets. Bars shall only be lapped where shown on the drawings. Splices in adjacent bars shall be staggered by at least 600 mm unless shown otherwise on the drawings.

Reinforcing bar tolerances shall be as specified in NZS 3109.

Clear cover to all reinforcement, including stirrups and ties shall be as shown on the Drawings.

7.7.3.5 Reinforcement schedules

Copies of bar bending schedules provided by the Contractor's suppliers shall be provided to the Engineer for approval not less than two weeks before commencing any cutting or bending to enable review by the Engineer.

The bar bending detailer shall cross reference all schedules to the appropriate drawings and bar call up numbers.

7.7.3.6 Testing of reinforcement

The Contractor shall supply a test certificate issued by the manufacturer of the steel, for each grade and parcel of reinforcement material supplied to the site. Independent tests shall also be carried out by the Contractor in accordance with the following procedures.

One tensile and two bend tests shall be carried out on a random sample of every bar size, type and grade, for every 250 tonnes batch of reinforcing steel delivered to the site, with a minimum of one series of tests for each bars size, type and grade. The tests shall be carried out in accordance with AS/NZS 4671, by an independent TELARC registered testing laboratory and the samples shall be selected by the Engineer.

Reinforcement carrying a manufacturer's certificate of compliance with AS/NZS 4671 may have its tests carried out whilst initial use has commenced.

Reinforcing steel shall be deemed acceptable if it complies fully with the requirements of AS/NZS 4671 when tested in accordance with the above.

7.7.3.7 Fibre reinforcement

The Contractor shall ensure that fibre reinforcement is dosed and mixed into the concrete such that they are uniformly distributed right through the parent matrix. Steel fibre reinforcement shall be manufactured in accordance with ASTM A820.

Polypropylene fibre reinforcement shall be manufactured in accordance with ASTM C1116.

Alkali resistant glass fibre reinforcement shall be manufactured in accordance with ASTM C1116.

7.7.4 Concrete placing

All concreting shall conform to NZS 3109, Section 7 and this Specification.

Concrete shall not be pumped without the Engineer's approval. Where such approval is given, only a special concrete mix design suitable for pumping may be used.

The Engineer, or his/her representative, shall be advised at least 48 hours before the Contractor intends to place any concrete to enable him to inspect the formwork and reinforcement.

The concrete shall not be placed if the slump as measured in accordance with this Specification is not within the acceptance criteria specified within. Concrete not complying with NZS 3109:1997 Clause 9.3 shall be liable for rejection.

Concreting must be carried out in one continuous operation between ends of members and/or construction joints. Except at properly formed construction joints, fresh concrete must not be placed against concrete that has taken its initial set.

The concrete must be supplied and placed at an adequate rate to ensure that all the concrete in the forms can be kept plastic until placed in its final position and compacted, and all temporarily exposed surfaced covered by and knit in with fresh concrete so that no cold joints are formed. Equipment and personnel must be adequate to maintain the rate of concrete placement adopted.

The concrete shall be placed in the form as near as practicable in its final position to avoid rehandling. The depositing of large quantities of concrete at any one point and running or working it along the forms will not be allowed.

Concrete shall not be dropped from a height greater than 2 m unless the Contractor can demonstrate that their method of placement prevents segregation and that the mix design used for the concreting has been specifically designed to minimise the potential for segregation. The Contractor shall clearly specify in the Construction Method Statement the method of concrete placement. It shall remain the sole discretion of the Engineer to accept or reject the Contractor's mix design and any proposal to freely drop concrete from heights in excess of that specified above. The Contractor shall comply with the Engineer's decision at no additional cost of the project.

Concrete must not be moved horizontally more than 1.0 m by the use of vibrators.

All concrete shall be placed so as to completely avoid segregation. Segregated concrete will be taken as evidence of improper workmanship or incorrect mix proportions and such concrete shall be removed at the Contractor's expense.

When the concrete is conveyed by chutes, the angle and shape of the chutes shall be such as to allow the concrete to flow without separation of the ingredients. Chutes must be baffled or hooded at the discharging end to prevent segregation. These chutes shall be kept as vertical as possible and shall be kept as far as practicable full of concrete with their lower ends immersed in the newly placed concrete.

The delivery end of the chute shall be as close as possible to the point of deposit. The chute shall be thoroughly flushed with fresh clean water before and after each run, the water used for this purpose being discharged outside the form.

Concrete shall be deposited continuously and as rapidly as practicable until the section of the work is completed. Any concrete which has taken its initial set shall be rejected.

In hot weather, where necessary, the surfaces against which concrete is to be placed, including reinforcement and formwork, shall be lightly sprayed with water to prevent excessive absorption of water from the fresh concrete. Pre-wetted surfaces shall be free from excessive water before concreting.

Accumulations of water on the surface of the concrete due to water gain, segregation or other causes during placing and compacting shall be prevented as far as possible by adjustments in the mixture. Provision shall be made for the removal of such water as may accumulate so that under no circumstances will concrete be placed in such accumulations.

The concrete shall be placed in horizontal layers not more than 500 mm thick and each layer shall be compacted before the preceding layer has taken its initial set.

Cofferdams constructed to maintain a dry work area shall be sufficiently tight to prevent the flow of water through the space in which the concrete is being placed.

Concrete surfaces, upon or against which concrete is to be placed and to which new concrete is to adhere, that have stiffened to such an extent that the new concrete cannot be incorporated integrally with that previously placed, are defined as construction joints. The surfaces of construction joints shall be prepared in accordance with this Specification.

7.7.5 Transporting

The concrete shall be transported to its final position as rapidly as possible, by means which will prevent segregation, loss of materials and contamination.

Pump lines shall be supported clear of the reinforcement.

The equipment used in placing the concrete, and the method of its operations, shall be such as will permit introduction of the concrete into its final location without high-velocity discharge and resultant segregation.

Concrete pumps shall have a variable speed control and shall be capable of pumping concrete containing 20 mm aggregate through delivery lines not less than 75 mm diameter and for a distance required for placement within the works to meet requirements of this Specification.

Under no circumstance shall aluminium alloy pipes or fittings be used nor shall concrete be permitted to come into contact with aluminium during its manufacture, transport or placing.

Where delivery lines are exposed to the direct sunlight, they shall be protected by a covering of bags, fully saturated hessian or other approved means.

During delays in delivery of concrete to the pump, concrete in the lines shall be pumped at regular intervals to ensure that the concrete is "live". For piston type pumps at least two strokes of the piston shall be made at each pumping interval.

7.7.6 Compaction

The concrete shall be thoroughly compacted using form vibrators of approved types and such hand tools, vibrators and methods as necessary to ensure that the concrete is carefully worked around the reinforcement and embedded fixtures and into the corners of the formwork. Extreme care shall be exercised in thoroughly compacting concrete.

The Contractor shall maintain immersion vibrators in addition to the form vibrators to enable additional manual compaction where required to thoroughly compact the concrete.

Depth and rate of placement of concrete shall be such that preceding layers shall remain in a soft and plastic state when covered by each succeeding lift or layer. Vibration and other compaction measures shall extend through the full depth of each layer of fresh concrete and 150 mm minimum into the preceding layer to completely consolidate successive batches and merge them with each other. The vibrator shall not be allowed to penetrate into partially hardened concrete or to disturb reinforcement. Vibrators shall be inserted on a regular pattern with the radii of influence overlapping.

Immersion vibrators shall be inserted into the concrete continuously and progressively along the pour for sufficient time for each insertion to compact the concrete thoroughly and remove bubbles of entrapped air.

Form vibrators shall be firmly attached to the forms but the vibrator shall be capable of being moved up or down or laterally along the forms.

Immersion vibrators shall of such diameter and capable of transmitting a frequency such that the compaction and durability of the completed concrete can be achieved to the required standard.

At least one vibrator in working order shall be held in reserve for emergency use.

If pools of grout, water or laitance form readily during vibration, the concrete mixture shall be modified and submitted to the Engineer for review.

The use of self-compacting concrete may be permitted by the Engineer where circumstance prevents the use of vibrators to remove entrapped air pockets.

7.7.7 Construction loads and deflections

In providing support to construction from a previously constructed portion of the works the Contractor shall plan the removal of propping such that stresses and deflections are not excessive.

The construction loads imposed on a structure of age 28 days or more shall be such that the strength requirements do not exceed those induced by the design loading, unless it is demonstrated by calculation that the strength requirements are within the capacity of the supporting structure.

Where the structure is less than 28 days old, the allowable loads shall be appropriately reduced. The Contractor shall seek approval from the Engineer for all loads imposed during construction.

The Contractor shall demonstrate that calculated final deflections (following removal of construction loads and application of design loads) are either less than the calculated deflections which would be caused by the application of the design loading or within acceptable limits as defined by the Engineer. Such calculations shall take into account, where appropriate, non-recoverable deflections due to creep of young concrete.

7.7.8 Concrete curing and protection

7.7.8.1 General

The method of curing and protection of concrete proposed by the Contractor shall be such that the strength and durability requirements of the concrete as required by the Specification and the Drawings are met.

The minimum concrete curing requirements are as provided in Table 7.3 and Table 7.4 below. These tables reflect, with some amendment, to the requirements of Table 3.5 of NZS 3101:2006 and Table 5.1 of NZS 3101:1995 for Exposure Classification U.

Table 7.3: Minimum Concrete Curing Requirements (based upon NZS 3101:2006)

Exposure Classification	Curing Period (1)(3)(4) (under ambient conditions)
A1, A2, A3	3 days
B2	7 days
C	7 days (2)
XA1	3 days
XA2	7 days (2)
XA3	7 days (2)

Note:

1 Curing shall comply with Clause 7.8 of NZS 3109.

2 Concrete in C, XA2, and XA3 zones shall be cured continuously by direct water application such as ponding or continuous sprinkling, or by continuous application of a mist spray.

3 In addition to the requirements of Table 4 and Table 5 the concrete shall be cured for at least the period required to achieve the 7-day compressive strength of the concrete taken from cylinders cast at the time and place of the concrete pour for the element in question. The longer of the two durations defined by the curing time specified and the 7-day compressive strength shall apply.

4 Curing shall commence immediately after initial set of the concrete and shall continue until the cumulative number of days during which the temperature of the air in contact with the concrete is above 10C has totalled the minimum number of days required for curing in accordance with this Table.

Curing requirements for other exposure classifications are as follow:

Table 7.4: Minimum Concrete Curing Requirements – Other Exposure Classifications (based upon Table 5.1 of NZS 3101:1995)

Exposure Classification	Curing Period (1)(4)(5) (under ambient conditions)
U	7 days (2)(3)(4)

Note:

1 Curing shall comply with Clause 7.8 of NZS 3109.

2 Concrete in C, XA2, and XA3 zones shall be cured continuously by direct water application such as ponding or continuous sprinkling, or by continuous application of a mist spray.

3 In addition to the requirements of Table 4 and Table 5 the concrete shall be cured for at least the period required to achieve the 7-day compressive strength of the concrete taken from cylinders cast at the time and place of the concrete pour for the element in question. The longer of the two durations defined by the curing time specified and the 7-day compressive strength shall apply.

4 Curing shall commence immediately after initial set of the concrete and shall continue until the cumulative number of days during which the temperature of the air in contact with the concrete is above 10C has totalled the minimum number of days required for curing in accordance with this Table.

Where curing compounds are accepted by the Engineer for use in the Works the Contractor shall be responsible for ensuring that the curing compound proposed is compatible with any subsequent covering or surface treatments including coatings, sealers, paints, or waterproofing membranes.

When concreting in hot weather the work shall, where practicable, be protected from the direct rays of the sun and from drying winds.

When rain is likely to cause damage to fresh concrete or leaching of cement, the work shall be protected.

7.7.8.2 Curing

Freshly cast concrete shall be protected from premature drying and temperatures not permitted in this Specification. The concrete shall be maintained at a reasonably constant temperature with minimum moisture loss for the curing period. Curing methods which do not conform to the requirements of this Specification shall not be used.

Where heat accelerated curing is proposed for non-prestressed precast concrete elements it shall be as described in the relevant section of this Specification.

7.7.8.3 Curing methods

Curing methods shall comply with Clause 7.8 of NZS 3109.

7.7.8.4 Curing compounds

Curing compound details shall be approved in writing by the Engineer. Membrane curing compounds applied to the surface of construction joints or bearing areas where concrete or grout is subsequently to be placed shall be removed by scabbling, grinding or sand blasting. Only approved coloured liquid membrane-forming compounds are to be used.

The compound shall be applied in strict accordance with the Manufacturer's Specification and shall be applied as soon as the surface water has disappeared.

At least six (6) days prior to the use of curing compound, full details of the proposed compound shall be submitted to the Engineer for review. Such details shall be accompanied by test certificates to show that the compound will give satisfactory results for the proposed application.

Curing compounds shall be delivered to the Site in suitably labelled containers to enable identification of the batch number and date of manufacture.

Curing compounds shall comply with the requirements of AS 3799 for the classes and types specified in Table 7.5.

Table 7.5: Classes and Types of Curing Compounds

Description of Curing Compound	Class (to AS 3799)	Type (to AS 3799)
Wax-based compounds (wax emulsion)	A	1-D
Resin-based compounds (Hydrocarbon resin)	B	
Water-borne compounds	Z	

The water retention efficiency of the curing compounds for concrete works (AS 3799 – Appendix B) shall not be less than 92%. Curing compound shall be applied in two coats at a rate of not less than 0.2 litres per m2 per coat. The time between first and second coat must be in accordance with the manufacturer's printed recommendations.

The curing compound supplier shall implement and maintain a quality system in accordance with AS/NZS ISO 9001, as a means of ensuring that the product conforms to the Specification requirements.

For each curing compound proposed for the Work, a Certificate of Compliance to AS 3799 and this Specification shall be submitted to the Engineer for approval.

7.7.8.5 Cold weather curing

Precaution shall be taken to prevent the plastic concrete from freezing at any time. When the temperature of the surrounding air during curing is less than 10□C the temperature of the concrete shall be maintained at a temperature between 10□C and 20□C for the required curing period. Salts or chemicals shall not be used for the prevention of freezing.

7.7.8.6 Curing period

The minimum period for curing shall be in accordance with Table 4 and Table 5 of this Specification.

7.7.8.7 Protection against damage

The concrete shall be protected from damage due to load overstresses, heavy shocks, excessive vibrations and the effects of rain and running water particularly during the curing period.

All finished concrete surfaces shall be protected from damage from any cause including construction equipment, materials or methods.

Damaged concrete shall be repaired or replaced in accordance with this Specification or as otherwise approved by the Engineer.

7.7.8.8 Heat accelerated curing

Low pressure steam curing of precast concrete unit cast in steel forms shall be performed in accordance with NZS3109. Where steam curing is proposed the Contractor shall provide a Construction Method Statement to the Engineer for approval. The method of steam curing shall be such that the strength and durability requirements of the concrete as required by the Specification and the Drawings are met. As a minimum the following requirements shall be addressed in the Method Statement;

The form of enclosure to ensure containment of live steam in order to minimise moisture and heat losses.

The period of initial application of steam after the final placement and compaction of concrete, and after the initial set of concrete.

The maximum rate of temperature increase of the air within the steam enclosure in Centigrade per hour.

Maximum temperature of the air within the steam enclosure.

The maximum rate of cooling per hour.

Confirmation that steam jets will not be allowed to impinge upon any part of the concrete units or of a test specimen, or of their formwork or moulds. Neither shall any steam delivery pipe be attached directly to any formwork or moulds in such a manner as may cause localised overheating of the concrete.

The number of steam jets to ensure that a substantially uniform temperature is maintained under the steam covers such that the difference in temperature between any two points adjacent to the concrete units is not more than 10°C.

The method of allowing the concrete to cool gradually and evenly.

The proposed timing to expose the concrete to the surrounding environment indicating the temperature of the concrete in relation to the ambient temperature.

7.8 Embedded items

7.8.1 Cast-in items

Holding down bolts and inserts shall be secured and fixed before the concrete is placed or, if shown on the Drawings or if directed by the Engineer, recesses or blockouts shall be made in the concrete and the holding down bolts or inserts shall be grouted in place, or embedded in the second-stage concrete. The surfaces of all holding down bolts or inserts to be in contact with concrete shall be thoroughly cleaned immediately before the grout or concrete is placed. Holding down bolts and/or inserts shall be positioned and aligned in accordance with the tolerances specified or shown on the Drawings before the concrete is placed, and shall be held securely in the correct position during placing and setting of the concrete. All proprietary cast in bolts and inserts shall be installed strictly in accordance with the manufacturer's instructions.

The position tolerance on cast-in items shall be as follows:

- bolts and inserts +/- 10 mm

All cast-in items shall meet the durability requirements of the New Zealand Building Code.

7.8.2 Epoxy grouting of reinforcement

This section covers the general requirements for the materials and workmanship relating to the epoxy grouting of reinforcing bars into hardened concrete. Generally this will only be required for remedial works – epoxy grouting is not an alternative construction method to the Drawing details. Grout materials shall be supplied as pre-proportioned factory packaged products, and shall be stored, handled, placed and cured to the manufacturer's printed instructions. The Contractor shall obtain from the manufacturer a written procedure for use of the product, together with certification that the product is suitable for the intended application. These details shall be supplied to the Engineer not less than one week prior to commencing work. A copy of the manufacturer's procedure and instructions shall be kept on the site of the work for reference at all times. Reinforcing bars shall be deformed grade 300 or grade 500 bars complying with AS/NZS 4671. Concrete shall be at least seven days old before epoxy grouting work may proceed. Holes required for setting of bars shall be hammer drilled and shall comply with the Drawings unless otherwise specified/approved by the Engineer. Diamond core drilling will not be accepted.

All holes shall be thoroughly cleaned. This will require the use of clean water to flush out all dust and foreign matter. A wire bottle brush shall be used to ensure no dust or paste remains on the side of the hole. All free water shall be removed prior to grouting. Holes for vertical bars shall be vertical and holes for horizontal starters shall slope downward at 15 degrees.

Areas of bars to be bonded shall be blast cleaned to a "Near White" metal finish according to Australian Standard 1627, Part 4 Class 2½.

After blast cleaning all traces of loose material, blast products etc. shall be removed by brushing with a clean brush. Blast cleaning shall be carried out not more than 6 hours before grouting bars into place. Epoxy grout has a limited working time after mixing. This working time shall be as specified by the manufacturer. The placing of grout shall be performed only during this specified working time and any unused grout remaining beyond this time shall be discarded.

Particular care shall be taken to ensure that:

- Ambient conditions during placing and curing are correct for the product used.
- Epoxy components are thoroughly mixed, and in the correct ratio.
- The annular space around the bar is completely filled with epoxy (no air locks).

The Contractor shall use an epoxy grout suitable for the conditions.

Completed anchorages shall be randomly selected by the Engineer for testing by loading to twice the working load specified on the drawings, or the yield strength of the bar whichever is the lesser. The Contractor shall allow to test 10 percent of the completed anchors, with a minimum of two, unless directed otherwise by the Engineer. Test loading shall be carried out in the presence of the Engineer or his representative.

7.8.3 Drilled-in fixings

All post-installed anchors shall be either mechanical (expansive type) anchors or chemical (adhesive type) anchors manufactured and supplied from an approved anchor supplier. The selection of anchor type, embedment depth and spacing shall be as shown on the Drawings or otherwise approved by the Engineer. The installation of these anchors shall be carried out strictly in accordance with the manufacturer's instructions.

The Contractor shall advise the Engineer immediately where anchors are to be drilled in cracked concrete zones. All anchors shall meet the durability requirements of the New Zealand Building Code and be tested in accordance with the same requirements as for epoxy grouted reinforcement.

7.8.4 Embedded sleeves, conduits and pipes

Sleeves, conduits, pipes, cores or other penetrations shall not be cast into the concrete unless they are stipulated on the Drawings, Specification, or instructed to be inserted by the Engineer.

7.9 Construction and contraction joints

7.9.1 General

Construction and contraction joints shall be formed in the positions shown on the Drawings unless a change is approved by the Engineer. Additional construction joints may not be provided nor may any horizontal or vertical joints be omitted without written approval by the Engineer. Any such approval will not be a Variation.

Construction joints shall be made in accordance with Section 5.6 of NZS 3109. Movement control joints shall have a concrete finish similar to that of the adjacent concrete.

Proprietary products included in or at the joints shall be installed in strict accordance with the supplier's specifications and recommendations.

Unless shown otherwise on the Drawings all joints between precast and in situ concrete shall be prepared and constructed to meet the requirements for 'Type B' construction joints in accordance with NZS 3109.

The Contractor's proposals for the pattern of joints for surfaces that will be visible on completion of the work shall be agreed by the Engineer before any concrete is poured, including precast concrete.

20 x 20 mm fillets and chamfers shall be used on all corners, unless expressly noted otherwise on the Drawings. All external angles in exposed members shall be protected against damage after stripping of formwork. The Contractor shall give a slight bevel to all insertions to ensure easy removal without damage to the concrete.

The Contractor shall nominate a limited number of responsible personnel who shall be involved in the mixing and placement of the in situ joints. The Contractor shall demonstrate his familiarity with the procedures required to ensure that the quality requirements shown on the Drawings and contained in this Specification are achieved.

Should edges at construction joints be damaged the Engineer reserves the right to nominate the method of repair, at the Contractor's cost.

7.9.2 Preparation of joints

For horizontal joints the Contractor shall prepare the surface of the concrete by green cutting (using high velocity water/air jets or vigorous wire brushing) to remove all laitance and inferior surface concrete after the concrete has hardened sufficiently to prevent loosening of any aggregate which is not removed. The time during which green cutting is feasible may be extended by the application of a surface retarder (Rugasol or equal approved). Light scabbling will be permitted providing the Contractor can demonstrate to the Engineer that the joint surface does not become cracked and exposed aggregate loosened.

For other than horizontal joints a retarder shall be used to prepare the joint. Immediately on removal of the formwork the surface of the concrete shall be prepared in a similar manner to horizontal joints. Surfaces to all joints shall be in a saturated condition prior to placing of the fresh concrete but all standing water shall be removed to leave a bright clean surface free of laitance before concreting commences. Care shall be taken to avoid damage to the edges of the surface and to the aggregate forming the surface of the joint. Construction joint types as shown on the Drawings shall comply with clause 5.6.3 of NZS 3109.

7.9.3 Sealants

Joints so designated on the Drawings shall be sealed, with all work being undertaken strictly in accordance with the manufacturer's instructions. The Contractor shall submit to the Engineer for approval, details of his proposed sealants and the Contractor shall provide to the Engineer a suitable manufacturer's guarantee. This guarantee shall be subject to approval by the Engineer. All work shall be undertaken by an experienced applicator approved by the manufacturer and evidence of such approval shall be furnished to the Engineer before any sealing work is undertaken.

The concrete to which the sealant is to be attached shall be of the highest quality, with no loose laitance honeycombing or air voids. The standard of cleanliness of the joint and the amount of moisture in the concrete shall be controlled to the manufacturer's requirements. The applicator shall be responsible for the supply and application of all sealants and he shall have the right to require restoration of the joint if the standard of concrete at the joint is considered to be unsatisfactory. The applicator shall liaise with the Engineer at all times and if necessary shall consult with him before concrete is placed to ensure that the method of forming the chase for the sealant will give a satisfactory surface finish.

7.9.4 Waterstopped joints

Waterstops shall be in the form of flexible strips of moulded polyvinyl chloride (PVC) of the patterns shown on the Drawings and shall be obtained in long rolls so that jointing on site is eliminated as far as possible. All waterstops shall be of the material and within 10 mm of the width specified or shown on the Drawings, and shall be suitable in the opinion of the manufacturer to withstand the local head of water and relative joint movement.

The only joints to be made on site in PVC waterstop shall be straight heat welded butt joints. Only joints between lengths having the same section shall be made on the site. All such joints shall be formed in accordance with the recommendations of the manufacturer. Waterstops shall be continuous and watertight at all junctions and intersections.

Joints in waterstops shall have a tensile strength of not less than 75 per cent of the strength of the parent material. Jointing tests to destruction on each type of waterstop shall be made on each joint type to confirm strength, prior to adopting the material or joint.

Minor surface defects shall not exceed 100 mm in length of 1.5 mm in depth.

PVC waterstops shall be manufactured from pure PVC with suitable stabilising and plasticising agents added but containing no fillers or scrap materials. The PVC material shall conform to Class 3, Grade 2 of BS 2571. Testing of the waterstops shall be to methods accepted by the Engineer which give a reliable indication of performance.

Waterstops shall be installed so that they are securely held in their correct position whilst concrete is placed. The concrete shall be fully and properly compacted around the waterstops to ensure that no voids or porous areas remain. Where reinforcement is present, adequate clearances shall be left between this and all waterstops to permit proper compaction of the concrete. No holes shall be made through any waterstops for fixing purposes other than through the flanges.

Alternative hydrophilic type waterstops will be permitted if the Contractor can provide to the Engineer a suitable manufacturer's guarantee that the product is suitable for the application where used. Any alternative waterstop shall be installed in strict accordance with the manufacturer's instructions.

7.10 Tolerances and surface finishes

All concrete work shall be set out and constructed to achieve the structural tolerances specified in NZS 3109. The Contractor as part of quality assurance shall check setting out for all concrete work for accuracy.

Surface finishes shall comply with the requirements of the landscape architects drawings and specification.

7.11 Repair of concrete

7.11.1 General

The Contractor shall advise the Engineer of the presence of any defective concrete.

Unless otherwise approved by the Engineer, repair of imperfections in the concrete shall be completed within seven days of removal of the forms. Where epoxy resin repairs are required, the repair shall not be made until 28 days after the concrete has been placed. Repair of the concrete shall be performed by skilled workmen, and the repairs shall be of a quality comparable to the adjacent area of the structure. Special attention may be required to achieve acceptable long term surface finish match to visible areas such as colour and texture matching.

7.11.2 Mortar or repair with new concrete

Existing concrete surfaces to which the new concrete or dry pack mortar is to be bonded shall be clean and rough. The surface shall, if necessary be roughened, and all damaged, loosened or unbonded portions of existing concrete shall be removed.

The methods of repair shall follow the procedures set out in the Guide to Concrete Repair and Protection (Standards Australia) for dry pack mortar, concrete or plaster. Where steel remains exposed after all areas of loose or otherwise unacceptable concrete have been removed, the material for repair shall be concrete. Existing concrete shall be removed from behind the steel for a distance of at least twice the normal aggregate size, and fresh concrete placed in accordance with the above referenced guide. The repair shall be free of shrinkage cracks and drummy areas.

Dry pack mortar filling shall not be used for holes extending entirely through concrete sections, for holes which are greater in area than 300 mm square and deeper than 100 mm or for holes in reinforced concrete which extend beyond the reinforcement nearest the surface.

Holes remaining after dismantling form ties shall be neatly plugged with mortar filling. The mortar shall be colour matched with surrounding concrete and the finished surface of the repair shall be recessed 3 mm below the surface.

7.11.3 Epoxy resin repair

Epoxy resin or epoxy resin bonded repairs shall be used when the repairs have not been made within 48 hours after the removal of forms, or, in the case of unformed concrete, within 48 hours after the placing of concrete. Such repairs shall not be undertaken until the parent concrete has had a minimum of 28 days curing.

Imperfections to be repaired with epoxy resin shall be chipped back to sound concrete and the edges of the holes trimmed square for a minimum depth of 3 mm. If reinforcement is exposed, remove concrete to 30 mm behind it prior to the filling of the repair.

Immediately prior to carrying out a repair, the surface of the concrete in the hole shall be cleaned of all contaminants by sandblasting.

The surface of the hole shall be mopped dry, and where necessary, effective means taken to exclude all surface water. The surface of the hole and the concrete immediately surrounding it shall be dried using lamps, an oxy-acetylene flame fitted with a descaling tip, heater, dry oil-free compressed air or other suitable means approved by the Engineer. Prevent damage to the concrete during drying. When the hole is free from surface moisture, a temperature of approximately 25⍰C shall be maintained over the whole affected area for a period of 30 minutes.

When so prepared, the surface of the hole shall be painted with one or more coats of an approved unfilled epoxy resin prepared and cured in accordance with the manufacturer's written instructions.

A compatible epoxy resin with approved filler prepared in accordance with the manufacturer's written instructions or wet concrete shall be applied to the clean surface of the unfilled epoxy resin, steel trowelled to a smooth surface and allowed to cure as detailed in the manufacturer's written instructions. During that time the area shall be kept dry.

Unused epoxy resin, either filled or unfilled, shall be discarded when it loses its plastic condition. Excess or spilled resin shall be cleaned up whilst in a plastic state.

Should it be necessary to use formwork to mould the filled resin mixture, the forms shall be coated with an appropriate release agent.

The finished surface of the epoxy resin mortar or concrete shall conform to NZS 3114. Any grinding required shall be performed using a silicon carbide or other suitable abrasive, and under water if possible.

Repairs shall not be made with epoxy resin mortar infills to surfaces for which F3 or better finish are specified, but an epoxy bonded repair may be used. Care shall be taken that accumulation of foreign matter or staining due to any cause does not occur on such finished surfaces. Any accumulation or staining which is in the Engineer's opinion unsightly, shall be cleaned off by the Contractor using a method approved by the Engineer.

7.11.4 Crack repair

The Contractor shall notify the Engineer prior to commencing the repair of concrete cracks. When repairs are to be undertaken the methodology and selection of repair material shall take account of whether the crack is dormant or live, width and depth, whether or not sealing against pressure is required, appearance of repair surface or colour match is necessary.

The Contractor shall obtain the Engineer's approval for the crack repair method and the materials used shall be applied strictly in accordance with the manufacturer's instructions by suitably qualified applicators.

7.12 Slab on grade

The Contractor shall observe the following minimum requirements before placing concrete ground slabs. The Contractor shall compact the subgrade with a footpath roller or plate compactor. The Contractor shall obtain the Engineer's approval before placing hardfill on the subgrade, and shall remove any soft spots as directed. The Contractor shall ensure that any backfill around foundations, tie beams, sumps and the like is well compacted and not subject to further movement.

The Contractor shall place and compact hardfill or basecourse to the specified thickness on the prepared subgrade. The Contractor shall place a blinding layer of sand over the hardfill followed by a layer of 0.4 mm thick Permathene or other approved dampstop. Permathene joints shall be lapped 150 mm and secured by an approved adhesive tape.

The Contractor shall place without cold joints, compact and finish the concrete after placing the reinforcing steel. Slabs shall be laid to the falls shown on the Drawings and shall be finished by mechanical trowelling to the specified finish.

The concrete shall be cured for 14 days.

7.13 Precast units

7.13.1 General

Previous sections shall apply unless noted otherwise. The Contractor shall comply with the recommendations of the Precast Concrete Handbook as amended by the New Zealand Commentary.

Concreting and Curing:

Each unit shall be cast in one continuous operation, and shall be properly cured as soon as possible after casting. Steam curing may be used, or wrapping in polythene. Final units shall be crack free.

Inspection and Testing:

The Contractor shall provide delivery records for each unit constructed in accordance with clause 7.5.2 of this Specification.

Handling, Transportation and Erection:

Units shall be handled, transported and erected so that no damage is caused to them at any stage. Every care shall be taken against damage or soiling in transit. Any damaged unit will be rejected by the Engineer. Units shall be lifted only by using the lifting eyes. The Contractor shall be responsible for the design, provision and subsequent removal and making-good of such lifting eyes as may be required.

The Contractor shall be responsible for erection and shall notify the Engineer of his methodology for lifting. Should the Contractor wish to modify the lifting eyes and connection details to facilitate his assembly method, he shall notify the Engineer in sufficient time to allow his modifications to be properly considered by the Engineer prior to the casting of the units.

Tolerances:

The onus is on the Contractor to study the Drawings, and to advise the Engineer, before construction begins, if he considers he will not be able to meet the tolerances inherent in the details shown in the Drawings. In this connection, his attention is drawn to the fact that the total tolerance indicated by a detail may have to be achieved by the co-operation of several trades (e.g. Precaster for fixings in precast work, Metalworker for brackets and the like, and Concreter for cast in situ fixings).

7.13.2 Boardwalk and Tukutuku bridge decks

The Contractor shall be responsible for the design and construction of the precast prestress decking for the 5.0m and 3.0m Boardwalk and the 2.4m and 1.05m Tukutuku bridges.

The Contractor shall supply the Engineer with a set of calculations and supporting Producer Statement – Design (PS1) for each element prior to construction.

The elements shall be designed in accordance with the requirements shown on the drawings together with the following requirements:

- Design life 50 years.
- The loads noted on the drawings are un-factored ULS loads. The Contractor shall consider the following as a minimum in their design:
    - The uplift wave loads shall consider 0.9G (Dead) plus the wave live load
    - The downward wave loads shall consider 1.0G (Dead) plus the wave live load
    - Seismic load cases
    - Static live loads.
- The Contractor shall consider how the connection of the timber nailing strips to the prestress slab and consider within his design the effect these may have on the integrity of the prestress strands and durability requirement. The contractor may consider the use of cast in inserts subject to the Engineer's approval.
- The ends of the prestress slabs shall be epoxy coated to prevent corrosion of the prestress stands. The Contractor shall propose a suitable epoxy for Engineer's approval for use under regular wetting, wave environment, geothermal atmosphere and comply with the Chemical Exposure Classification XA2. Alternatively the Contractor may propose an alternative measures to meet this requirement for the Engineer's approval.

7.14 Shop drawings

The Contractor shall produce Shop Drawings for all precast units and metal work cast into concrete, and submit them to the Engineer for approval prior to commencing operations.

The Contractor shall be responsible for the accuracy of all such Drawings and details. These Drawings shall be produced so as to enable competent tradesmen to fabricate the structures to the dimensions and standards given in the Drawings and this Specification.

8 Rock Armouring

8.1 Scope

This section covers supply and construction of rock armouring and riprap protection, including grouted sloping rock revetments.

8.2 Compliance

Materials and construction work performed under this section shall be tested to the general requirements of the following documents and the specific requirements of this section:

NZS 4407	Methods of testing road aggregates
AS 3706	Geotextiles – Methods of test
AS 2001	Methods of test for textiles – Physical tests

8.3 Materials

8.3.1 Rock armour

8.3.1.1 Grading

This section pertains to the rock rip rap scour protection at the toe of the retaining walls. For grading of other rock rip rap elements refer to Landscape Architect specification and drawings.

Rock armour material shall be well graded and shall conform to the following grading limits:

Rock Gradings		**Mean size**	**Range**	**60% by mass to be between**	**50% by mass to be greater than (W_{50})**
Rock armour at toe of retaining walls	Block weights (kg)	*50*	*20-150*	*20-80*	*50*
	Equivalent "mean diameter" (mm)[1]	*300*	*230-450*	*230-370*	*300*

1. Equivalent mean diameter is given for guidance only and is based on the rock density specified.

Poorly graded or gap graded armour rock shall not be permitted except as approved by the Engineer.

8.3.1.2 Shape

The rock armour shall not contain stones with a length to thickness (L/d) ratio greater than 3; where the length, L, is defined as the greatest distance between any two points on the stone and the thickness, d, as the minimum distance between two parallel straight lines through which the stone can just pass.

Armour rocks showing clear signs of significant edge or corner wear or of severe rounding on more than one face shall not be used.

8.3.1.3 Density

For 2.65t/m3 parent rock, the average density of rock used for riprap shall be not less than 2650kg/m3 with 90% of the rocks having a density of greater than 2550kg/m3.

8.3.1.4 Weathering resistance

Rock weathering resistance shall be tested and the resulting quality index shall be AA, AB, AC, BA, BB or CA.

8.3.1.5 Rock integrity

Rock shall be free from visually observable cracks, veins, fissures, laminations, unit contacts, cleavage planes, or other such flaws which could result in breakage during loading, unloading or placing.

8.3.1.6 Impurities

Rock shall be visually clean and free from impurities such as clays and soils when placed in the construction works.

8.3.2 Geotextile filter fabric

The geotextile filter fabric under the rock rip rap shall be Texcel 600R.

Storage and handling of fabric shall be in accordance with the manufacturer's recommendations except that in no case shall the fabric be exposed to heat or direct sunlight to the extent that its strength or toughness is diminished. Fabric which is not to be installed immediately shall not be stored in direct sunlight.

Fabric shall not be stored in contact with the ground. The storage area shall be such that the cloth is protected from mud, soil, dust, debris and direct sunlight. Torn or punctured fabric shall not be used.

8.4 Construction

8.4.1 General

The works shall be constructed in the locations and to the dimensions shown on the Drawings. The Contractor will be responsible for arranging stockpile areas and work areas with the Principal.

8.4.2 Protection of placed material

Each layer shall be protected by the subsequent layer as soon as possible after placement, with a maximum unprotected length of each material of 20 metres, in order to minimise wave damage or slumping in the event of storms during the construction period.

The Contractor shall obtain daily weather forecasts and if storms are predicted every effort shall be made to complete armour layers to protect partially completed works. Where the Contractor is to leave the works for the weekend and there is a possibility of storm erosion of the constructed works, particularly the exposed subgrade or underlayer construction, the Contractor shall provide all necessary temporary protection in the form of temporary placement of geotextile and armour across the end of the construction, or other methods, as may be necessary. These temporary protection works shall be removed prior to construction commencing again. No payment will be made for temporary protection works.

Material eroded by wave action or other causes shall be made good at the Contractor's cost before placing the subsequent layer. All material eroded and deposited on the foreshore or seabed outside the area of the Works shall be removed by the Contractor at his own cost.

8.4.3 Geotextile handling and placement

Geotextile material delivered to site shall be stored in a dry condition and shall remain in its protection wrapper until use. Geotextile shall be carefully handled at all times. Damage such as rips, tears or holes shall be repaired as directed by the Engineer. When patching of the geotextile is permitted to repair damage, the patch shall extend a minimum of 300 mm in all directions from the damaged area, and shall be sewn in place to the manufacturer's recommendations.

All geotextile fabric sheets shall be placed loosely and flat against the prepared slope without any folds or wrinkles. All adjacent geotextile fabric sheets shall be lapped to form a continuous membrane. Laps in all directions shall be a minimum of 500 mm.

If approved by the Engineer, the geotextile fabric may be sewn into sheets prior to placement in the works, with the size of the sheet being determined by the Contractor to suit his placement method. All sewn joints shall be to the manufacturers recommendations a copy of which shall be forwarded to the Engineer. The Engineer shall inspect and approve the joints prior to the sheets being placed in the works.

The geotextile sheets shall be firmly held in place to prevent movement during the placement of overlying rock. If movement occurs prior to or during placement of rock, then the rock shall be removed and the geotextile re-laid.

If the geotextile fabric needs to be placed underwater, the contractor shall provide a method statement to be approved by the Engineer prior to commencing placing.

8.4.4 Rock armour

The subgrade shall be shaped and prepared for the subsequent placing of rock armour as specified in the Earthworks section of this Specification. Geotextile filter fabric shall be placed on the prepared subgrade.

The placement method of armour and underlayers directly on geotextile filter fabric shall be approved by the Engineer prior to placement and shall NOT include, end tipping, drifting or rolling stones down the slope.

Armour rock shall be carefully placed to avoid damage to any already placed rock or underlayers, with large rock fragments uniformly distributed and smallerTouche rock fragments filling spaces between the larger rocks ensuring that the resulting layer is well keyed together, densely packed and of the specified dimensions.

Where armour directly contacts the geotextile, armour stones shall be individually placed onto the geotextile with a maximum drop height of 300 mm.

8.5 Testing and tolerances

8.5.1 Testing

The Contractor shall arrange all required testing and shall supply results to the Engineer for approval. The Engineer reserves the right to carry out additional testing which, if uncovering deficiencies or discrepancies, shall be paid for by the Contractor.

The Contractor shall interrupt or divert his operations as necessary to permit any tests required with complete safety. The following tests shall be used as a minimum to confirm material properties and construction accuracy:

Test No.	Test	Test method	Frequency of testing
1	Solid density	NZS 4407: 1991, Test 3.7.1 (non vesicular aggregate)	*One per source* [1]
2	Weathering resistance	NZS 4407: 1991, Test 3.11	*One per source* [1]
3	Underlayer rock grading	NZS 4407: 1991, Test 3.8.2	*One per 500* m^3 *per source* [1]
4	Set out dimension	Survey	*One section per 10 m of wall*
5	Armour layer thickness	Survey as specified in following notes	*One section per 20 m of wall*
6	Armour grading	See following notes	*One per 10 m of wall*

Notes:

1. A rock "source" is defined as each new face opened within a quarry, or each 1,000 m3 of rock from the same face or lava flow.

Note on testing armour grading

At 10 metre intervals along the seawall, rock armour grading shall be tested by selecting a sample of at least 20 stones and evaluating their "mean diameter" by measurement, prior to the rocks being placed in their final position in the works. Alternatively testing may be from a sample of at least 100 randomly selected stones from a stockpile with one sample tested per source of rock armour.

Where measurements are taken, the "mean diameter" shall be calculated from the average of at least 3 circumferential measurements measured orthogonally and approximately about the stone centre of mass.

8.5.2 Tolerances

Rock materials shall be placed to the levels, dimensions and slopes shown on the Drawings. When the surface profile is measured using the techniques specified, the following vertical tolerances shall be achieved.

Subgrade levels	-0.1 m to +0.0 m
Rock rip rapthickness (average per profile):	-0.0 m to +0.1 m

8.5.3 Weathering resistance alternative

As an alternative to the specified weathering resistance test a 'Schmidt' hammer may be used utilising the following test procedure, "Suggested Method For Determination of the Schmidt Rebound Hardness - Rock Characterization Testing and Monitoring, I.S.R.M. ,1981".

Minimum acceptable Schmidt hardness is 50. Sampling shall be taken randomly throughout each batch and shall be representative of the general consistency of the stabilised material produced.

REPORT

Tonkin+Taylor

Rotorua Lakefront Development

Geotechnical Interpretive Report

Prepared for
Rotorua Lakes Council
Prepared by
Tonkin & Taylor Ltd
Date
February 2019
Job Number
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Exceptional thinking together
www.tonkintaylor.co.nz

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Table of contents

Executive Summary

The table below summarises the geotechnical considerations and recommendations for redevelopment of the Rotorua Lakefront. This summary is an overview of advice presented in the report and should be read in conjunction with the relevant detail included in the main body of the report.

Consideration	Outcome
Subsurface conditions	• The site was observed to comprise up to 2 m of fill (loose sands and silts) overlying Holocene age alluvial deposits comprising very soft to soft silts which overlay layers of loose to dense sands and gravels and firm to stiff silts at depth. • The site is classified under the GNS soft ground map as within Zone D indicating Holocene deposits deposited in the last 10,000 years. • On the lakebed (over water) very soft silts were observed at lakebed level (no fill present). • Weak rhyolite rock was observed in BH03 at a depth of 14 m below ground level (bgl), rock was not observed in any of the other boreholes. • Groundwater levels were typically around 1.5 m bgl, which is consistent with the lake water level. • We recommend a site subsoil class of 'Class D – Deep or soft soil sites' be adopted for seismic design. • The site is located within the Rotorua Geothermal Field. Ground temperatures of 40°C were measured at depths ranging from 2 to 12.2 m bgl across the site. • The nearest known active fault is the Horohoro Fault approximately 5 km south of the site, due to the distance of the fault from the site, we expect the risk of fault rupture affecting the site to be low.
Liquefaction and lateral spread	• Following a design SLS seismic event, we expect liquefaction-induced land deformation of the existing ground and the associated risk to shallow and deep founded structures to be negligible. • Following a design ULS seismic event, we expect the majority of the site to be susceptible to minor to moderate, damage to buildings and infrastructure. In localised areas of the site liquefaction induced damage may be moderate to severe, in these zones liquefaction may cause substantial damage and disruption to buildings and infrastructure, and repair may be difficult or uneconomic in some cases. • Raised boardwalks, bridges and buildings may require specific ground improvement and/or foundation design to mitigate the effect of liquefaction induced settlement. This could incorporate a reinforced gravel raft below foundations or ground improvement such as mixing the in-situ soils with cement to form a 'mudcrete' raft. Laboratory and in-situ trials would be required at detailed design to confirm 'mudcrete' suitability and mix ratios. • Alternatively, deep foundations (piles) could be employed to transfer the building loads below the layers of soil with the greatest susceptibility to liquefaction. • The free face is not anticipated to exceed 1.5 m. Non-liquefiable soils are present to depths of greater than twice the free face height at each CPT location, and therefore the risk of significant lateral spreading is considered to be low.
Shallow foundations	• Shallow foundations could be a suitable option for structures on the redevelopment subject to site specific remediation (such as undercuts and/or ground improvement). • We expect that shallow foundations will be founded in either sand fill or very soft soils. These soils are expected to be highly susceptible to excessive static settlement and shear failure under structural loads. Any building load and boardwalk loads placed on these soils without some ground improvement or undercutting is likely to give rise to excessive differential static settlements. • To reduce excessive settlement and limit foundation bearing pressures on the subsoils undercutting and replacement with granular fill below the foundations could be considered. • Ground improvement such as mixing the soft soils with cement to form a 'mudcrete' could be an alternative to undercutting and replacement of soft soils. Laboratory and in-situ trials would be required at detailed design to confirm suitability and mix ratios. • Ground improvements such as soil/cement stabilisation may be a suitable option, but will be governed by the presence of any organic soils. • We expect any ground improvement foundations to be carried out below the groundwater table or within the lake and will likely require temporary works to retain the water such as cofferdam and dewatering to allow construction. • Differential settlements of structures could be managed by preloading the proposed building footprint to induce settlement prior to construction of footings.
Piled foundations	• Piled foundations can be a suitable option for structures on the redevelopment, subject to site specific design. Due to the sites geothermal location, the presence of soft/loose soils below the water table, we recommend adopting precast driven concrete piles, bored piles are not suitable for this site. • Piles would need to extend below compressible and/or liquefiable soil layers (Unit 2 and Unit 3) and embed in medium dense to dense sands and firm to stiff silts. Pile lengths could therefore be as much as 15 – 25 m below present ground levels depending on building requirements. • Vertical, lateral and uplift capacity of piles will need to consider the effects of loss of strength due to the presence of potentially liquefiable soils under seismic conditions and the effects of negative skin friction for post liquefaction ground settlement. • Geothermal conditions at the site mean that piles require specific detailed design to achieve any desired design life. Allowance should be made in the selection of the pile type for high soil temperatures and chemical attack. Consideration may also need to be given to the presence of geothermal aquifers and associated gases. • Pile driving criteria should be based on medium driving conditions due to the sensitivity and relatively low density of the founding materials, and to avoid crushing of the pumice gravels.

1 Introduction

1.1 General

Tonkin & Taylor Ltd (T+T) has been commissioned by Rotorua Lakes Council (RLC) to undertake a geotechnical assessment for the proposed redevelopment of the Rotorua Lakefront site.

This Geotechnical Interpretive Report (GIR) provides interpretation of geological, hydrogeological and geotechnical conditions based on information available from ground investigations and published information.

1.2 Project overview

The proposed lakefront redevelopment comprises a new boardwalk, concrete and grass terraces, car parking, bus parking, new road pavements, footpaths, play spaces and landscaping. A number of new buildings are also proposed, although these are not included in the current development package. The proposed development has been split into five stages (numbered 1 to 5) with stages 1 and 3 further divided into Stages 1, 1a, 3 and 3a.

The boardwalk and terracing are included within Stages 1, 1a and 3a. Stage 2 includes car and bus parking, and pavements. Play spaces are located within Stage 3. Future restaurants, café and kiosks are included within Stage 3a. Stage 4 includes further car parking and pavements. The Wharewaka and Waka Ama buildings, car and trailer parking are included within the Stage 5 works. Footpaths, paving and landscaping are included in all proposed stages. The masterplan and staging plan prepared by Isthmus are presented below in Figure 1.1 and Figure 1.2.

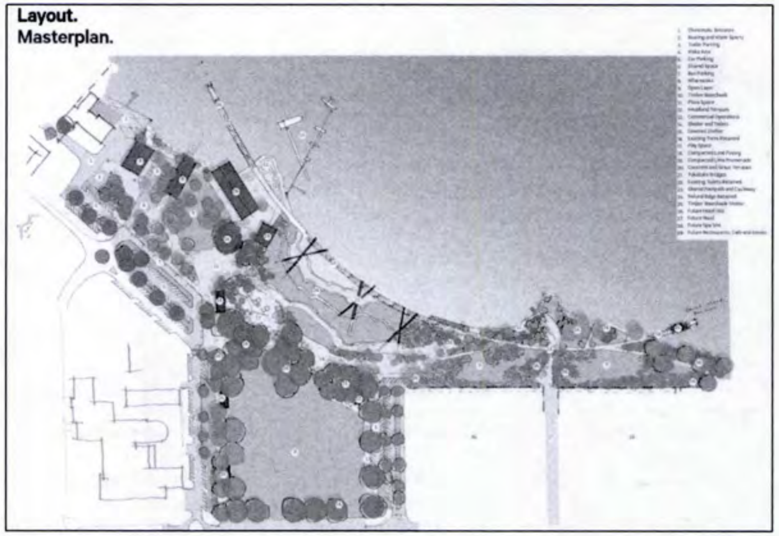

Figure 1.1 Proposed masterplan extracted from RLC Lakefront Redevelopment Developed Design for Stage 1 and 1a, dated August 2018.

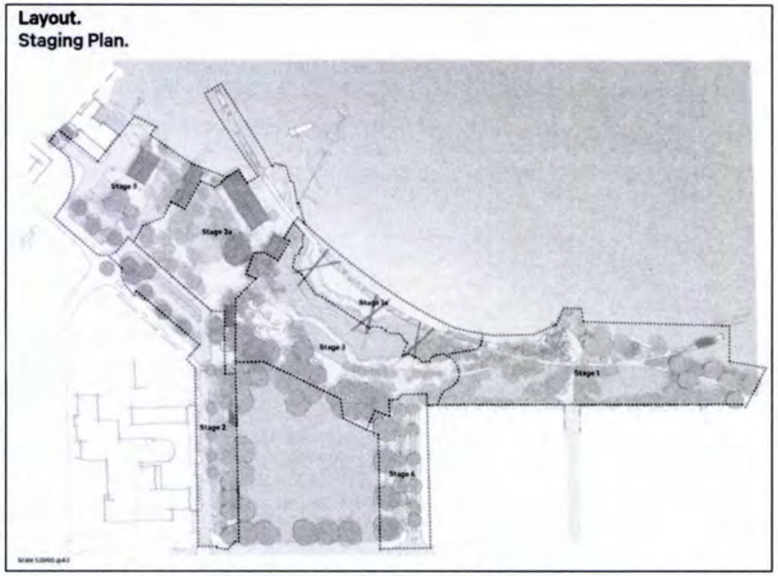

Figure 1.2: Proposed staging plan extracted from RLC Lakefront Redevelopment Developed Design for Stage 1 and 1a, dated August 2018.

1.3 Scope and purpose of this document

The purpose of this GIR is to support detailed geotechnical design of various structures associated with the RLC Lakefront redevelopment.

Our scope of works for this GIR includes:

- Assessment of ground conditions from the Stage 1 and Stage 2 ground investigations.
- Derivation of geotechnical design parameters.
- Assessments of seismic subsoil class and shaking hazard.
- Liquefaction susceptibility assessment, including estimation of liquefaction induced settlement and lateral spreading.
- Assessment of bearing capacity and settlement for shallow foundations.
- Assessment of vertical pile capacity.

2 Desk Study

2.1 Site description

The site is located on the lake margins at the southern end of Lake Rotorua. The extent of the proposed development is approximately defined by the red line shown on Figure 2.1.

The site is partly located on reclaimed land which we understand was constructed in the 1960s. The site is currently used as a grassed and brick paved recreational area with a series of existing timber piers and esplanade which borders the lake. A number of small timber kiosks are present on the site which are used as tour booking offices. Two timber pile piers and some smaller docking facilities extend, into the lake, from the esplanade. There are several underground storage tanks (USTs) beside the kiosks that contain fuel for boats and aircraft associated with the tourist operations.

Lakefront Drive and Oruawhata Drive runs east-west through the site, following the lake margin and a number of car parks are associated with these roads. A Scout hall is present on the eastern lakefront of the site, with a small timber piled jetty.

The lakefront water area is used for various mooring of commercial vessels and aircraft.

Figure 2.1 Site location plan (sourced from Land Information New Zealand, Crown Copyright Reserved).

2.2 Sources of factual information and drawings

Table 2.1: Information sources

Date	Name	Report/DWG no. + revision	Description
Reports			
October 2018	Tonkin & Taylor Ltd Stage 1 Geotechnical Factual Report	Final	Presents ground investigation data for onshore section of the Lakefront development
December 2018	Tonkin & Taylor Ltd Stage 2 Geotechnical Factual Report	Final	Presents ground investigation data for offshore section of the Lakefront redevelopment
August 1998	Tonkin & Taylor Ltd Feasibility study for Rotorua lake front development (1998)	Issued	GI as part of Feasibility study
Drawings			
1997	Bathymetry	Published	Scan of a figure from a unknown report
June 1998	Bathymetry	Issued	Bathymetry and site investigation plan for Rotorua Lake front pier project
September 2018	DML Topographic Survey	Prelim	
August 2018	Isthmus Masterplan	Prelim	Masterplan drawings
24/09/2018	Isthmus Demolition plan	Draft	Shows what will be removed from study area and the location of the lake front development
Other data			
1945-1996	Historical aerial photos	Published	Sourced from Retrolens Shows change in the study area over time

2.3 Site history

Aerial imagery sourced from http://retrolens.nz/ has been used to review the previous land use and site history. A summary of the site history and previous land use is presented in Table 2.2.

Table 2.2: Previous land use and site history

Date of historic photos	Description
Prior to 1941	Area undeveloped
Circa 1945	Minor land development with little to no building within study area. To the north west and south of the study area residential housing is observed.
Circa 1951	No change from 1945
Circa 1963	Reclamation of nearshore area and buildings
	Construction of buildings booking offices
	Construction of Jetties
	Construction of Scout hall
	Construction of Public Toilets
1981-1995	Boardwalk opened on lake front within study area
1995-2017	No significant change observed

2.4 Previous investigations

Previous investigations and assessment for a proposed pier and information centre were carried out by T+T in 1998. The investigations included 3 No. Cone Penetration Tests (CPT), 1 No. machine borehole and 9 No. Scala Penetrometer tests as part of their investigations. This report is referenced in Table 2.1.

2.5 Recent investigations

Recent geotechnical investigations, over two stages (Stage 1 and Stage 2) have been carried out by T+T to support this geotechnical assessment, these are presented in our Geotechnical Factual Reports (GFR) referenced in Table 2.1 of this report and provided in Appendix A. The borehole and CPT locations are presented in Figure 1, Appendix B.

2.6 Published geology

The GNS Science 1:250,000 published geological map of the area [REF 1] indicates that the site is underlain by lake sediments and alluvial deposits of the Tauranga Group overlying a mixture of undifferentiated rhyolite lavas. The location of the site in the context of the regional geology is presented in Figure 2.2.

Most of the lake sediments are volcanic ash derived and will be of rhyolitic composition which tends to weather to a sensitive silt.

The nearest known active fault is the Horohoro Fault approximately 5 km south of the site, faulting is discussed in later sections of this report.

The site is classified under the GNS soft ground map as within Zone D indicating Holocene deposits deposited in the last 10,000 years.

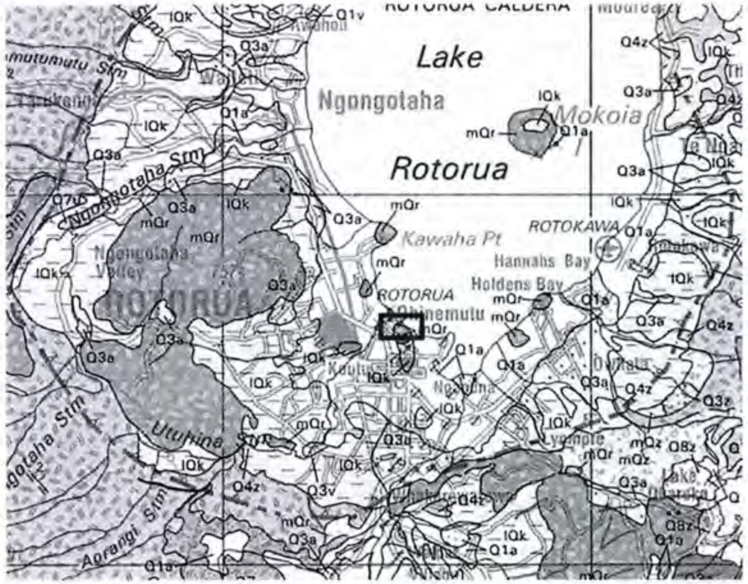

Figure 2.2: Geological setting with site location overlaid by red box.

2.7 Geomorphology and bathymetry

The existing topography of the area gently grades from RL 282 m in the southwest gently towards the lakefront, where the esplanade elevation is approximately RL 280.6 m.

The ground level near the visitor centre and tourist boat area to the northwest has a lake edge elevation typically around RL 281.2 m. The ground level near the existing Sea Scout hall to the east has a lake edge elevation typically above RL 279.5 m.

The recorded water level on the day of a T+T site visit on the 28 June 2018 was RL 280.0 m.

The lake bed level is at an RL of around 279.3 m near the esplanade and becomes gradually deeper to an RL towards the lake shelf approximately 350 m from the lakefront.

A channel immediately to the west of the existing jetties has been dredged to an RL of 278 m (approximately 1 m below surrounding lakebed level) to allow larger commercial boats to access the foreshore area.

2.8 Geothermal conditions

The site of the proposed development is located within the Rotorua Geothermal Field toward the northern end of the city.

The nearest known geothermal well in active use is within the QEII Hospital grounds to the east of the site. A number of other geothermal wells, both active and inactive, are close to the site.

Steam vents and fumaroles are present in the Government Gardens to the east of the site and Ohinemutu to the west.

3 Ground Model

3.1 General

The site was observed to comprise up to 2 m of sand fill overlying lakebed deposits of very soft to soft silts. Layers of loose to dense sands, gravels and firm to stiff silts underlie the soft silts. On the lakebed, very soft silts are present at lakebed level. Rhyolite rock was encountered in BH03 at around 14 m below ground level (bgl), this was not encountered at any other exploratory hole locations.

The geological units have been developed to characterise the materials encountered at the site and described in Section 3.1.1 to Section 3.1.8 and are summarised and in Table 3.1.

Geological sections showing the extent of the geological units along the boardwalk and through the site are presented in Appendix C.

Table 3.1: Summary of geological units

Geological Unit	Engineering Description (Stratigraphic Unit)	Typical thickness (m)	CPT cone resistance (qc) (MPa)	SPT N Value	Comments
Unit 1	Loose to medium dense Sand/Silty Sand (FILL)	1.5	1 - 6	N/A	Not observed offshore
Unit 2A	Very soft to soft Silt (Lake sediments)	3.5	0 - 0.4	0 - 3	Exposed at surface offshore
Unit 2B	Very loose to medium dense Gravel/sandy Gravel (Tauranga Alluvium)	2	2 - 10	2 - 12	
Unit 2C	Very loose to loose Sand (Tauranga Alluvium)	2	1 - 4	0 - 19	
Unit 3A	Firm to stiff Silt (Tauranga Alluvium)	2.5	1 - 2	0 - 19	
Unit 3B	Medium dense Sand (Tauranga Alluvium)	3	n/a	8 - >50	
Unit 4	Stiff Silt (Tauranga Alluvium)	5	n/a	0 - 32	
Unit 5	Weak slightly weathered Rhyolite	Only observed in BH03	n/a	n/a	

3.1.1 Loose to medium dense Sand - Fill (Unit 1)

A layer of loose to medium dense sand fill was observed to cover the majority of the lakefront area. The aerial photos and site records show the existing esplanade area was constructed on top of the lakebed in the 1960s.

Loose to medium dense sand fill was observed in boreholes BH01, BH02 and BH03 and inferred from CPT traces to depths of around 0.5 to 2.0 m bgl. The sands were generally logged as 'loosely packed' and 'well graded'. CPT cone resistance (qc) values generally ranged from 1 to 6 MPa indicating the sands are generally loose to medium dense.

Fill may be deeper below the existing roads and car parks that run through the site, as some undercuts to improve subgrade may have been carried out during previous works.

3.1.2 Very soft to soft Silt (Unit 2A)

Very soft to soft silt was observed in the majority of boreholes except BH04 and inferred from the majority of CPT traces (excluding CPT 01, 17, 18, 19 and 20, which were generally terminated at shallow depths due to refusal on hard stratum or high ground temperatures).

In the lakefront area (on land) Unit 2A was observed to underlie the Sand Fill (Unit 1). In the boreholes carried out on the lake (BH05 – BH08) the top of Unit 2A was observed at lakebed level.

Unit 2A was observed to range in thickness from 1 to 6.5 m (CPT07) with a typical thickness of 3.5 m. Standard Penetration Test 'N' values (SPT N) range from 0 to 3, with the majority of SPTs undertaken falling under the weight of the SPT hammer. CPT qc values generally range from 0.05 to 0.4 MPa for the unit.

The silt was observed to have a rapid dilatancy, indicating the material is extremely sensitive to disturbance and reworking.

Laboratory testing, presented in our GFRs for the site, reported the natural moisture content of the unit was around 200%, and based on organic content testing and borehole logs the material is not organic.

We expect that this unit is made up of weathered and transported rhyolitic tephra deposits. These deposits, when weathered are typically rich in the mineral halloysite which results in an open voided soil fabric creating a high water content. This is consistent with the high void ratios measured in the oedometer testing.

Some thin discontinuous sand lenses (less than 0.3 m) were observed within this unit.

3.1.3 Very loose to medium dense Gravel (Unit 2B)

Very loose to loose gravel and sandy gravel (Unit 2B) was observed to underlie Unit 2A across the majority of the site. Unit 2C (described below) underlies Unit 2B and Unit 2A across most of the site, however, in the western part of the site Unit 2C overlies Unit 2B.

The thickness of this unit was observed to range from 0.5 to 4.0 m with an average of 2 m. The gravel was logged as pumiceous and well graded.

SPT 'N' values in the unit ranged from 2 to 12 and CPT qc generally from 2 to 10 MPa, indicating the unit is very loose to loose.

3.1.4 Very loose to loose Sand (Unit 2C)

Very loose to medium dense sand was observed in the majority of boreholes and CPT traces to underlie Unit 2A and Unit 2B. This unit was observed to range in thickness from 1.0 to 4.5 m, with an average thickness of 2 m. The sand is generally pumiceous and well graded.

SPT 'N' values ranged from 0 to 19 but were typically around 1 to 8. CPT qc values generally ranged from 1 to 4 MPa.

3.1.5 Firm to stiff Silt (Unit 3A)

A layer of firm to stiff silt/sandy silt was observed across the site and underlies Unit 3A and Unit 3B. The thickness of the layer observed in the boreholes and inferred from the CPTs is 1 to 5 m, with an average of 2.5 m thickness.

CPT qc values for the unit generally ranged from 1 to 3 MPa, with an average of 1.2 MPa. SPT N values ranged from 2 to 19, but were typically around 2 to 10. No increase in SPT N value was observed with depth.

3.1.6 Medium dense Sand (Unit 3B)

A layer of medium dense to dense sand was observed below Unit 3A, this unit ranges in thickness from 0.5 to 5.0 m with an average of 3.0 m. SPT N values generally ranged from 8 to >50 and CPT qc from 2 to 4 MPa.

3.1.7 Stiff to very stiff Silt (Unit 4)

Stiff to very stiff silt/clay layers were observed at depth in the majority of boreholes (except for BH01 and BH04). SPT N values ranged from 0 to 32 in this unit.

3.1.8 Rhyolite Rock (Unit 5)

Weak, slightly weathered rhyolite rock was observed in BH03 at a depth of 14 m bgl. The rhyolite was logged weak and slightly weathered. The rhyolite rock level is expected to be variable across the site.

3.2 Groundwater

Groundwater levels were measured 1.0 to 2.1 m bgl in the borehole and CPT locations and were typically 1.5 m bgl. The groundwater level at the time of our investigations was observed to have a slight gradient toward the lakefront. Groundwater RLs ranged from 279.3 m to 280 m in the boreholes during our July to August investigations, the lake level was recorded at RL 279.9 m during this period, indicating measured groundwater levels are relatively consistent with the lake water level.

A summary of the groundwater levels and lake levels from the boreholes are presented below.

Table 3.2: Groundwater levels following investigations

Investigation ID	Date recorded	Ground level (m) RL(1)	Mean static groundwater level (m) RL(3)	BOPRC Lake Level (m) RL(2)	Depth below ground level (m)(3)
BH01	30/07/2018	281.7	280.2	279.92	1.5
BH02	31/07/2018	280.6	279.3	279.91	1.3
BH03	01/08/2018	281.2	280.0	279.91	1.2
BH04	02/08/2018	281.4	280.0	279.91	1.4

(1) RL was inferred from contour maps.

(2) Source: Bay of Plenty Regional Council http://monitoring.boprc.govt.nz. Rounded to two decimal places. Readings taken at midday.

(3) Levels presented above in Table 3.2 supersede those presented in Table 3.3 of Rotorua Lakefront Redevelopment - Stage 1 Geotechnical Factual Report (v01) dated October 2018.

3.3 Geothermal conditions

Ground temperature was recorded continuously during penetration of the CPT probe. Temperature was also measured between runs in the boreholes. The temperature readings are recorded on the logs and CPT data, presented in our GFRs referenced in Table 2.1.

All CPTs terminated at temperatures of around 40°C at depths ranging from 2 to 12.2 m bgl but were generally in the order of 9 to 12 m bgl. Generally, CPTs terminated at shallower depths in the western portion of the site.

Boreholes recorded temperatures of 52 – 72 °C. The maximum temperature measured was in BH04 at 3.45 m bgl. This reading is likely to be cooler than actual ground temperature due to the use of drilling fluid and water.

Generally ground temperatures were higher than ambient ground temperatures from 1 – 2 m bgl.

No gas upflows were observed during the investigations.

Nowhere in the study area were the surface effects of geothermal heating identified; for example steaming ground, stressed or dead vegetation, or barren ground with crusts of sulphur or salt crystals.

4 Geotechnical Parameters

4.1 Recommended geotechnical parameters

Our recommended geotechnical parameters for the various geological units observed at the site are presented in Table 4.1 and Table 4.2. These are inferred from the in-situ and laboratory testing, correlations from CPT and SPT data and our experience in similar soils.

A range of geotechnical design values has been provided to reflect the variability in drained and undrained strength of the geological units over the site. A location specific assessment of these parameters and geological unit thickness should be undertaken at detailed design stage prior to selecting design values.

Table 4.1: Geotechnical parameters

Geological Unit	Engineering Description (Stratigraphic Unit)	Unit weight γ (kN/m³)	Undrained shear strength Su (kPa)	Effective friction angle Φ' (degrees)	Effective cohesion c' (kPa)
Unit 1	Loose to medium dense Sand/Silty Sand (FILL)	16 - 18	-	30 - 35	0
Unit 2A	Very soft to soft Silt (Lake sediments)	16 - 18	5 - 25	24 - 28	0 - 2
Unit 2B	Very loose to medium dense Gravel/sandy Gravel (Tauranga Alluvium)	16 - 18	-	35 - 38	0
Unit 2C	Very loose to loose Sand (Tauranga Alluvium)	16 - 18	-	30 – 35	0
Unit 3A	Firm to stiff Silt (Tauranga Alluvium)	16 - 18	40 - 80	26 - 30	3 - 6
Unit 3B	Medium dense Sand (Tauranga Alluvium)	16 - 18	-	32 - 36	0
Unit 4	Stiff Silt (Tauranga Alluvium)	16 - 18	80 - 120	28 - 30	5 - 10

Table 4.2: Settlement parameters

Soil	Es (MPa)	mv (m2/MN)	Cv (m2/year)	e0	Cα
Unit 1	5 - 15	-	-	Secondary compression settlement considered negligible	
Unit 2A	0.5 - 1	1.0 – 2.0	5 - 10	5.25	0.016
Unit 2B	15 - 25	-	-	Secondary compression settlement considered negligible	
Unit 2C	5 - 15	-	-		
Unit 3A	10 - 20	0.05 – 0.1*	2 – 6*		
Unit 3B	20 - 30	-	-		
Unit 4	15 - 20	-	-		

E_s	Elastic Modulus (medium to large strain range) from correlation with SPT and CPT data
M_v	Coefficient of volume compressibility (Stress range: 15 to 60 kPa)
C_v	Coefficient of consolidation derived from 1D consolidation test results (Stress range: 15 to 60 kPa)
C_c	Compressibility index derived from 1-D consolidation test results
e_0	In situ initial void ratio derived from 1-D consolidation test results
C_α	Secondary Compressibility index = 0.04 x Cc (Mesri & Godlewski, 1977)
γ	Bulk unit weight

*inferred from CPT results and published values

5 Geotechnical Assessment

5.1 General

The key geotechnical considerations for the lakefront redevelopment include the following:

- Liquefaction trigger susceptibility and consequential effects, principally settlement and lateral spread.
- The presence of compressible soils and the risk of settlement due to foundation and fill loads.
- Foundation bearing capacity.
- Site subsoil classification.

These considerations are addressed in the following sections of this report.

5.2 Seismic shaking hazard

5.2.1 Site subsoil class

Based on the presence of very soft cohesive soils and very loose granular deposits in accordance with NZS 1170.5:2004 [REF 2], a site subsoil class of 'Class D – Deep or soft soil sites' should be adopted for seismic design.

5.2.2 Ground shaking hazard

Minimum seismic design requirements are adopted from NZS 1170.0:2002 [REF 3], summarised below in Table 5.1.

Table 5.1: Minimum seismic design requirements (NZS 1170)

Design Earthquake Case	**Simplified Description**
Serviceability Limit State (SLS)	In the event of an SLS earthquake, the design shall maintain serviceability as originally intended without requiring repair
Ultimate Limit State (ULS)	In the event of an ULS earthquake, the design shall avoid collapse of the structural system and parts representing a hazard to human life, and shall maintain operation of systems necessary for emergency building evacuation.

Seismic design parameters have been calculated in accordance with the NZTA Bridge Manual [REF 4] and NZS 1170.5:2004. These parameters are presented in Table 5.2.

Table 5.2: Seismic design parameters

Design Case	**Earthquake return period (years)**	**Peak Ground Acceleration (PGA), g**	**Effective magnitude, M_{eff}**
SLS Event	1 in 25	0.075	6.0 for all cases
ULS Event	1 in 500	0.300	

Note: PGA and effective magnitude has been assessed based on NZS1170 and NZTA Bridge Manual SP/M/022 Third Edition, Amendment 2 for the following:

Building design life	50 years - assumed
Building importance level	IL2 (NZS 1170.0:2004, Table 3.2)
Return period factor, Ru	1.0 for 500 yr, 0.5 for 100 yr and 0.25 for 25yr return period (NZS 1170.5:2004, Table 3.5)
Subsoil class	D (Deep soil) – refer Section 5.2.1 of this report.
Return period PGA coefficient, $C_{0,1000}$	0.39 (Bridge Manual Table 6A.1)
Site subsoil class factor, f	1.0 (Bridge Manual Section 6.2)
PGA	$= C_{0,1000} \times Ru/1.3 \times f \times g$ (Section 6.2 of Bridge Manual)
Effective Magnitude, M_{eff}	6.0 for 500 yr, 100 yr and 25 yr return period (Bridge Manual Table 6A.1)

5.3 Liquefaction and lateral spread

5.3.1 General

Liquefaction is a phenomenon which can occur in loose, saturated, non-plastic soils which are subjected to an applied cyclic load, such as that generated from earthquake shaking. Liquefaction can cause significant damage to land, buildings and infrastructure (e.g., differential settlement, sand boils and lateral spreading).

A liquefaction assessment has been undertaken using the methodology outlined in Appendix A of Tonkin & Taylor Ltd (2015) [REF 5] and National Academies of Sciences, Engineering, and Medicine [REF 6] and comprises the following assessments:

- Soil susceptibility to liquefy and depth to groundwater.
- Liquefaction triggering from earthquake shaking.
- Assessment of consequences of liquefaction.
- Assessment of lateral spreading.

A summary of potential consequences of liquefaction is provided in Table 5.3.

Table 5.3: Potential consequences of liquefaction

Phenomenon	Description
Differential settlement	Uneven settlement of the ground surface which can cause damage to building foundations, services and roads.
Sand and water ejected to the surface (sand boils)	This exacerbates differential settlement, can result in damage to paved and other ground surfaces, reduced clearances under buildings, ingress, and block buried pipes, etc.
Reduced support to foundations bearing above the liquefied soil	Bearing capacity of the soil could be reduced resulting in subsidence of building foundations.
Buoyancy effects	Liquefaction can result in upward movement (floatation) of manholes, tanks and other buried vessels.
Lateral spread	Land above the liquefied soil layer moving either down slope or toward a free edge such as a lake. This lateral movement can cause severe damage to buildings and infrastructure.

5.3.2 Susceptibility

The soils across the site comprise predominantly sand and silt (Unit 2, Unit 3 and Unit 4) and some areas of fill (Unit 1). These soils are generally considered to have some potential to liquefy and have been assessed for liquefaction susceptibility.

The liquefaction assessment is sensitive to the estimated groundwater level. Soil layers are only analysed if they are below the estimated groundwater level and soil layers above the estimated groundwater level are not considered liquefiable in the analysis. A regional groundwater level of 1.5 m bgl has been adopted for our assessment based on our recent investigations.

5.3.3 Assessment depth

Due to high ground temperatures none of the CPTs reached their intended target depth of 20 m. The majority of the CPTs were terminated between 9 – 12 m and 3 of the CPTs terminated less than 2.5 m. Based on observed building performance in Christchurch during the 2010-2011 earthquakes only the liquefaction in the top 10 m affected the building performance.

Liquefaction at greater depths had limited to no effect on building performance (Tonkin & Taylor Ltd, 2015) [REF 5]. However, because any new structures on this development will have their own unique characteristics, future assessments may require a deeper zone of assessment to be considered for some buildings.

For this size and scale of project the majority of the CPT investigation depths are considered suitable for the assessment. In areas where CPTs terminated at shallow depths (generally less than 9 m) we have considered the broader ground model for the site, inferring susceptibility from adjacent CPTs.

5.3.4 Liquefaction triggering

The CPTs have been assessed for liquefaction triggering and potential consequence using in-house software which utilises the triggering methods recommended by Boulanger and Idriss (2014) [REF 7].

The liquefaction triggering has been assessed for SLS and ULS design seismic events.

We have assessed the site based on three indicators of liquefaction vulnerability. These indicators are detailed below, and their performance thresholds described in Table 5.4.

- **Liquefaction Severity Number (LSN)**, an indicative measure of the liquefaction induced damage expected at that location.
- **Settlement (S_{v1d})**, one dimensional vertical settlement, calculated using Zhang et al. (2002), is indicative of the approximate amount of expected settlement at the ground surface.
- **Non-liquefied Crust Thickness**, the depth to the first significant liquefied layer. Where a crust thickness is a minimum of 3 m, the effects of liquefaction should be largely mitigated at ground surface and will likely have minor consequences on shallow foundations. This expected ground performance is based on published theories (Ishihara, 1985; Zhang et al, 2002) [REF8, 9] as well as experience gained from the 2010-2011 Canterbury Earthquake Sequence.

Table 5.4: Summary of liquefaction vulnerability indicators

Expected liquefaction vulnerability	LSN	S_{v1d}	Non-liquefied crust thickness
None to minor	0 – 16	0 – 25 mm	>3 m
Moderate	16 – 25	25 – 100 mm	2 – 3 m
[illegible]	[illegible]	[illegible]	[illegible]

5.3.4.1 The effect of pumiceous soils

Various amounts of pumice was observed in the soils within the project area which is consistent with similar Tauranga Group soils found elsewhere in the Bay of Plenty. The liquefaction potential of pumice soils is typically overestimated using penetrative investigation techniques such as CPTs as the pumice grains are susceptible to crushing during testing, (Liu, Orense, & Pender, 2015) (MBIE, 2016) [REF 10, 11].

Our assessment is based on CPTs, which are an industry standard approach for liquefaction assessment. There are currently no reliable industry standard methods for quantifying the amount of over-prediction in pumiceous soils. However, advanced specialist investigation methods could be used to quantify the higher resistance to liquefaction for the pumice soils compared to ordinary soils. However, in this instance, we do not consider this advanced testing to be necessary.

5.3.5 Liquefaction results

The output of the liquefaction analyses are presented in Appendix D.

For a SLS seismic event soils at the site are considered to have a negligible risk of liquefaction.

In the ULS seismic case, estimated free field settlements range from around 35 mm to 140 mm and LSN values ranging from 11 to 40 were estimated. Typically free field settlements are in the order of 50 – 100 mm across the site, at CPT01 in the western extent of the development 145 mm of free field settlement has been estimated.

A non-liquefied crust thickness ranging from 1.5 m to 5.8 m is expected to be present under a ULS event. The crust thickness is generally greater than 3.5 m along the northern section of the site and typically less than 3.5 m along the southern section of the site.

A plan showing liquefaction vulnerability and estimated free field settlement for each CPT location is presented in Appendix E.

5.3.6 Liquefaction consequences

For this development we expect the principal liquefaction consequence to be uneven settlement of the ground surface which can cause damage to building foundations, services and roads.

Across the majority of the site liquefaction induced damage may be minor to moderate, and in some localised areas in the western portion of the site may be severe following a ULS event.

In areas of minor to moderate liquefaction damage, it is anticipated that liquefaction will cause moderate but typically repairable damage to buildings (where liquefaction is not addressed during detailed design).

In areas of moderate to severe liquefaction, damage and disruption to buildings and infrastructure is anticipated. Repair may be difficult or uneconomic in some cases. Damage may be substantially less, and more likely to be repairable, where liquefaction is addressed during design (e.g. enhanced foundations and robust infrastructure detailing).

A plan showing liquefaction vulnerability for a ULS event is presented in Appendix E.

Mitigation for liquefaction induced damage to structures is discussed in further detail in Section 6.0 of this report.

5.3.7 Lateral spread

Lateral spreading is a phenomenon whereby land above a liquefied soil layer moves either down slope or towards a free edge such as a stream channel or lakes edge, resulting in potentially large lateral and differential ground displacements.

The site is adjacent to Lake Rotorua and it has been considered in our lateral spread assessment during a ULS event. The height difference between existing lake bed level and ground level is approximately 0.5 m in the east of the development and up to 1.5 m in the west of the development, this height has been adopted as the 'free face' in our assessment. The free face is the result of a series of man-made retaining structures along the lake shore. The lakebed gradient is very shallow (less than 1V:20H) and therefore is not considered to pose a lateral spread risk, due to the absence of any notable free face.

The risk of lateral spreading has been assessed for the site based on the presence of significant liquefiable layers identified in CPTs from the liquefaction analysis over the zone equal to two times the height of the free face.

Non-liquefiable soils are present to depths of greater than twice the free face height at each CPT location, and therefore the risk of lateral spread is considered to be very low.

5.4 Fault rupture

The nearest known active fault is the Horohoro Fault approximately 5 km south of the site, due to the distance of the fault from the site, fault rupture is not expected to be an issue.

5.5 Static settlement of shallow foundations

We understand that it is proposed to build restaurants, café and kiosks within Stage 3a of the development and a boardwalk within Stages 1 and 1a. The proposed boardwalk, adjoining footbridges and any new buildings at the site will require shallow and/or deep foundations design.

One of the key geotechnical considerations for construction of the boardwalk and new building foundations at the site is settlement of the near surface compressible soils. These soils are expected to be susceptible to settlement due to building induced loading.

A 1.5 to 2.0 m thick layer of fill has been placed across the majority of the site some 50 to 60 years ago. We consider that the settlements of the soft soils due to the fill surcharge are likely to be largely complete. However, some small creep settlements within the very soft silts may still occur.

In this section we have carried out preliminary static settlement estimates for the boardwalk area.

For the boardwalk foundations we have adopted some typical foundation dimensions, layouts and loads based on internal discussions with our structural engineers.

The ground settlements discussed in the following sections are static only and exclude liquefaction-induced settlements predicted for the site under seismic events.

The settlements presented in this section are indicative only and are not specific to any particular location, structure or foundation load. Estimates of settlement for shallow foundations will need to be assessed at detailed design stage.

5.5.1 Typical loading

We have adopted typical loads and dimensions to provide an assessment of settlement, these typical dimensions are presented in Table 5.5. Loads outside of the applied pressures below will likely need specific improvements to satisfy bearing capacity.

The foundation dimensions and loads presented below are typical and must be reviewed at detailed design

Table 5.5: Typical shallow foundation load and dimensions

Foundation	Typical Size	Typical Bearing Pressure (kPa)
Typical pad foundation A	3.0 m x 3.0 m	15 kPa
Typical pad foundation B	5.0 m x 4.0 m	20 kPa

5.5.2 Settlement methodology

We have estimated total settlements for typical foundation types and expected bearing pressures for structures founded on the typical soil profile shown in Table 3.1.

We note that in the offshore section of the boardwalk Unit 1 is not present and the very soft silts (Unit 2A) are at lake bed level, this has been considered in our assessment.

For the boardwalk foundations we have assessed Unit 2A at surface.

The upper very soft to soft silts (Unit 2A) are highly compressible and are expected to undergo settlement by primary consolidation under the weight of the new building loads. Secondary compression settlement, which occurs under a constant effective stress, is likely to occur within the compressible silt layer underlying the site as a result of particle rearrangement and creep. The layers of reclamation fill, loose to medium dense sand (Unit 1, 2B and 2C) are likely to undergo immediate, elastic settlement under future building and boardwalk loads.

Immediate settlements have been estimated using elastic theory. Consolidation settlements were estimated using the commonly adopted Terzaghi's 1 dimensional consolidation theory.

To model a number of different loading scenarios and foundation sizes the assessment was carried out using Rocscience Settle3D v4.0.

Settlement results for various load cases are presented in the following sections. We have given a settlement range to account for the variability in thickness of soft compressible layers across the site.

5.5.3 Settlement results

Estimates of settlements are presented in Table 5.6 below:

Table 5.6: Settlement estimates

Type of Foundation	Typical Size	Loading Pressure (kPa)	Predicted Settlement
Boardwalk Foundations (founded on Unit 2A)			
Typical pad foundation A	3.0 m x 3.0 m	15 kPa	30 - 50 mm
Typical pad foundation B	5.0 m x 4.0 m	20 kPa	50 - 80 mm

5.5.4 Settlement discussion

The predicted settlements above can be treated as differential settlements, this is due to the variable thickness of the underlying soft soils.

It is expected that 40% of the anticipated settlement stated above will be immediate, with the remainder being primary consolidation settlement. We expect the remaining primary consolidation settlement to occur over a period of around three months to reach 90% of the total settlements stated above.

The magnitude of settlements will be highly dependent on building size, type, load and locations and will need to be addressed at detailed design stage.

Total and differential settlement mitigation (such as undercuts or ground improvement) beneath shallow foundations is likely to be necessary for the site or deep foundations could be adopted. This is discussed and assessed further in Section 6.2 of this report.

5.6 Shallow foundation bearing capacity

We have estimated ultimate bearing capacity for shallow foundations adopting the soil profile in Section 3.1.

We expect an ultimate geotechnical bearing capacity of 50 – 100 kPa will be available for shallow foundations. The above bearing capacity does not take in to consideration any foundation shape factors.

The above bearing capacity estimates do not take in to consideration the effects of strength loss of the underlying potentially liquefiable layers during a seismic event, this will need to be assessed at detailed design stage of the project.

The soft soils will likely need ground improvement or undercutting to achieve desired bearing capacities if shallow foundations are adopted, these are discussed further in Section 6.0.

5.7 Piled foundations

Piled foundations could be considered for support of building and boardwalk structures as an alternative to shallow foundations subject to the following considerations.

Due to the depth of the water table and the geothermal conditions, bored piles are not considered to be appropriate. Driven concrete piles are likely to be the most appropriate pile option, as steel may suffer excessive corrosion and timber piles are not expected to meet the required design life.

Pile design parameters have been derived from the CPT data using the Bustamante and Gianeselli method which involves analysing the CPT cone tip resistance to determine end bearing and skin friction values. At depths below the CPT data, correlations with SPT N value were used to estimate pile capacities.

Table 5.7: Typical Pile Capacity for driven concrete piles

Geological Unit	Engineering Description	Geotechnical ultimate skin friction capacity (kPa)	Geotechnical ultimate end bearing capacity (kPa)
Unit 1	Loose to medium dense Sand/Silty Sand (FILL)	n/a	n/a
Unit 2A	Very soft to soft Silt	5	n/a
Unit 2B	Very loose to medium dense Gravel/sandy Gravel	70	n/a
Unit 2C	Very loose to loose Sand	30	n/a
Unit 3A	Firm to stiff Silt	50	900
Unit 3B	Medium dense Sand	50	1500
Unit 4	Stiff Silt	75	1650
Unit 5	Weak slightly weathered Rhyolite	500	5000

To provide sufficient capacity the piles would need to extend below compressible and/or liquefiable soil layers (Unit 2 and Unit 3) and embed in medium dense to dense sands and firm to stiff silts. Pile lengths could therefore be as much as 15 – 20 m below present ground levels.

5.8 Expansive soils

Based on the presence of sands and low plasticity silts present at the site the risk of encountering expansive soils is very low.

5.9 Geothermal hazard

The site is located in a geothermally active area that creates corrosive ground conditions.

Samples of groundwater were taken from BH01 to BH04 (inclusive) for pH and sulphate testing. The results from the laboratory testing indicate that groundwater in the vicinity of the site is generally below the thresholds for chemical attack to concrete in accordance with NZ 3101. We note that the groundwater samples from the boreholes may be affected by flushing water added during machine drilling.

Elevated ground temperatures were observed at typically 2 m below existing ground level. In the west of the site high ground temperatures (in the order of 50 – 70 °C) are expected at relatively shallow depths (3 – 5 m)

6 Geotechnical Recommendations

This section provides recommendations for redevelopment of the Rotorua Lakefront based on the ground investigations and geotechnical assessment carried out for the site.

6.1 Liquefaction and lateral spread

Following a design SLS seismic event liquefaction-induced land deformation of existing the ground and the associated risk to shallow and deep founded structures is expected to be negligible.

Following a design ULS seismic event, we expect the majority of the site to be susceptible to minor to moderate liquefaction induced damage. In localised areas liquefaction induced damage may be moderate to severe, liquefaction causes substantial damage and disruption to buildings and infrastructure, and repair may be difficult or uneconomic in some cases.

It is recommended that consideration is given to implementing liquefaction mitigation measures, as part of any detailed design works for any future development of this site.

Raised boardwalks, bridges and buildings may require specific ground improvement and/or foundation design to mitigate the effect of liquefaction induced settlement, depending on the performance criteria.

Ground improvement such as soil-cement mixing to form a mudcrete raft may be a suitable ground improvement option, subject to laboratory and in-situ trials at detailed design to confirm suitability and mix ratios.

Alternatively, deep foundations (piles) can be employed to transfer the building loads below the layers of soil with greatest susceptibility to liquefaction. Deep foundations in potentially liquefiable soils are discussed further in the following sections.

Non-liquefiable soils are present to depths of greater than twice the free face height at the lake edge, and therefore the risk of lateral spread is expected to be very low. Any significant changes to the landform which may create a free face, shall be assessed for lateral spread at detailed design.

6.2 Shallow foundations

6.2.1 General

Future buildings are expected in Stage 3a of the redevelopment and a new boardwalk in Stages 1 and 1a. Shallow foundations may be a suitable option for structures on the redevelopment subject to localised ground improvement.

We expect that shallow foundations will be founded in either sand fill or very soft soils. These soils are expected to be highly susceptible to excessive settlement and shear failure under structural loads. Any building load and boardwalk loads placed on these soils is likely to give rise to excessive differential settlements and potential bearing failure. Accordingly, careful design of foundations will be required to avoid unsatisfactory foundation performance.

6.2.2 Static settlement and bearing capacity

We expect an ultimate geotechnical bearing capacity of 50 – 100 kPa will be available for shallow foundations. The above bearing capacity does not take in to consideration any foundation shape factors. The above bearing capacity estimates do not take in to consideration the effects of strength loss of the underlying potentially liquefiable layers during a seismic event, this will need to be assessed at detailed design stage of the project.

For shallow foundations, with relatively light loadings, partial undercutting may be appropriate to mitigate effects of settlement and foundation bearing pressures. Undercutting of the existing soil to depths in the order of 2B (twice the depth and width of the proposed pad dimension) and replacement with compacted granular fill (such as lightweight pumice) may be required.

For shallow foundations where high loads are anticipated we recommend the following options are considered at detailed design to mitigate differential settlement:

- Partial undercut of the existing soil to depths in the order of 1.0 – 1.5 m and replacement with a geogrid reinforced gravel raft could be considered. This would spread loads and reduce bearing pressures, on underlying in situ soils.
- Soil cement mixing could be carried out to improve the soft silts and form a mudcrete raft. Full scale trials would need to be carried out at detailed design to confirm the suitability of this method.
- Differential settlements could be managed by preloading the proposed building footprint to induce settlement prior to construction of footings.
- Adoption of load balancing by removal of existing soils and replacement with layers of lightweight polystyrene blocks may be effective, but consideration of groundwater and geothermal conditions would need to be given.
- Deep foundations (piles) can be employed to transfer the building loads below the layers of soil susceptible to excessive settlement.

Any undercutting works to improve foundations are expected to be carried out below the water table or within the lake. Undercutting works will likely require temporary works to retain the water such as cofferdam and/or dewatering to allow construction.

6.2.3 Expansive soils

Based on the presence of sands and low plasticity silts at the site the risk of encountering expansive soils is very low.

6.3 Piled foundations

Piles would need to extend below compressible and/or liquefiable soil layers (Unit 2 and Unit 3) and embed in medium dense to dense sands and firm to stiff silts. Pile lengths could therefore be as much as 15 – 25 m below present ground levels depending on building requirements. Piles would need to give consideration to the variable rock level to be relied upon for end-bearing capacity.

Vertical, lateral and uplift capacity of piles will need to consider the effects of loss of strength due to the presence of potentially liquefiable soils under seismic conditions and the effects of negative skin friction (downdrag) for post liquefaction ground settlement.

Geothermal conditions at the site mean that piles require specific detailed design to achieve any desired design life. Allowance should be made in the selection of the pile type for high soil temperatures and chemical attack. Consideration may also need to be given to the presence of geothermal aquifers.

Potentially liquefiable layers are present between 1.6 and 9.0 m, negative skin friction effects may need to be considered for all soils at and above liquefiable zones. Detailed pile design will need to address liquefied strength of the soils susceptible to liquefaction. We note that Rhyolite rock was encountered at 14 m bgl in BH03, which may limit the pile embedment through this area, Rhyolite rock was not encountered in other boreholes at the site.

Soft compressible soils are present on the site and negative skin friction effects of consolidation settlement due to surface surcharging (fill or floor loads) must be considered at detailed design stage.

Reinforced concrete driven piles are considered to be the most suitable due to high groundwater levels, the presence of the very soft/loose upper soils and geothermal conditions.

Pile driving criteria should be based on medium driving conditions due to the sensitivity and relatively low density of the founding materials, and to avoid crushing of the pumice gravels.

The lateral carrying capacity of the pile will likely need to be addressed at detailed design to consider actions such as wave loading.

6.4 Geothermal hazard

The site is located in a geothermally active area that creates corrosive ground conditions.

The results from the laboratory testing indicate that groundwater in the vicinity of the site is generally below the thresholds for chemical attack to concrete. However, the site is located in a known geothermally active area that can create corrosive ground conditions.

It is therefore recommended that all concrete in contact with the ground should be designed for chemical exposure category XA2, in accordance with NZ3101.

Concrete design should include the effects of potentially corrosive soils and embedment of steel reinforcing within shallow and deep foundations will also need to be considered to ensure that the steel has an adequate cover of concrete.

Certain considerations around the geothermal hazard need to be followed at detailed design stage, these include (but are not limited to):

- Such standards comprise a minimum clearance between the floor and the ground and the use of floor liners. These allow adequate ventilation beneath the structures and prevent upwards migration of potentially harmful geothermal gases through the floor.
- Geothermal issues will also have to be considered when determining the depth and construction materials of foundations and pipework for any development.
- Wastewater and stormwater must also be disposed of in a manner that minimises the effects on the geothermal field (e.g. soak-holes may initiate surface geothermal activity) and in ground ducts and wiring need to be heat tolerant.
- Digging of any soak-holes may reach boiling geothermal fluids and could vent gases and steam to the surface. Such gases would be corrosive to building floors and in-ground footings. They would also create strongly acidic ground waters and condensation. Shallow soak-holes may also vent poisonous geothermal gases.
- Piled foundations may create conduits for geothermal gases and fluids which would require consideration for the construction works and the permanent structures integrity.
- If piles foundations are to be used then driven rather than bored piles would be preferable. Driven piles would inhibit the venting of geothermal gases during their construction.
- The use of geothermal membranes (heat resistant and gas proof) beneath slab foundations.

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7 References

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8 Applicability

This report has been prepared for the exclusive use of our client Rotorua Lakes Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

During excavation and construction, the site should be examined by an engineer or engineering geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. We would be pleased to provide this service to you and believe your project would benefit from such continuity. However, it is important that we be contacted if there is any variation in subsoil conditions from those described in the report.

Tonkin & Taylor Ltd

Report prepared by: Authorised for Tonkin & Taylor Ltd by:

Peter Molyneaux
Geotechnical Engineer

Lance Partner
Project Director

Report reviewed by:

Craig Davanna
Senior Geotechnical Engineer

11-Feb-19
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Appendix A: Factual reports

- **Tonkin & Taylor Ltd, Stage 1 Geotechnical Factual Report (October 2018)**
- **Tonkin & Taylor Ltd, Stage 2 Geotechnical Factual Report (December 2018)**

REPORT

Tonkin+Taylor

Rotorua Lakefront Redevelopment

Stage 1 Geotechnical Factual Report

Prepared for
Rotorua Lakes Council
Prepared by
Tonkin & Taylor Ltd
Date
October 2018
Job Number
1007467.1000.v01

Exceptional thinking together
www.tonkintaylor.co.nz

Document Control

Title: Rotorua Lakefront Redevelopment, Stage 1 Geotechnical Factual Report					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
12/10/2018	01	Final Issue	P Molyneaux	C Davanna	L Partner

Distribution:

Rotorua Lakes Council	1 copy (electronic)
Tonkin & Taylor Ltd (FILE)	1 copy (electronic)

Table of contents

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1 Introduction

Tonkin & Taylor Ltd (T+T) has been commissioned by Rotorua Lakes Council (RLC) to undertake a geotechnical investigation for a proposed redevelopment of the Rotorua Lakefront site.

The study covers two stages; Stage 1 being a geotechnical investigation on the lakeshore side (onshore), and Stage 2 being a geotechnical investigation within Lake Rotorua itself (offshore). This report documents the outcome of Stage 1 only and comprises of factual reporting of the onshore geotechnical investigation and associated laboratory testing for the Stage 1 investigation.

2 Site Description

The site is located on the lake margins at the southern end of Lake Rotorua. The extent of the proposed development is approximately defined by the red line shown on Figure 2.1.

The site is currently used as a grassed and brick paved recreational area with an existing esplanade which borders the lake. A number of small timber framed booking offices are located on the esplanade. There are several underground storage tanks (USTs) beside one of these offices that contain fuel for boats and aircraft associated with the tourist operations.

Lakefront Drive and Oruawhata Drive runs east-west through the site, following the lake margin and a number of car parks are associated with this road. Three jetties are present on the western part of the site and timber frame steel clad Scout hut on the east, with a small timber piled jetty.

The lakefront water area is used for various mooring of commercial vessels and aircraft.

Figure 2.1 Site location plan (Source: Rotorua Lakes Council GIS).

2.1 Proposed development

The proposed lakefront redevelopment comprises a new boardwalk, concrete and grass terraces, car parking, bus parking, new road pavements, footpaths, play spaces and landscaping. A number of new buildings are also proposed, although these are not included in the current development package. The proposed development has been split into five stages (labelled 1 to 5) with Stages 1 and 3 further divided into Stages 1, 1a, 3 and 3a.

The boardwalk and terracing are included within Stages 1, 1a and 3a. Stage 2 includes car and bus parking and pavements. Play spaces are located within Stage 3. Future restaurants, café and kiosks are included within Stage 3a. Stage 4 includes further car parking and pavements. The Wharewaka and Waka Ama buildings, car and trailer parking are included within the Stage 5 works. Footpaths, paving and landscaping are included in all proposed stages. The masterplan and staging plan prepared by Isthmus are presented below in Figure 2.2.

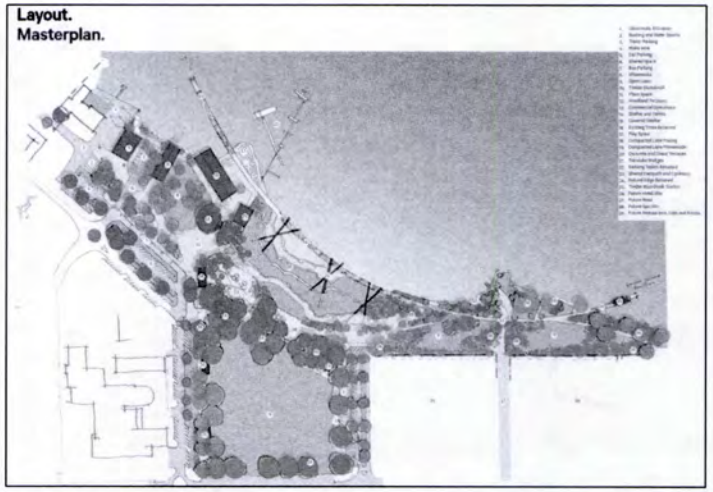

Figure 2.2 Proposed masterplan extracted from RLC Lakefront Redevelopment Developed Design for Stage 1 and 1a, dated August 2018.

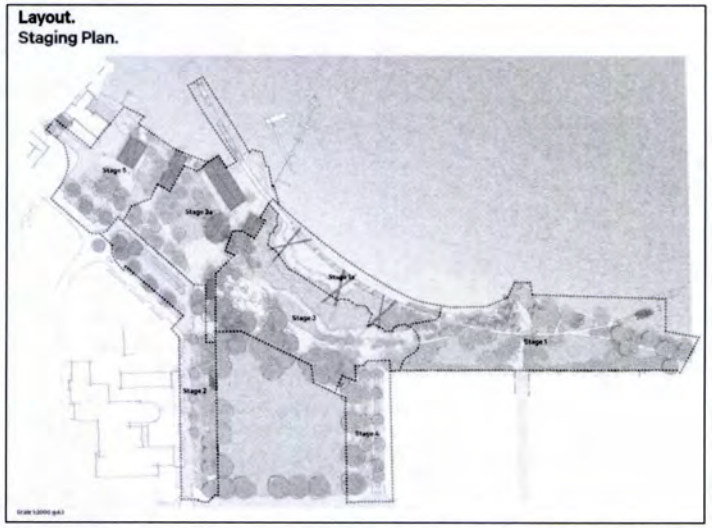

Figure 2.3: Proposed staging plan extracted from RLC Lakefront Redevelopment Developed Design for Stage 1 and 1a, dated August 2018.

3 Ground and Groundwater Conditions

3.1 Published geology

A published geological map of the area[1] indicates that the site is underlain by a mixture of undifferentiated rhyolite lavas and alluvium of the Tauranga Group. The location of the site in the context of the regional geology is presented on Figure 3.1 below.

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[1] Leonard, G.S.; Begg, J.G.; Wilson, C.J.N. (compilers) 2010: Geology of the Rotorua area. Institute of Geological & Nuclear Sciences 1:250,000 geological map 5. 1 sheet + 102 p. Lower Hutt, New Zealand. GNS Science.

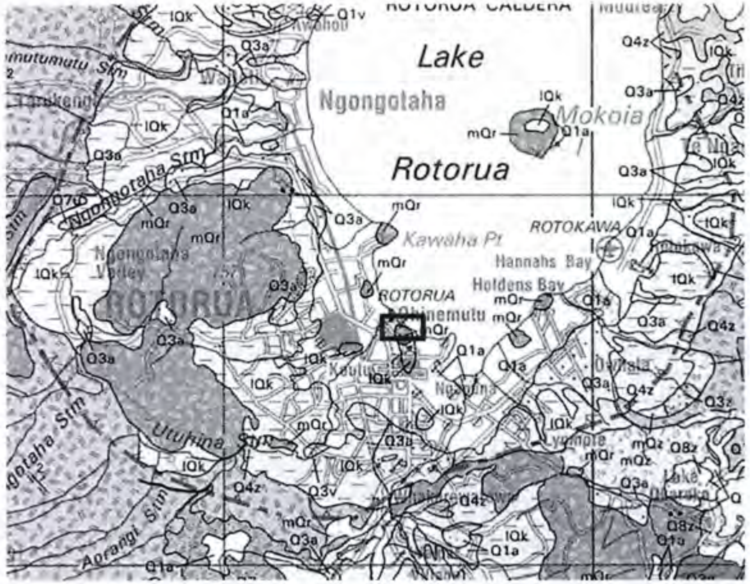

Figure 3.1: Geological setting with site location overlaid by red box.

3.2 Scope and location of investigations

The geotechnical investigations were carried out at the project site between 30 July 2018 and 2 August 2018. The investigations comprised:

- 4 No. machine-drilled boreholes (BHs); and
- 20 No. Cone Penetration Tests (CPTs).

Actual investigation locations were selected by T+T on the basis of access, presence of overhead obstructions, buried services and traffic management considerations.

The approximate locations of the investigations with associated 'as-drilled' coordinates are shown on the site plan in Appendix A.

3.2.1 Machine-drilled boreholes

The machine drilled boring of 4 No. vertical boreholes (BH01 through BH04) were undertaken from 30 July 2018 to 2 August 2018. The works were carried out using a rotary coring drilling rig using HQ3 triple tube coring, supplied and operated by Perry Geotech Ltd. The target depth of each borehole was 20.00 m.

In situ Standard Penetration Tests (SPTs) were carried out at regular (1.5 m) intervals for the full depth of each borehole. Up to 3 No. in situ push tube samples were proposed for each borehole if favourable soil conditions to undertake the test existed. However, there was only one instance where an in situ push tube test was deemed appropriate (within BH3) which resulted in no recovery due to unfavourable ground conditions. All drilling works were completed under the full time supervision of a T+T geotechnical engineer and due to the geothermal activity of the area, a geothermal supervisor was provided by Rotorua Well Drilling Co Ltd. The recovered drill core was photographed and logged to NZGS 'Field Description of Soil and Rock' guidelines. Each borehole was dipped to record groundwater level. Due to the close proximity of the investigation holes to the lake, the installation of standpipe piezometers was considered unnecessary.

Summary borehole logs which include ground temperature at regular intervals, as well as core photographs are presented in Appendix B. Table 3.1 below summarises the borehole details and temperature at finished borehole depth.

Table 3.1: Machine borehole summary

BH ID	Location (NZTM)		Ground Surface Elevation RL (m)	Depth (m)	Temp. at final depth (degrees)
	Easting (m)	Northing (m)			
BH01	1885401	5774626	281.7	19.95	57
BH02	1885186	5774666	280.6	19.95	52
BH03	1885068	5774777	281.2	15.45	72
BH04	1884975	5774752	281.4	3.45	72

3.2.2 Cone Penetration Tests

The pushing of 20 No. CPTs were undertaken by Perry Geotech Ltd from 30 July 2018 to 1 August 2018. The target depth for each CPT was 20 m. However, all CPTs were terminated before reaching target depth due to either refusal on tip resistance (CPTs 6, 14, and 15), or in the majority of cases (17 No. cases), upon reaching maximum allowable temperature (40 °C). All CPTs were advanced from ground level with the exception of CPT17 and CPT18, which were both pre-drilled using a machine auger to 1.5 m below ground level (bgl) due to obstructions, before commencing the CPT investigation.

Summary graphical logs are presented in Appendix B. A summary of the CPTs and termination depths is presented in Table 3.2 below.

Table 3.2: CPT summary

CPT ID	Target depth (m)	Termination depth (m)	Reason for termination	Comments
CPT01	20	9.69	Temperature	Faulty temperature gauge
CPT02	20	5.70	Temperature	
CPT03	20	10.84	Temperature	
CPT04	20	10.13	Temperature	
CPT05	20	10.79	Temperature	
CPT06	20	9.36	Tip Resistance	
CPT07	20	9.18	Temperature	
CPT08	20	11.13	Temperature	
CPT09	20	9.00	Temperature	
CPT10	20	12.18	Temperature	
CPT11	20	5.31	Temperature	
CPT12	20	7.05	Temperature	
CPT13	20	3.83	Temperature	
CPT14	20	1.30	Tip Resistance	
CPT15	20	11.18	Tip Resistance	
CPT16	20	9.44	Temperature	
CPT17	20	2.40	Temperature	1.5 m pre-drill
CPT18	20	2.00	Temperature	1.5 m pre-drill
CPT19	20	2.47	Temperature	
CPT20	20	5.15	Temperature	

3.3 Groundwater

Groundwater levels were recorded following investigations using an electronic dip meter. The recorded groundwater levels are presented below in Table 3.3.

Table 3.3: Groundwater levels following investigations

Investigation ID	Date recorded	Ground level (m) RL$^{(1)}$	Mean static groundwater level (m) RL	BOPRC Lake Level (m) RL$^{(2)}$	Depth below ground level (m)
BH01	30/07/2018	281.7	279.5	279.92	2.2
BH02	31/07/2018	280.6	278.9	279.91	1.7
BH03	01/08/2018	281.2	279.8	279.91	1.4
BH04	02/08/2018	281.4	280	279.91	1.4

(1) RL was inferred from contour maps.

(2) Source: Bay of Plenty Regional Council http://monitoring.boprc.govt.nz. Rounded to two decimal places. Readings taken at midday.

3.4 Laboratory testing

Four (4 No.) groundwater samples were collected from the base of the machine drilled boreholes. The samples were tested at Hill Laboratories, which is an IANZ accredited institution, for pH and sulphate levels. Full laboratory transcripts (which include test method descriptions) are presented in Appendix C and summarised in Table 3.4 below.

Table 3.4: Groundwater analysis summary

Investigation ID and depth	Sample Date	Sample Time	pH	Sulphates (g/m^3)
BH01 – 19.95 m	30/07/2018	15:10 hrs	6.5	30
BH02 – 19.95 m	31/07/2018	14:25 hrs	6.7	19.9
BH03 – 15.45 m	01/08/2018	15:00 hrs	6.4	35
BH04 – 3.45 m	02/08/2018	08:30 hrs	6.4	14.3

4 Applicability

This report has been prepared for the exclusive use of our client Rotorua Lakes Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:

Peter Molyneaux
Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by:

Lance Partner
Project Director

PEMO
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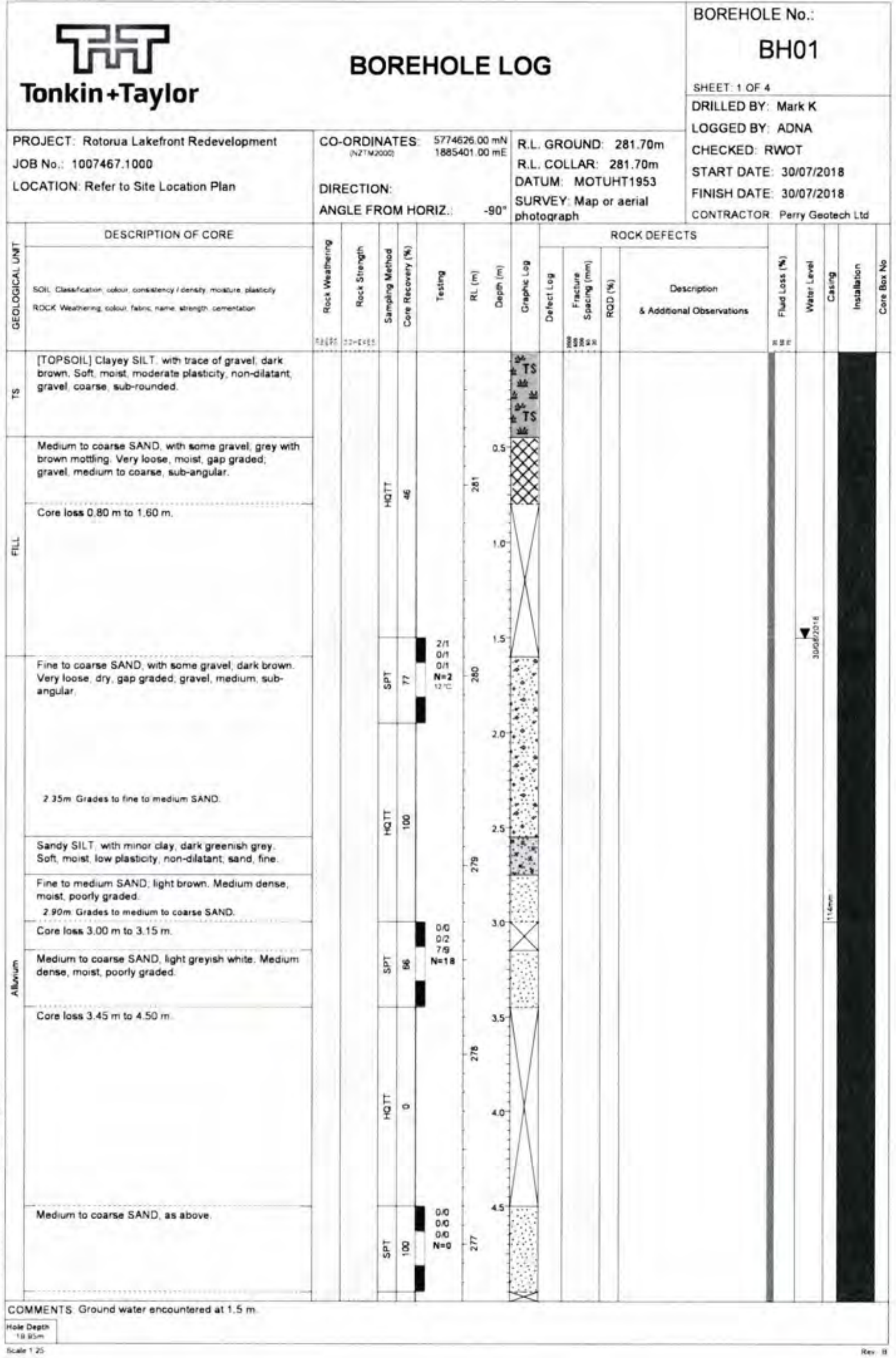
Appendix A: Site and exploratory hole location plan

Appendix B: Ground investigation results

- BH01 to BH04 Geotechnical logs and photographs
- CPT01 to CPT20 logs



Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: BH01

SHEET: 1 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 30/07/2018
FINISH DATE: 30/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774626.00 mN 1885401.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.70m
R.L. COLLAR: 281.70m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation
TS	[TOPSOIL] Clayey SILT, with trace of gravel, dark brown. Soft, moist, moderate plasticity, non-dilatant; gravel, coarse, sub-rounded.
FILL	Medium to coarse SAND, with some gravel; grey with brown mottling. Very loose, moist, gap graded; gravel, medium to coarse, sub-angular.
	Core loss 0.80 m to 1.60 m.
Alluvium	Fine to coarse SAND, with some gravel; dark brown. Very loose, dry, gap graded; gravel, medium, sub-angular. 2.35m: Grades to fine to medium SAND.
	Sandy SILT, with minor clay, dark greenish grey. Soft, moist, low plasticity, non-dilatant; sand, fine.
	Fine to medium SAND; light brown. Medium dense, moist, poorly graded. 2.90m: Grades to medium to coarse SAND.
	Core loss 3.00 m to 3.15 m.
	Medium to coarse SAND, light greyish white. Medium dense, moist, poorly graded.
	Core loss 3.45 m to 4.50 m.
	Medium to coarse SAND, as above.

Sampling Method	Core Recovery (%)	Testing
HQTT	46	
SPT	77	2/1 0/1 0/1 N=2 12°C
HQTT	100	
SPT	56	0/0 0/2 7/9 N=18
HQTT	0	
SPT	100	0/0 0/0 0/0 N=0

COMMENTS: Ground water encountered at 1.5 m.

Hole Depth
18.95m

Scale 1:25

Rev: B

General Log - 12/10/2018 9:04:29 AM - Produced with Core-GS by GeRoc

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 2 OF 4
DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 30/07/2018
FINISH DATE: 30/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774626.00 mN 1885401.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.70m
R.L. COLLAR: 281.70m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm)	RQD (%)	ROCK DEFECTS Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Alluvium	Core loss 4.95 m to 5.20 m.																	
	Silty, coarse SAND; greenish brown. Loose, wet, uniformly graded, slow dilatancy.																	
	SILT, with minor sand, trace of clay; greenish brown. Very soft, wet, non-plastic, slow dilatancy; sand, fine to medium.			HQTT	76			5.5										Box 1: 0.0-5.6m
	Silty, fine SAND; light greyish white. Loose, moist, poorly graded, non-dilatant, pumiceous.						276											
	Silty, medium to coarse SAND; light brown. Medium dense, moist, poorly graded, non-dilatant, pumiceous.																	
	Core loss 6.00 m to 6.15 m.			SPT	66	0/1 3/4 2/3 N=12 31 °C		6.0										
	Medium SAND, with minor gravel; mottled light grey. Medium dense, moist, poorly graded, pumiceous; gravel, fine, sub-angular, pumiceous.																	
	Core loss 6.45 m to 6.90 m.						275	6.5										
	Medium to coarse SAND, with trace of gravel; light greenish grey. Very loose, moist, well graded, pumiceous; gravel, fine, sub-rounded, pumiceous. *7.15m:* Grades to coarse SAND, with minor gravel.			HQTT	57			7.0										
	Core loss 7.50 m to 7.70 m.			SPT	55	0/0 0/0 0/0 N=0 30 °C	274	7.5										
	Sandy, fine to medium GRAVEL, with some silt; greyish white with greenish mottling. Very loose, wet, gap graded, rapid dilatancy, pumiceous; sand, medium to coarse, pumiceous.																	
	Core loss 7.95 m to 8.10 m.							8.0										
	Silty, fine to coarse SAND, with trace of gravel; light greenish grey. Loose, wet, well graded, slow dilatancy, pumiceous; gravel, fine, sub-angular, pumiceous.			HQTT	85		273	8.5										
	Core loss 9.00 m to 9.15 m.			SPT	66	0/1 1/2 2/2 N=7 30 °C		9.0										
	Clayey SILT; light grey. Firm, moist, moderate plasticity, rapid dilatancy, pumiceous.							9.5										
	Silty, medium to coarse SAND; light grey. Loose, moist, gap graded, non-dilatant, pumiceous.						272											

COMMENTS: Ground water encountered at 1.5 m.

Hole Depth
19.85m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 3 OF 4
DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 30/07/2018
FINISH DATE: 30/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774626.00 mN 1885401.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.70m
R.L. COLLAR: 281.70m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm)	RQD (%)	ROCK DEFECTS Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Alluvium	Sandy SILT, with trace of clay; light grey. Medium dense, moist, non-plastic, non-dilatant, pumiceous; sand, medium, pumiceous.			HQTT	100													
Ignimbrite	Silty, fine to medium SAND; mottled grey. Medium dense, moist, well graded, non-dilatant, pumiceous.																	
	Core loss 10.50 m to 10.70 m.			SPT	55	8/6 4/4 4/4 N=16 32 °C	271	10.5										
	Medium to coarse SAND, with minor gravel; light grey. Medium dense, moist, gap graded, pumiceous; gravel, medium to coarse, angular to sub-angular, pumiceous. *11.75m:* Grades to fine to medium SAND, with trace of gravel.			HQTT	100		270	11.0 11.5										
	Core loss 12.00 m to 12.20 m.			SPT	55	4/4 4/4 4/5 N=17 28 °C		12.0										
	Fine to medium SAND, with trace of gravel; light grey. Medium dense, moist, gap graded; gravel, medium to coarse, angular to sub-angular. *12.65m:* Grades to medium to coarse SAND, with minor gravel. *13.25m:* Grades to coarse SAND, with minor gravel.			HQTT	100		269	12.5 13.0										
Alluvium	Medium SAND, with trace of gravel, dark grey. Dense, moist, gap graded; gravel, medium, sub-rounded.			SPT	100	1/4 8/10 11/11 N=40 30 °C	268	13.5										Box 3: 10.1-14.0m
	14.30m: Grades to medium to coarse SAND.			HQTT	100			14.0 14.5										
	SILT, with trace of clay; light grey. Soft, moist, non-plastic, slow dilatancy.						267											

COMMENTS: Ground water encountered at 1.5 m.

Hole Depth
19.85m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 4 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 30/07/2018
FINISH DATE: 30/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

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Alluvium	Core loss 15.00 m to 15.05 m. SILT, with some clay; light grey. Soft, moist, moderate plasticity, rapid dilatancy.			SPT	88	0/1 0/0 1/2 N=3 47 °C												
	Core loss 15.45 m to 15.65 m.			HQTT	80			15.5										
	SILT, with some clay, as above.						266	16.0										
	Sandy SILT, with minor clay; light grey. Soft, moist, low plasticity, rapid dilatancy.																	
	Sandy SILT, with minor clay; light grey. Stiff, moist, low plasticity, rapid dilatancy; sand, medium to coarse.			SPT	100	0/0 1/3 3/4 N=11 44 °C	265	16.5 17.0										
	Medium to coarse SAND; light grey. Medium dense, moist, poorly graded.			HQTT	100			17.5										Box 4: 14.0-17.6m
	Silty, fine SAND; light grey. Medium dense, moist, poorly graded, non-dilatant.						264											
	Core loss 18.00 m to 18.25 m.			SPT	44	1/3 3/3 3/4 N=13 56 °C		18.0										
	Silty, fine SAND, as above.																	
	Fine to medium SAND; light grey. Medium dense, moist, poorly graded. *19.25m*: Grades to medium to coarse SAND.			HQTT	100		263	18.5 19.0										
	Core loss 19.50 m to 19.70 m.			SPT	55	2/1 3/3 5/8 N=19 57 °C Water Sample @ 20.0m		19.5										
	Silty, fine SAND; light grey. Medium dense, moist, poorly graded, non-dilatant. 19.95m: Target depth						262											Box 5: 17.6-20.0m

COMMENTS: Ground water encountered at 1.5 m.

Hole Depth
19.95m

Scale 1:25

Rev: B

General Log - 12/10/2018 9:04:30 AM - Produced with Core-GS by GeRoc

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Core Photographs – BH01

BH01: 0.00m to 5.80m

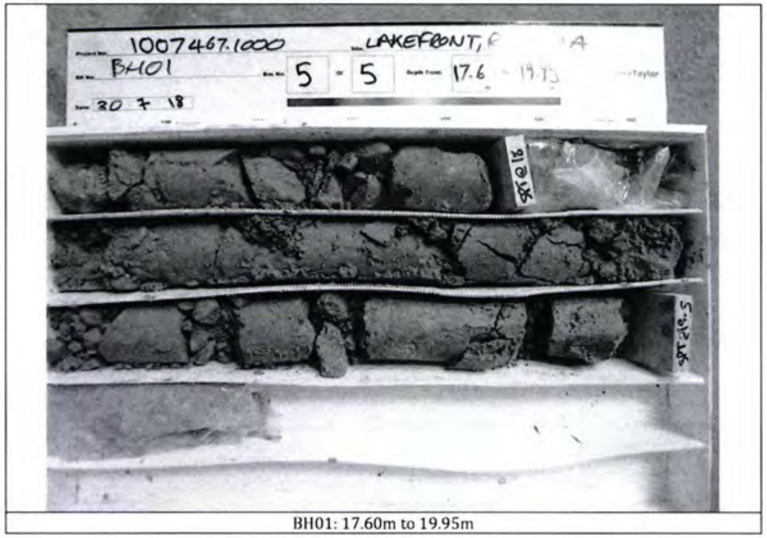
BH01: 5.80 m to 10.10m

Core Photographs – BH01

BH01: 10.10m to 13.95m

BH01: 13.95m to 17.60m

Core Photographs – BH01



BH01: 17.60m to 19.95m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH02**

SHEET: 1 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 31/07/2018
FINISH DATE: 31/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774666.00 mN 1885186.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 280.60m
R.L. COLLAR: 280.60m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

Columns: GEOLOGICAL UNIT | Rock Weathering | Rock Strength | Sampling Method | Core Recovery (%) | Testing | RL (m) | Depth (m) | Graphic Log | ROCK DEFECTS: Defect Log, Fracture Spacing (mm), RQD (%), Description & Additional Observations | Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

GEOLOGICAL UNIT	DESCRIPTION OF CORE	Sampling Method	Core Recovery (%)	Testing
TS	[TOPSOIL] Clayey SILT; dark brown. Firm, moist, low plasticity, non-dilatant.	HQTT	70	
FILL	Silty, fine to medium SAND, with trace of gravel; dark brown. Loosely packed, dry, gap graded, non-dilatant; gravel, fine, sub-angular.			
	Core loss 0.80 m to 1.25 m.			
	Silty, fine SAND, with trace of gravel; light grey. Loosely packed, dry, gap graded, rapid dilatancy; gravel, fine, sub-angular.			
Alluvium	Core loss 1.50 m to 1.55 m.	SPT	88	0/0 0/0 0/0 N=0
	SILT, with some clay, some sand; greenish brown. Very soft, moist, low plasticity, slow dilatancy; sand, medium.			
	Core loss 1.95 m to 2.05 m.	HQTT	90	
	Fine to medium SAND, with trace of gravel; dark brown. Very loose, saturate, gap graded; gravel, fine, sub-angular.			
	Sandy SILT, with minor clay; light greenish brown. Very soft, wet, low plasticity, slow dilatancy; sand, medium.			
	Coarse SAND; light greyish white. Very loose, moist, uniformly graded, very pumiceous.			
	Core loss 3.00 m to 3.05 m.	SPT	88	0/0 0/0 0/0 N=0
	SILT, with minor clay; dark greyish brown. Very soft, saturated, low plasticity, rapid dilatancy.	HQTT	100	
	SILT; light greyish white. Very loose, moist, non-plastic, rapid dilatancy.			
	Core loss 4.50 m to 4.55 m.	SPT	88	0/0 0/0 0/1 N=1 15°C
	Silty, medium SAND; light brown. Very loose, moist, gap graded, rapid dilatancy.			

RL (m): 280, 279, 278, 277, 276

Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

Graphic Log: TS

Water Level: 31/07/2018

Casing: 114mm

Core Box No: Box 1: 0.0-4.0m

COMMENTS: Ground water encountered at 1.3 m.

Hole Depth 19.95m

Scale 1:25

Rev. B

General Log - 12/10/2018 9:07:13 AM - Produced with Core-GS by GeRoc

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Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH02**

SHEET: 2 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 31/07/2018
FINISH DATE: 31/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774666.00 mN 1885186.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 280.60m
R.L. COLLAR: 280.60m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

Columns: GEOLOGICAL UNIT | Rock Weathering | Rock Strength | Sampling Method | Core Recovery (%) | Testing | RL (m) | Depth (m) | Graphic Log | ROCK DEFECTS: Defect Log, Fracture Spacing (mm), RQD (%), Description & Additional Observations | Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

GEOLOGICAL UNIT	DESCRIPTION OF CORE	Sampling Method	Core Recovery (%)	Testing
Alluvium	Medium SAND, with minor gravel; mottled greyish white. Loose, moist, poorly graded, pumiceous; gravel, fine, angular.	HQTT	100	
	Coarse SAND, with some gravel; greyish white. Loose, moist, gap graded, very pumiceous; gravel, medium, sub-angular, pumiceous.			
	Core loss 6.00 m to 6.10 m.	SPT	77	1/2 2/2 2/2 N=8
	Sandy, fine to medium GRAVEL; light greyish white. Loose, moist, well graded, very pumiceous; sand, coarse, pumiceous.	HQTT	100	
	Core loss 7.50 m to 7.65 m.	SPT	66	2/2 2/2 2/2 N=8 31°C
	Sandy, fine to coarse GRAVEL; light greyish white. Loose, moist, well graded, very pumiceous; sand, coarse, very pumiceous.	HQTT	100	
	SILT, with minor clay, trace of sand; light grey. Very soft, moist, low plasticity, non-dilatant; sand, fine.	SPT	100	1/0 0/0 0/0 N=0 23°C
	Core loss 9.45 m to 10.10 m.			

RL (m): 275, 274, 273, 272, 271

Depth (m): 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5

Core Box No: Box 2: 4.0-7.5m

COMMENTS: Ground water encountered at 1.3 m.

Hole Depth 19.95m

Scale 1:25

Rev. B

General Log - 12/10/2018 9:07:13 AM - Produced with Core-GS by GeRoc

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH02
SHEET 3 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 31/07/2018
FINISH DATE: 31/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774666.00 mN 1885186.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 280.60m
R.L. COLLAR: 280.60m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

ROCK DEFECTS

GEOLOGICAL UNIT	DESCRIPTION OF CORE	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm)	RQD (%)	Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Alluvium	Core loss 9.45 m to 10.10 m.																	
	SILT, with some sand, minor clay; light grey. Very soft, moist, low plasticity, non-dilatant, pumiceous; sand, fine to medium, pumiceous.			HQTT	38													
	Silty, fine to medium SAND, with trace of gravel; light grey. Very loose, moist, gap graded, non-dilatant; gravel, medium, sub-rounded.																	
	SILT, with some sand, minor clay; light greenish grey. Very soft, moist, low plasticity, non-dilatant; sand, fine to coarse.			SPT	100	0/0 0/0 0/2 N=2	270	10.5										
				HQTT	100		269	11.0 11.5										Box 3: 7.5-12.0m
Ignimbrite	Silty, fine to coarse SAND, with trace of gravel; mottled grey. Loose, moist, well graded, non-dilatant; gravel, fine, sub-angular.			SPT	100	1/1 2/2 2/2 N=8 44 °C		12.0										
				HQTT	100		268	12.5 13.0										
Alluvium	Medium SAND; mottled grey. Very dense, moist, uniformly graded.			SPT	100	1/5 10/15 19/6 for 25mm N>=50 44 °C	267	13.5										
	Medium to coarse SAND, with trace of gravel; mottled grey. Very dense, moist, well graded; gravel, fine, rounded. *14.85m: Grades to medium SAND, poorly graded.*			HQTT	100		266	14.0 14.5										

COMMENTS: Ground water encountered at 1.3 m.

Hole Depth
19.95m

Scale 1:25

General Log - 12/10/2018 8:07:13 AM - Produced with Core-GS by GeRoc

Rev: B

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH02
SHEET 4 OF 4

DRILLED BY: Mark K
LOGGED BY: ADNA
CHECKED: RWOT
START DATE: 31/07/2018
FINISH DATE: 31/07/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774666.00 mN 1885186.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 280.60m
R.L. COLLAR: 280.60m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

ROCK DEFECTS

GEOLOGICAL UNIT	DESCRIPTION OF CORE	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm)	RQD (%)	Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Alluvium	Fine to medium SAND; grey. Dense, moist, poorly graded.			SPT	100	7/9 8/9 9/11 N=37												Box 4: 12.0-15.6m
	Fine SAND, with some silt; grey. Medium dense, moist, poorly graded, slow dilatancy.			HQTT	100			15.5										
	Fine to medium SAND; grey. Dense, moist, poorly graded. *15.80m: Grades to medium to coarse SAND.*						265	16.0										
	Silty, fine SAND; light grey. Dense, moist, poorly graded, rapid dilatancy.																	
	16.50m: Grades to with trace of fine to medium gravel, angular to sub-angular.			SPT	100	6/6 7/5 8/10 N=30 40 °C	264	16.5										
	SILT, with minor sand; light grey. Medium dense, moist, non-plastic, rapid dilatancy; sand, fine. *17.80m: SILT, with minor sand, with minor clay.*			HQTT	100		263	17.0 17.5										
	Clayey SILT; light grey. Stiff, moist, moderate plasticity, slow dilatancy.			SPT	100	3/2 2/3 4/4 N=13 38 °C		18.0										
	Core loss 18.45 m to 18.80 m.			HQTT	66		262	18.5										
	Clayey SILT, with minor sand; light grey. Stiff, moist, moderate plasticity, slow dilatancy; sand, medium.							19.0										
	Silty, fine to medium SAND, with minor clay; light grey. Dense, moist, well graded, non-dilatant.																	Box 5: 15.6-18.4m
	Sandy SILT, with some clay, trace of gravel; light grey. Stiff, moist, moderate plasticity, slow dilatancy; sand, medium to coarse; gravel, medium, sub-angular. 19.95m: Target depth			SPT	100	1/1 1/2 4/3 N=10 52 °C Water Sample @ 20.0m	261	19.5										Box 6: 18.4-20.0m

COMMENTS: Ground water encountered at 1.3 m.

Hole Depth
19.95m

Scale 1:25

General Log - 12/10/2018 8:07:13 AM - Produced with Core-GS by GeRoc

Rev: B

Core Photographs – BH02

BH02: 0.00m to 4.00m

BH02: 4.00 m to 7.50m

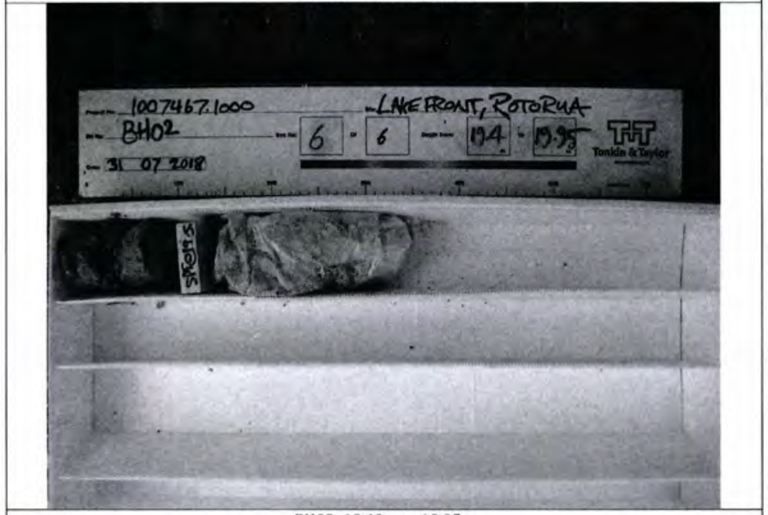
Core Photographs – BH02

BH02: 7.50m to 12.00m

BH02: 12.00m to 15.55m

Core Photographs – BH02

BH02: 15.55m to 19.40m



BH02: 19.40m to 19.95m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH03**
SHEET: 1 OF 4

DRILLED BY: Mark K
LOGGED BY: PEMO
CHECKED: RWOT
START DATE: 01/08/2018
FINISH DATE: 01/08/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES (NZTM2000): 5774777.00 mN 1885068.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.20m
R.L. COLLAR: 281.20m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

GEOLOGICAL UNIT	DESCRIPTION OF CORE	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)
Alluvium	[TOPSOIL] Sandy SILT, with minor clay; brown, loosely packed, moist, non-plastic, rapid dilatancy; sand, fine; trace of rootlets.	HQTT	100		281	0.5
FILL	Silty, fine to coarse SAND, with minor gravel; orange brown. Loosely packed, moist, well graded, rapid dilatancy; gravel, fine to medium, sub-angular.				280	1.0 1.5
Alluvium	Sandy CLAY, with some silt, trace of gravel, dark greyish brown. Soft, wet, low plasticity, extra sensitive, strong organic odour; sand, fine to medium, gravel, fine, angular.	SPT	100	0/0 0/0 0/0 N=0		
	Core loss 1.95 m to 3.00 m.	HQTT PT	0 0		279 278	2.0 2.5 3.0
	Clayey SILT; pale greyish brown with black mottling. Soft, wet, low plasticity, extra sensitive.Occasional pockets of medium to coarse sand (< 10 mm diameter).	SPT HQTT SPT	100 100 100	0/0 0/0 0/0 N=0 0/0 0/0 0/0 N=0 33 °C	277	3.5 4.0 4.5

Rock Weathering | Rock Strength | Graphic Log | ROCK DEFECTS: Defect Log | Fracture Spacing (mm) | RQD (%) | Description & Additional Observations | Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

Water Level: ▼ 01/08/2018
Casing: 114mm

COMMENTS: Ground water encountered at 1.2 m.Hole abandoned due to high ground temperature

Hole Depth 15.45m

Scale 1:25

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH03**
SHEET: 2 OF 4

DRILLED BY: Mark K
LOGGED BY: PEMO
CHECKED: RWOT
START DATE: 01/08/2018
FINISH DATE: 01/08/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: 5774777.00 mN 1885068.00 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.20m
R.L. COLLAR: 281.20m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

GEOLOGICAL UNIT	DESCRIPTION OF CORE
Alluvium	Clayey SILT; pale greyish brown with black mottling. Soft, wet, low plasticity, extra sensitive.Occasional pockets of medium to coarse sand (< 10 mm diameter).
	Silty, medium to coarse SAND, pale grey with orange and black motttling. Very loose, wet, gap graded, pumiceous.
	Core loss 7.95 m to 8.45 m.
	Silty, medium to coarse SAND, as above.
	Core loss 9.00 m to 9.15 m.
	Sandy SILT; pale grey. Stiff , moist to wet, low plasticity, very sensitive, sand, fine to medium.
	Core loss 9.45 m to 10.25 m.

Sampling Method	Core Recovery (%)	Testing
HQTT	100	
SPT	100	0/0 0/0 0/0 N=0 46 °C
HQTT	100	
SPT	100	1/0 0/0 0/0 N=0 51 °C
HQTT	52	
SPT	66	1/1 0/1 0/2 N=3 34 °C

RL (m): 276, 275, 274, 273, 272
Depth (m): 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5

Rock Weathering | Rock Strength | Graphic Log | ROCK DEFECTS: Defect Log, Fracture Spacing (mm), RQD (%), Description & Additional Observations | Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

Core Box No: Box 2: 5.2-9.5m

COMMENTS: Ground water encountered at 1.2 m.Hole abandoned due to high ground temperature

Hole Depth 15.45m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH03**
SHEET: 3 OF 4

DRILLED BY: Mark K
LOGGED BY: PEMO
CHECKED: RWOT
START DATE: 01/08/2018
FINISH DATE: 01/08/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: 5774777.00 mN 1885068.00 mE (NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.20m
R.L. COLLAR: 281.20m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

DESCRIPTION OF CORE — SOIL: Classification, colour, consistency / density, moisture, plasticity; ROCK: Weathering, colour, fabric, name, strength, cementation

GEOLOGICAL UNIT	DESCRIPTION OF CORE
Alluvium	Core loss 9.45 m to 10.25 m.
	Silty, medium to coarse SAND, with some gravel, pale grey. Very dense, moist, well graded, rapid dilatancy, cementitious, gravel, fine to coarse, sub-angular, rhyolite.
	SILT, with some sand, pale greyish white. Stiff, moist to wet, low plasticity, rapid dilatancy; sand, fine to medium.
	Sandy SILT, pale greyish white. Very stiff to hard, moist, low plasticity, extra sensitive, contains cemented lenses (<5 mm thick), sand, fine to coarse.
Ignimbrite	Unweathered to slightly weathered, light grey with black mottling IGNIMBRITE. Weak.
Alluvium	Sandy SILT, with trace of gravel; pale greyish white. Hard, moist, low plasticity, extra sensitive; sand, fine to coarse; gravel, fine, sub-angular.
Rhyolite	Unweathered to slightly weathered, grey RHYOLITE. Weak.

Sampling Method	Core Recovery (%)	Testing
HQTT	23	
SPT	100	0/0 0/0 0/0 N=0 43 °C
HQTT	100	
SPT	100	2/2 3/4 5/7 N=19 42 °C
HQTT	100	
SPT	100	4/10 8/6 7/8 N=29 45 °C
HQTT	100	

RL (m): 271, 270, 269, 268, 267
Depth (m): 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5

Rock Weathering | Rock Strength | Graphic Log | ROCK DEFECTS: Defect Log, Fracture Spacing (mm), RQD (%), Description & Additional Observations | Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

Core Box No: Box 3: 9.5-13.5m

COMMENTS: Ground water encountered at 1.2 m.Hole abandoned due to high ground temperature

Hole Depth 15.45m

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH03**

SHEET 4 OF 4

PROJECT: Rotorua Lakefront Redevelopment	DRILLED BY: Mark K
JOB No.: 1007467.1000	LOGGED BY: PEMO
LOCATION: Refer to Site Location Plan	CHECKED: RWOT
CO-ORDINATES: (NZTM2000) 5774777.00 mN 1885068.00 mE	START DATE: 01/08/2018
DIRECTION:	FINISH DATE: 01/08/2018
ANGLE FROM HORIZ.: -90°	CONTRACTOR: Perry Geotech Ltd
R.L. GROUND: 281.20m	
R.L. COLLAR: 281.20m	
DATUM: MOTUHT1953	
SURVEY: Map or aerial photograph	

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	ROCK DEFECTS: Defect Log	ROCK DEFECTS: Fracture Spacing (mm)	ROCK DEFECTS: RQD (%)	ROCK DEFECTS: Description & Additional Observations	Fluid Loss (%)	Water Level	Casing	Installation	Core Box No
Rhyolite	Unweathered to slightly weathered, grey RHYOLITE. Weak.			SPT	100	6/3 3/10 5 for 20mm N>=50 Bouncing 72°C	266											Box 4: 13.5-15.5m
	15.45m: Other - see notes					Water Sample @ 15.5m	265 264 263 262	15.5 16.0 16.5 17.0 17.5 18.0 18.5 19.0 19.5										

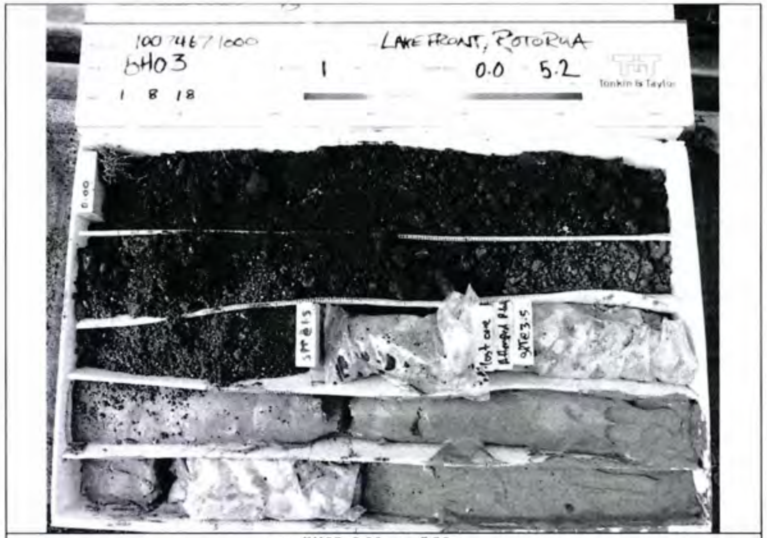
COMMENTS: Ground water encountered at 1.2 m. Hole abandoned due to high ground temperature

Hole Depth 15.45m

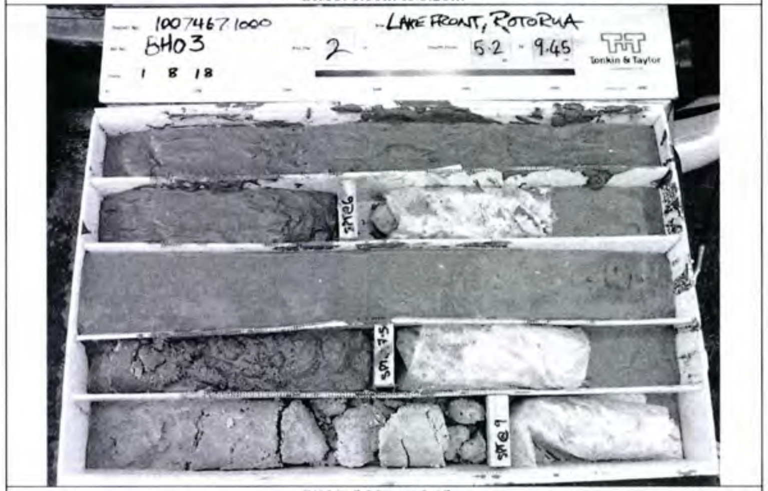
Scale 1:25

Rev. B

Core Photographs – BH03



BH03: 0.00m to 5.20m

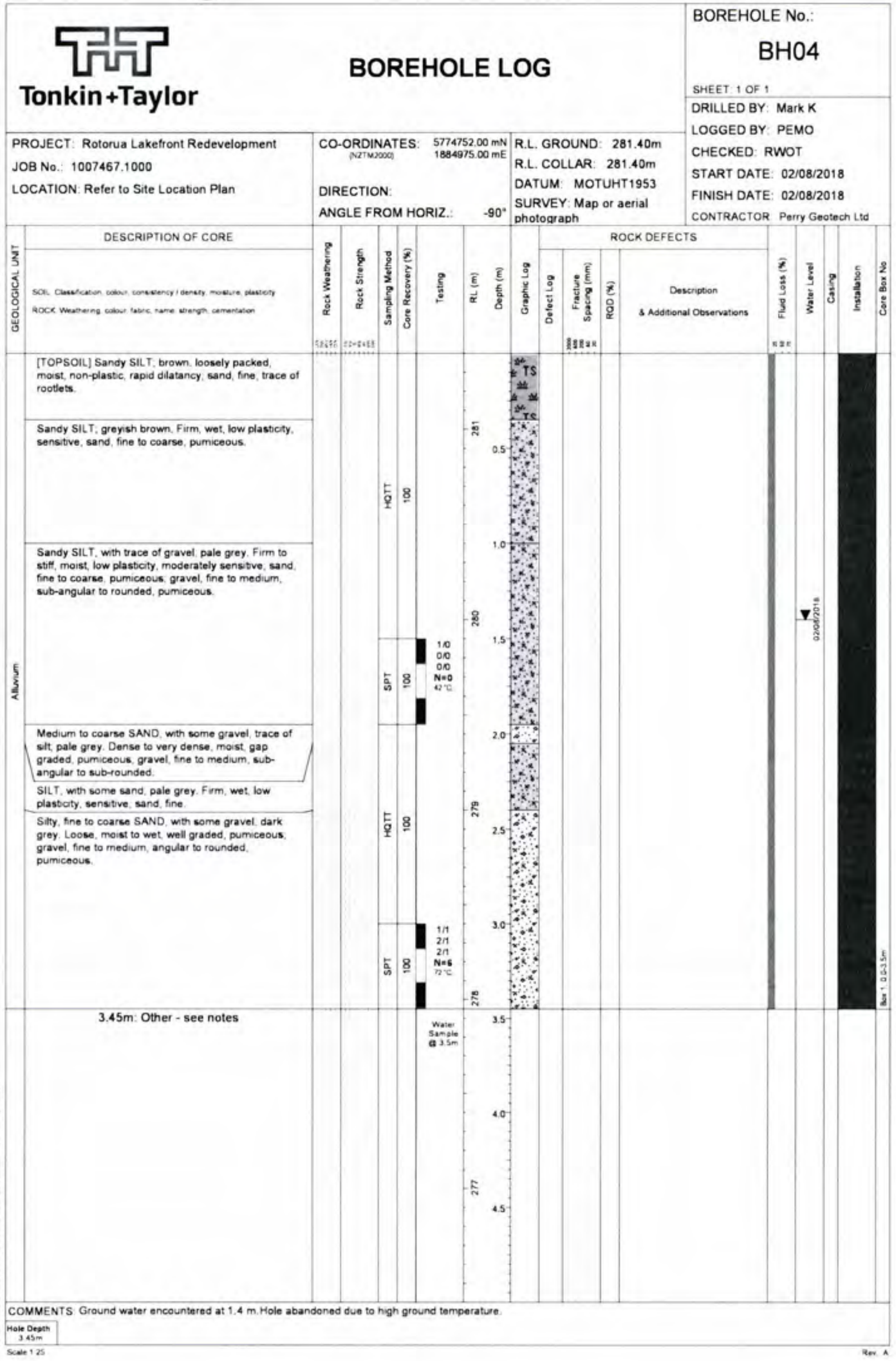


BH03: 5.20m to 9.45m

Core Photographs – BH03

BH03: 9.45m to 13.50m

BH03: 13.50 – 15.45m



Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.:
BH04
SHEET: 1 OF 1

DRILLED BY: Mark K
LOGGED BY: PEMO
CHECKED: RWOT
START DATE: 02/08/2018
FINISH DATE: 02/08/2018
CONTRACTOR: Perry Geotech Ltd

PROJECT: Rotorua Lakefront Redevelopment
JOB No.: 1007467.1000
LOCATION: Refer to Site Location Plan

CO-ORDINATES: (NZTM2000) 5774752.00 mN 1884975.00 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 281.40m
R.L. COLLAR: 281.40m
DATUM: MOTUHT1953
SURVEY: Map or aerial photograph

GEOLOGICAL UNIT	DESCRIPTION OF CORE SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering	Rock Strength	Sampling Method	Core Recovery (%)	Testing	Graphic Log
Alluvium	[TOPSOIL] Sandy SILT; brown, loosely packed, moist, non-plastic, rapid dilatancy; sand, fine, trace of rootlets.			HQTT	100		TS
	Sandy SILT; greyish brown. Firm, wet, low plasticity, sensitive, sand, fine to coarse, pumiceous.						
	Sandy SILT, with trace of gravel; pale grey. Firm to stiff, moist, low plasticity, moderately sensitive, sand, fine to coarse, pumiceous; gravel, fine to medium, sub-angular to rounded, pumiceous.			SPT	100	1/0 0/0 0/0 N=0 42 °C	
	Medium to coarse SAND, with some gravel, trace of silt; pale grey. Dense to very dense, moist, gap graded, pumiceous, gravel, fine to medium, sub-angular to sub-rounded.			HQTT	100		
	SILT, with some sand; pale grey. Firm, wet, low plasticity, sensitive, sand, fine.						
	Silty, fine to coarse SAND, with some gravel; dark grey. Loose, moist to wet, well graded, pumiceous; gravel, fine to medium, angular to rounded, pumiceous.			SPT	100	1/1 2/1 2/1 N=6 72 °C	
	3.45m: Other - see notes					Water Sample @ 3.5m	

RL (m): 281, 280, 279, 278, 277
Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

ROCK DEFECTS: Defect Log | Fracture Spacing (mm) | RQD (%) | Description & Additional Observations

Fluid Loss (%) | Water Level | Casing | Installation | Core Box No

Water Level: 02/08/2018

Core Box No: Box 1: 0.0-3.5m

COMMENTS: Ground water encountered at 1.4 m.Hole abandoned due to high ground temperature.

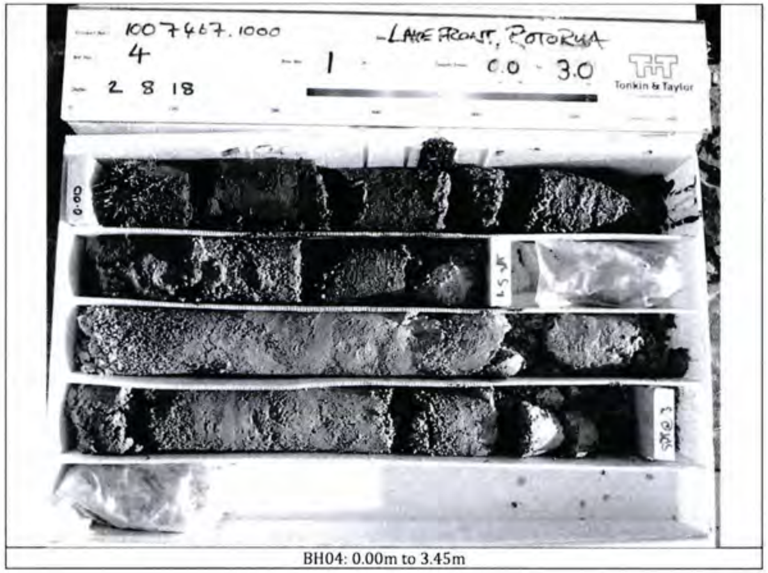
Hole Depth 3.45m

Scale 1:25

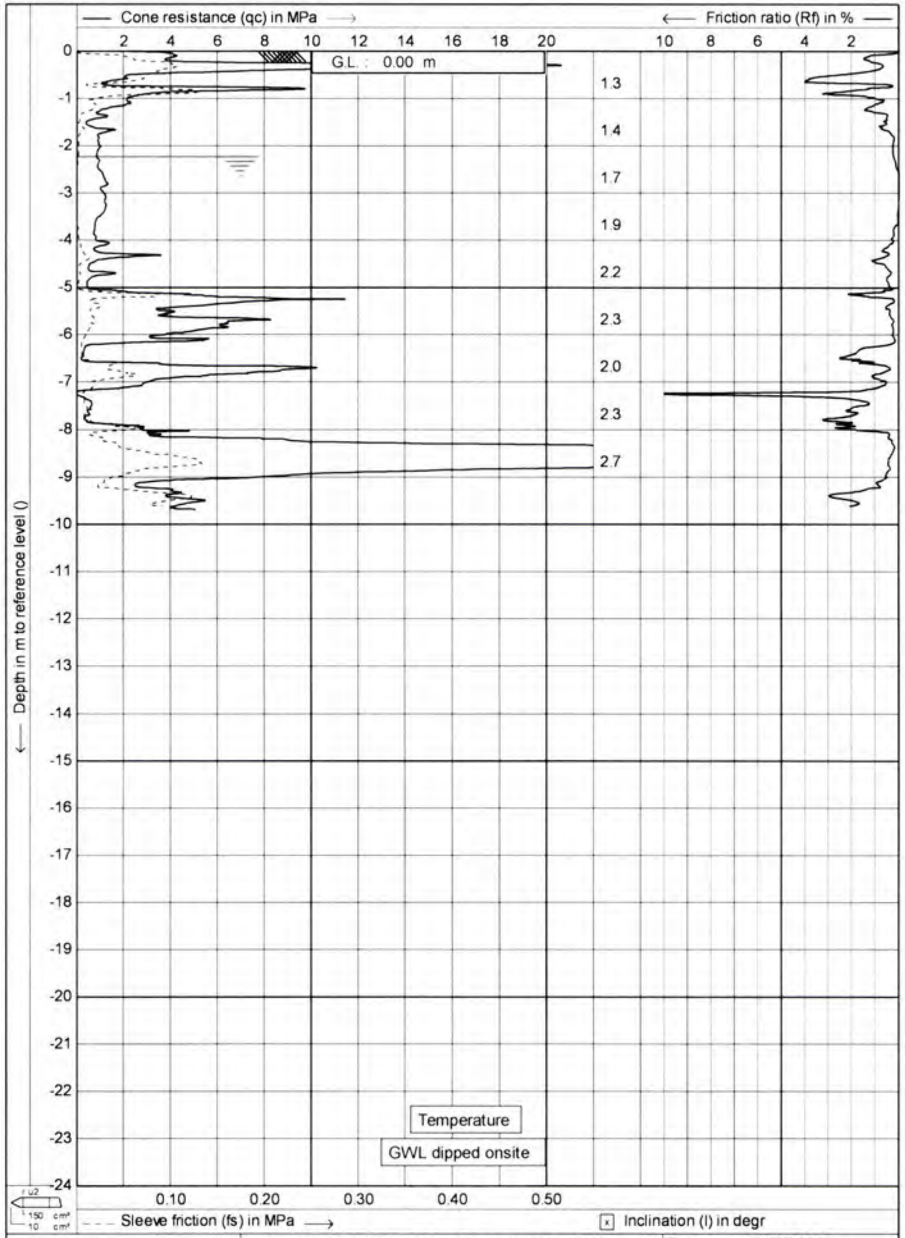
Rev. A

General Log - 12/10/2018 9:18:02 AM - Produced with Core-GS by GeRoc

Core Photographs – BH04

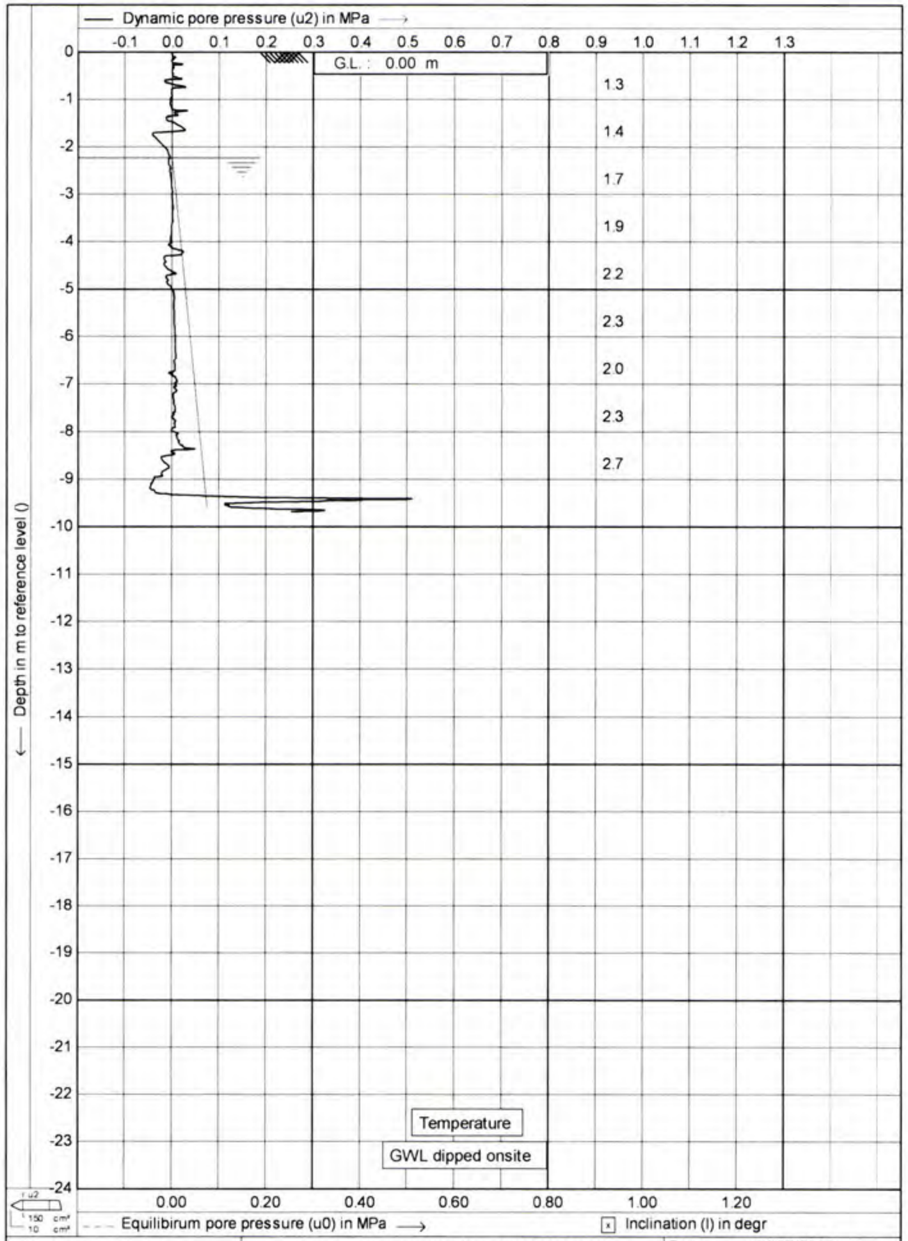


BH04: 0.00m to 3.45m



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12		
Project : **Site Investigations**	Date : **31/07/2018**	
Location: **Rotorua Lakefront - Rotorua**	Cone no. : **C10CFIIPT.C18265**	
	Project no. : **05TT08**	
Position: **0, 0 RD**	CPT no. : **01**	**1/15**

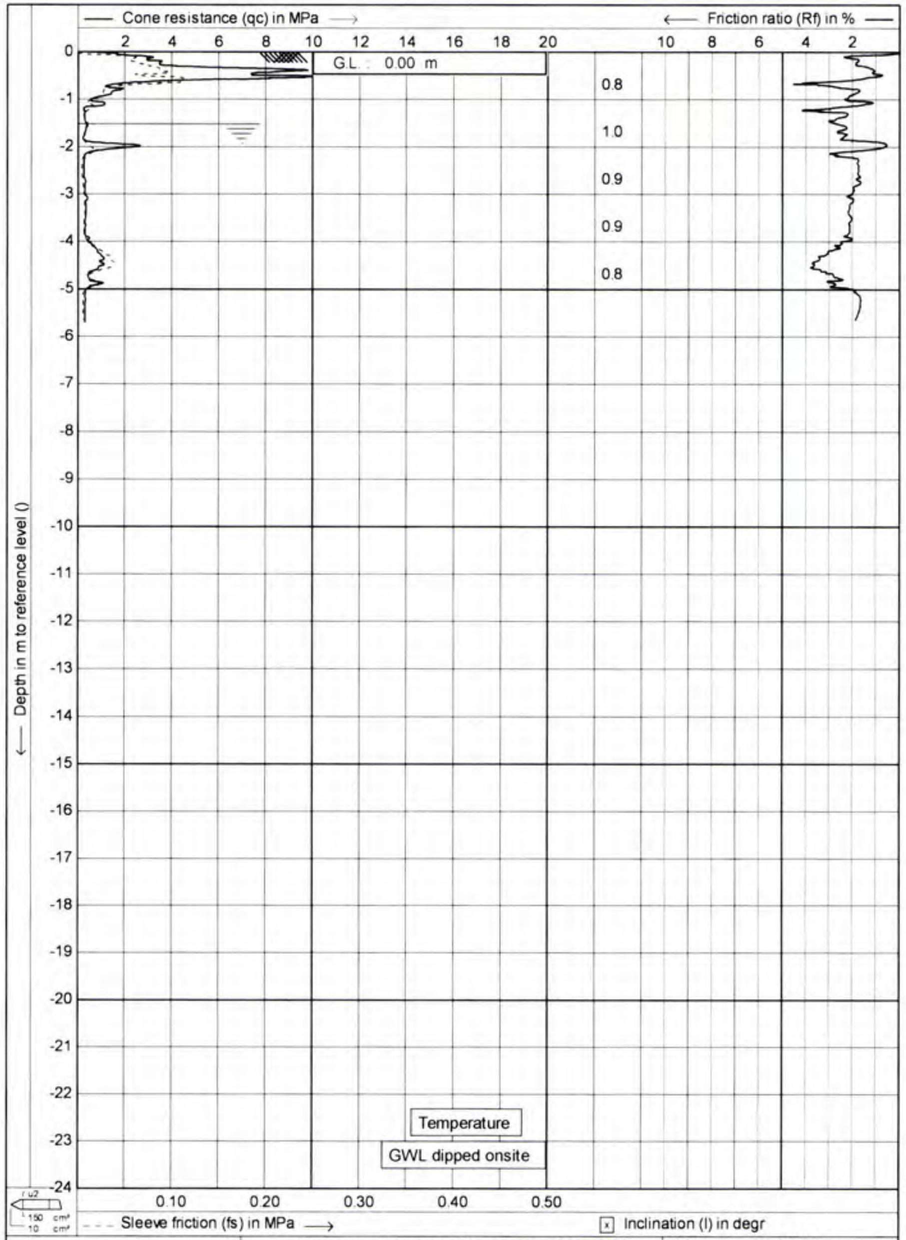


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIPT.C18265**
Project no. : **05TT08**
CPT no. : **01** **2/15**

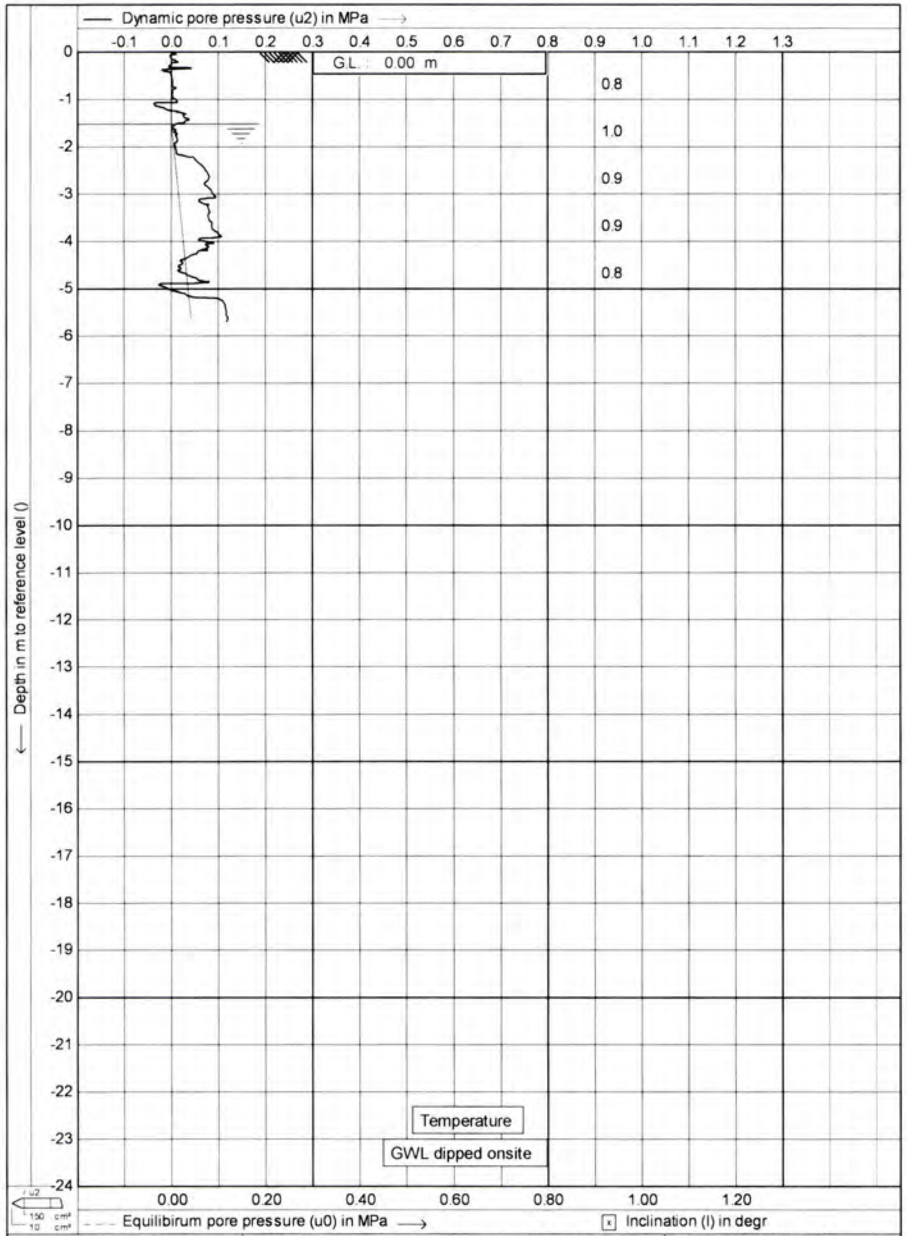


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIPT.C09015**
Project no. : **05TT08**
CPT no. : **02** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : Site Investigations

Location: Rotorua Lakefront - Rotorua

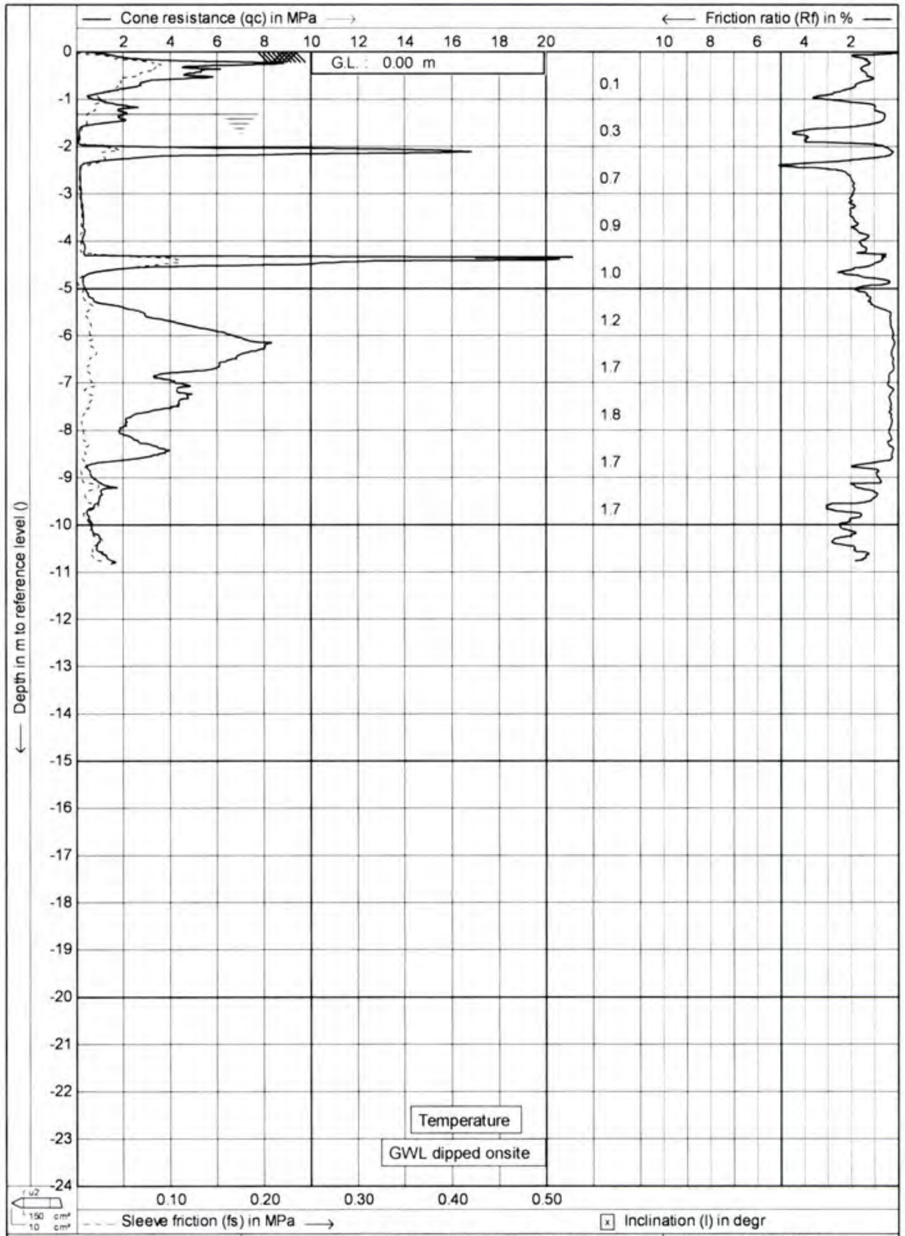
Position: 0, 0 RD

Date : 31/07/2018

Cone no. : C10CFIPT.C09015

Project no. : 05TT08

CPT no. : 02 2/15



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : Site Investigations

Location: Rotorua Lakefront - Rotorua

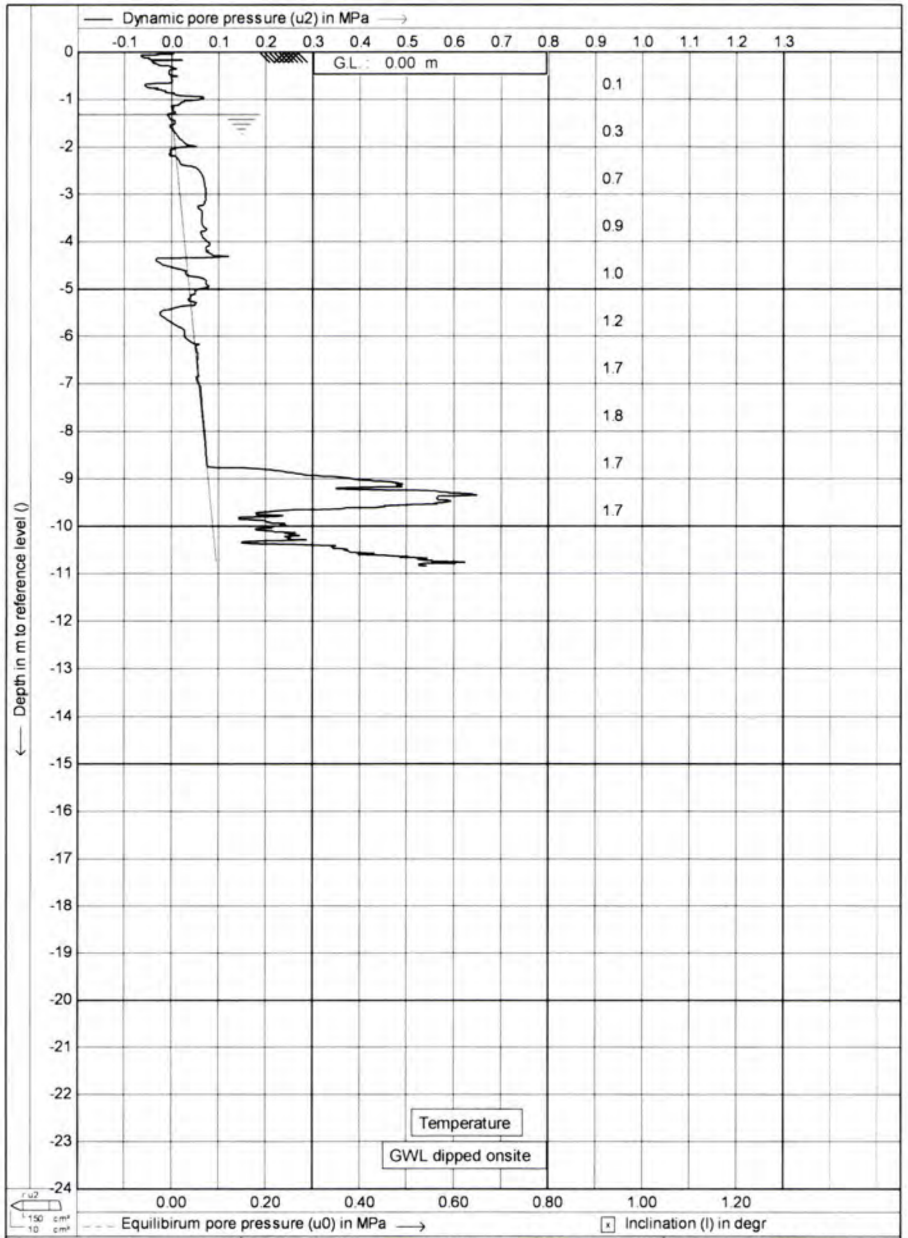
Position: 0, 0 RD

Date : 30/07/2018

Cone no. : C10CFIPT.C09015

Project no. : 05TT08

CPT no. : 03 1/15

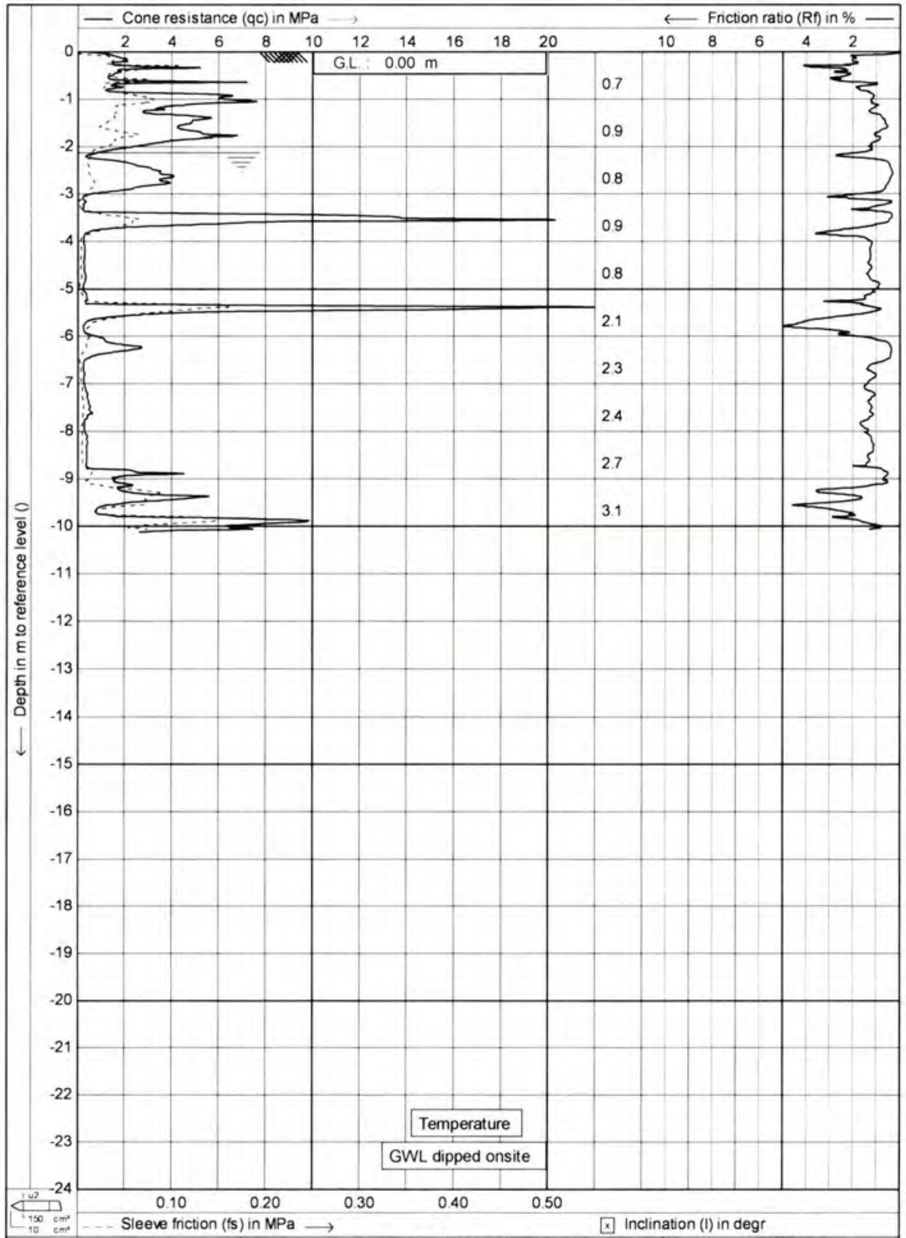


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **30/07/2018**
Cone no. : **C10CFIIPT.C09015**
Project no. : **05TT08**
CPT no. : **03** **2/15**

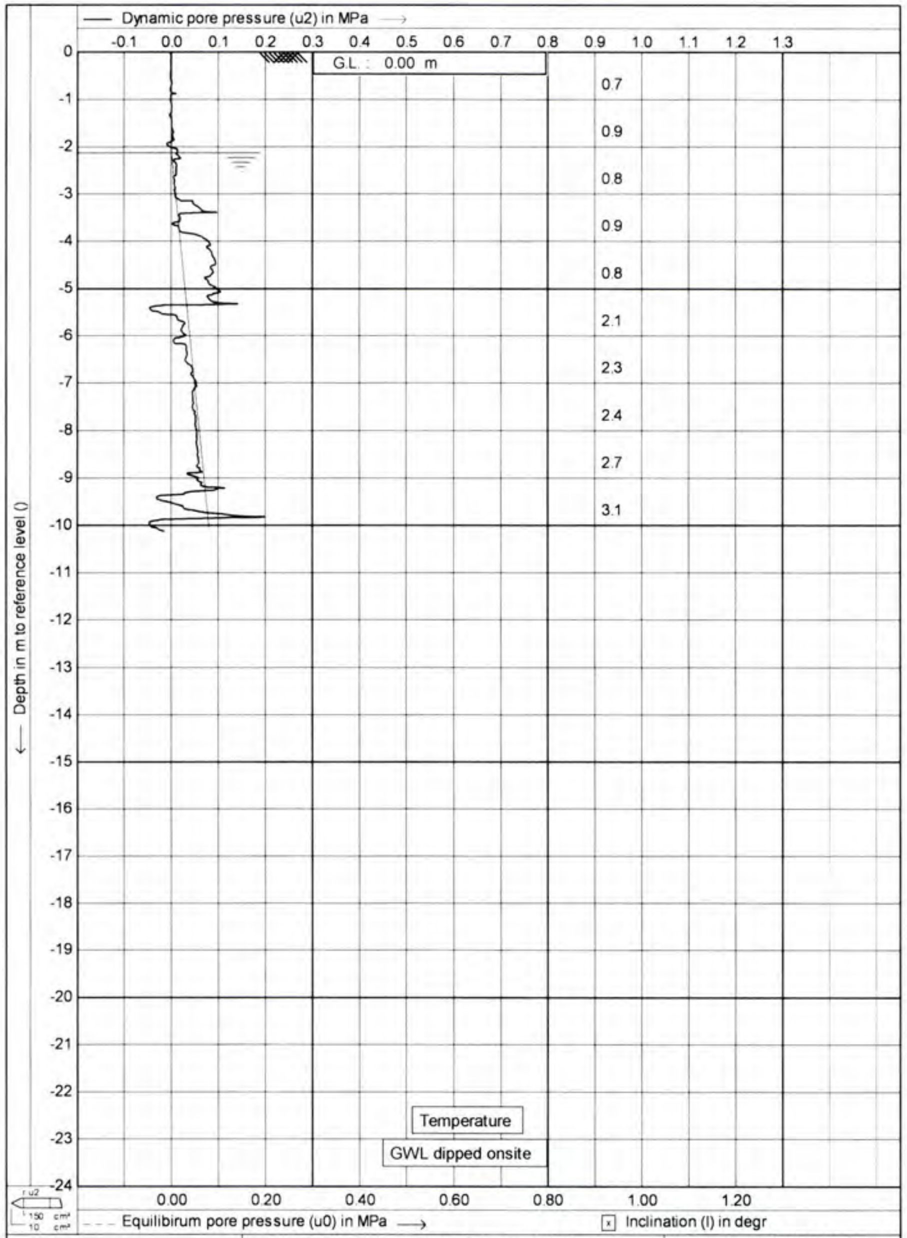


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

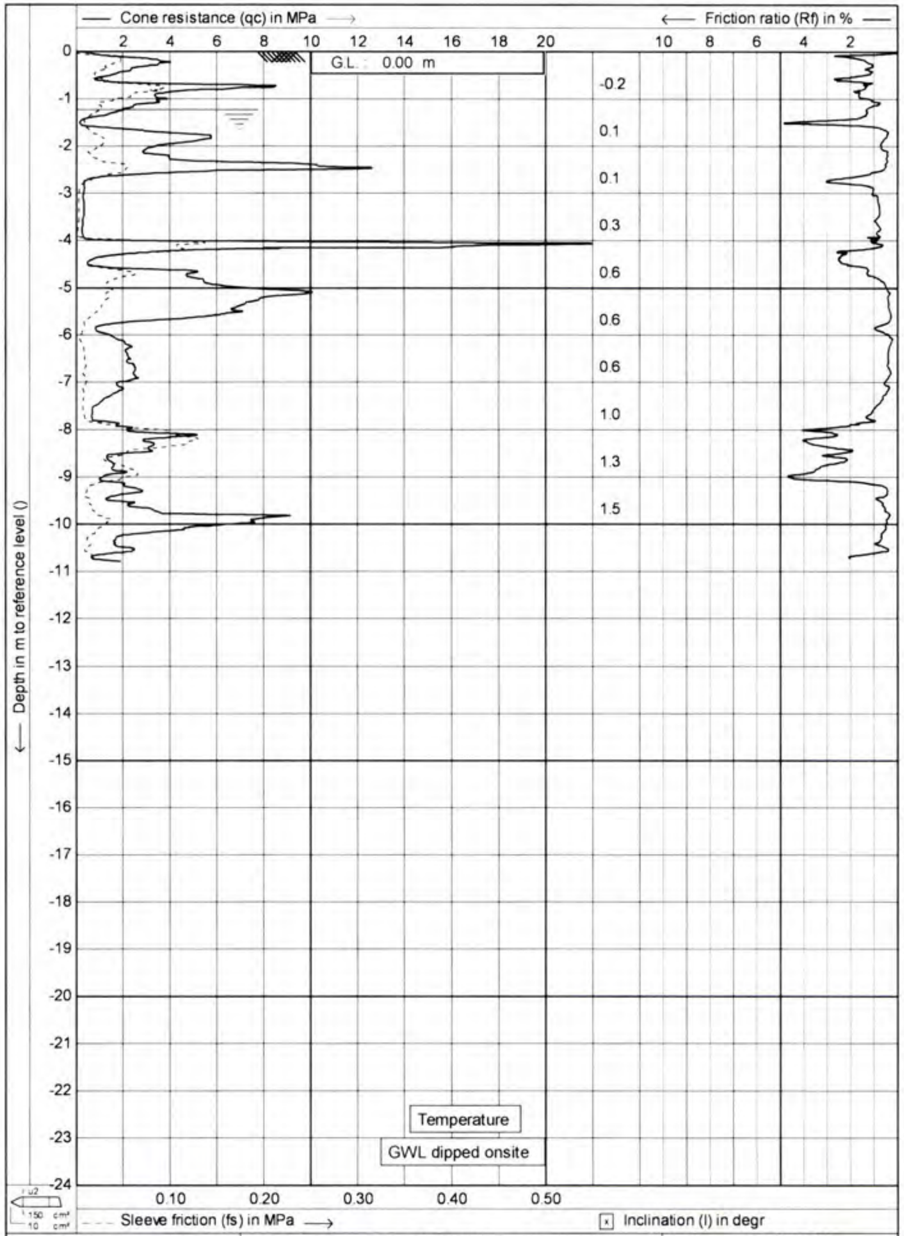
Date : **30/07/2018**
Cone no. : **C10CFIIPT.C09015**
Project no. : **05TT08**
CPT no. : **04** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12
Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

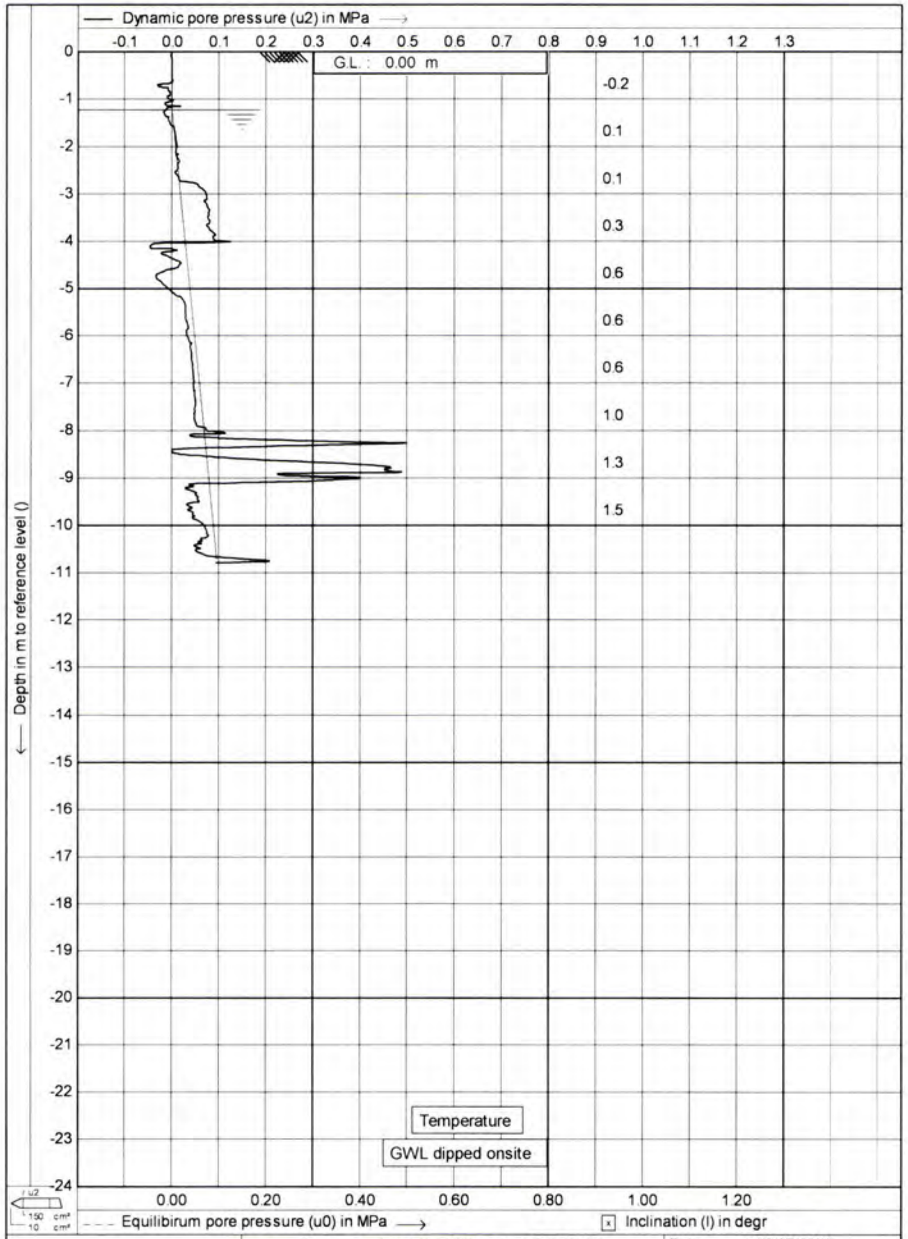
Date : **30/07/2018**
Cone no. : **C10CFIPT.C09015**
Project no. : **05TT08**
CPT no. : **04** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12
Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **30/07/2018**
Cone no. : **C10CFIPT.C09015**
Project no. : **05TT08**
CPT no. : **05** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **Rotorua Lakefront - Rotorua**

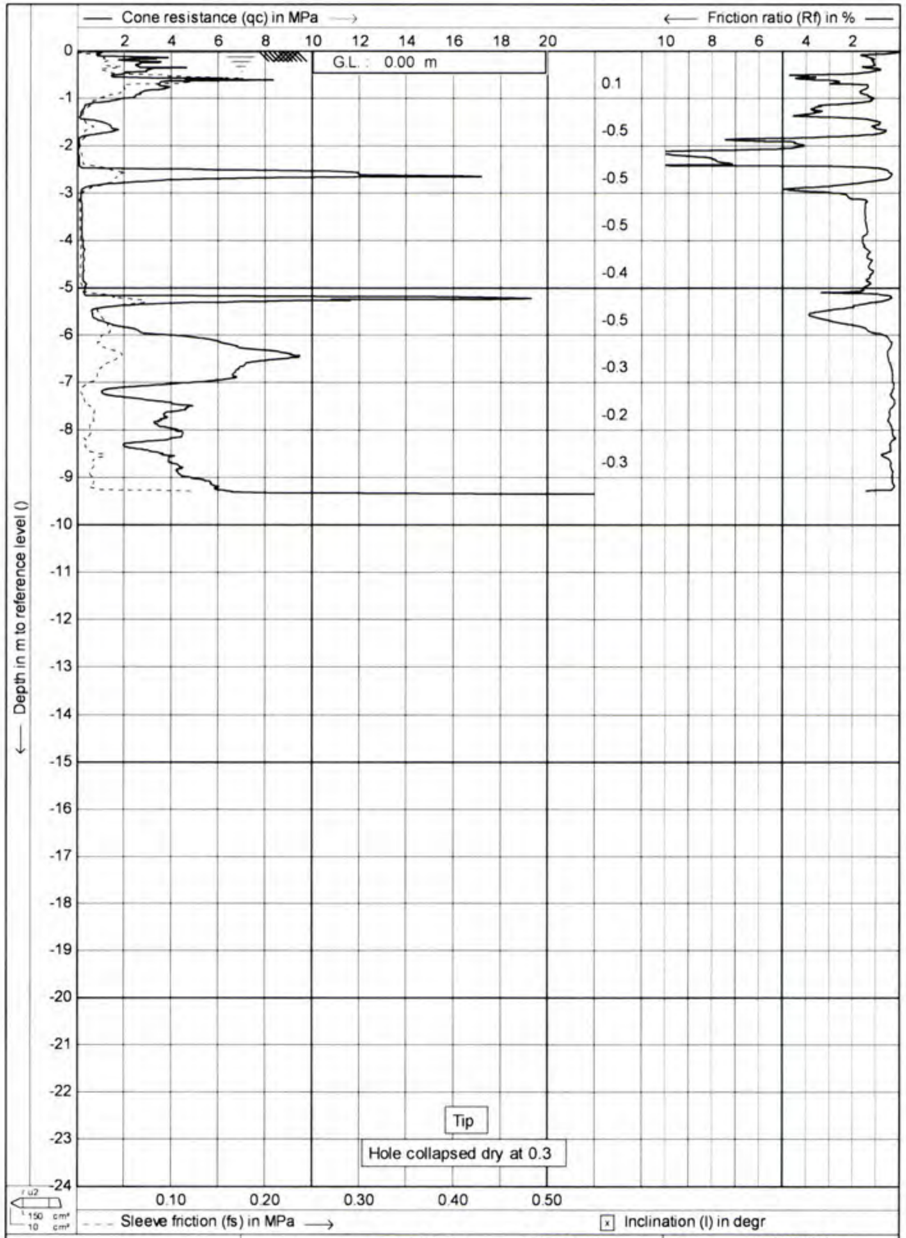
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CFIPT.C09015**

Project no. : **05TT08**

CPT no. : **05** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **Rotorua Lakefront - Rotorua**

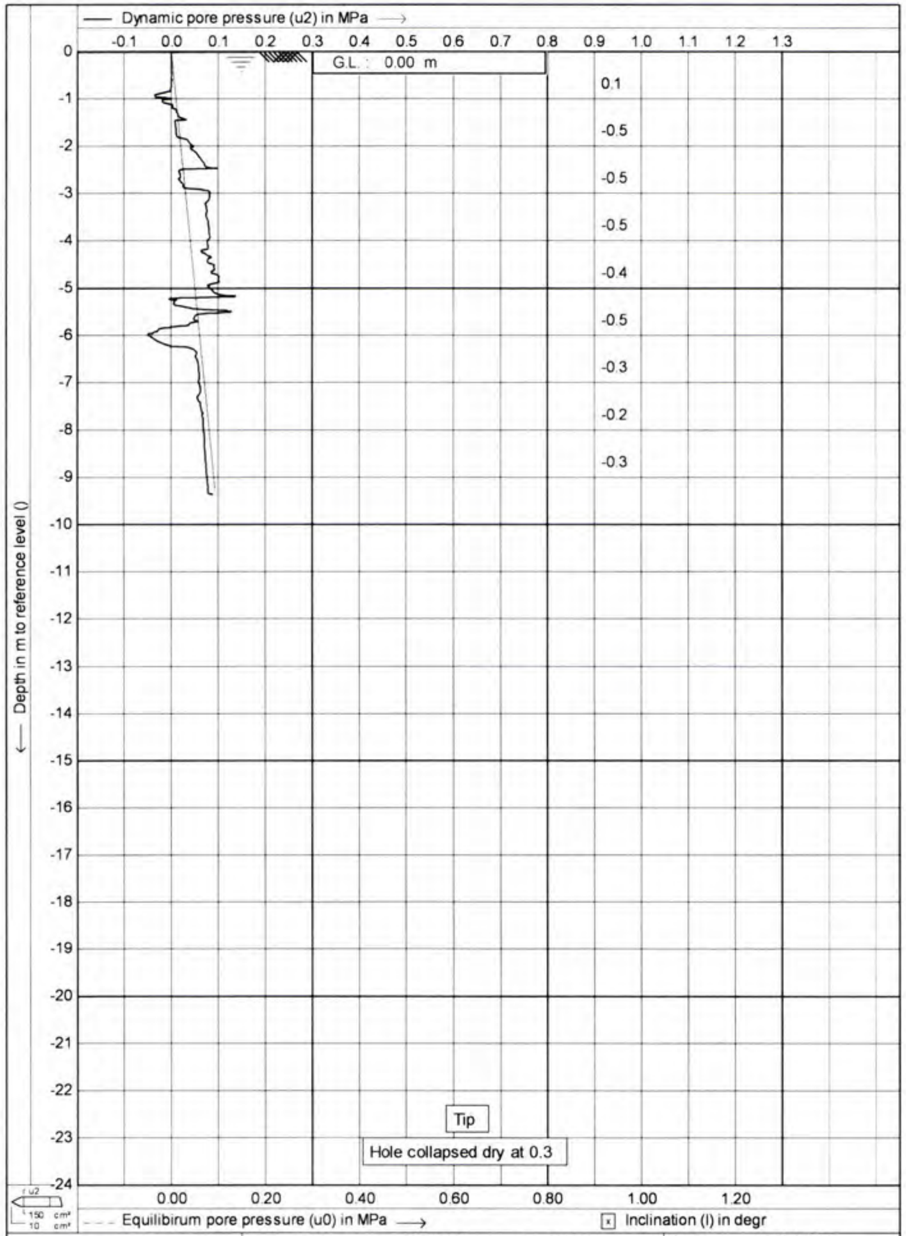
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CFIPT.C09015**

Project no. : **05TT08**

CPT no. : **06** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **Rotorua Lakefront - Rotorua**

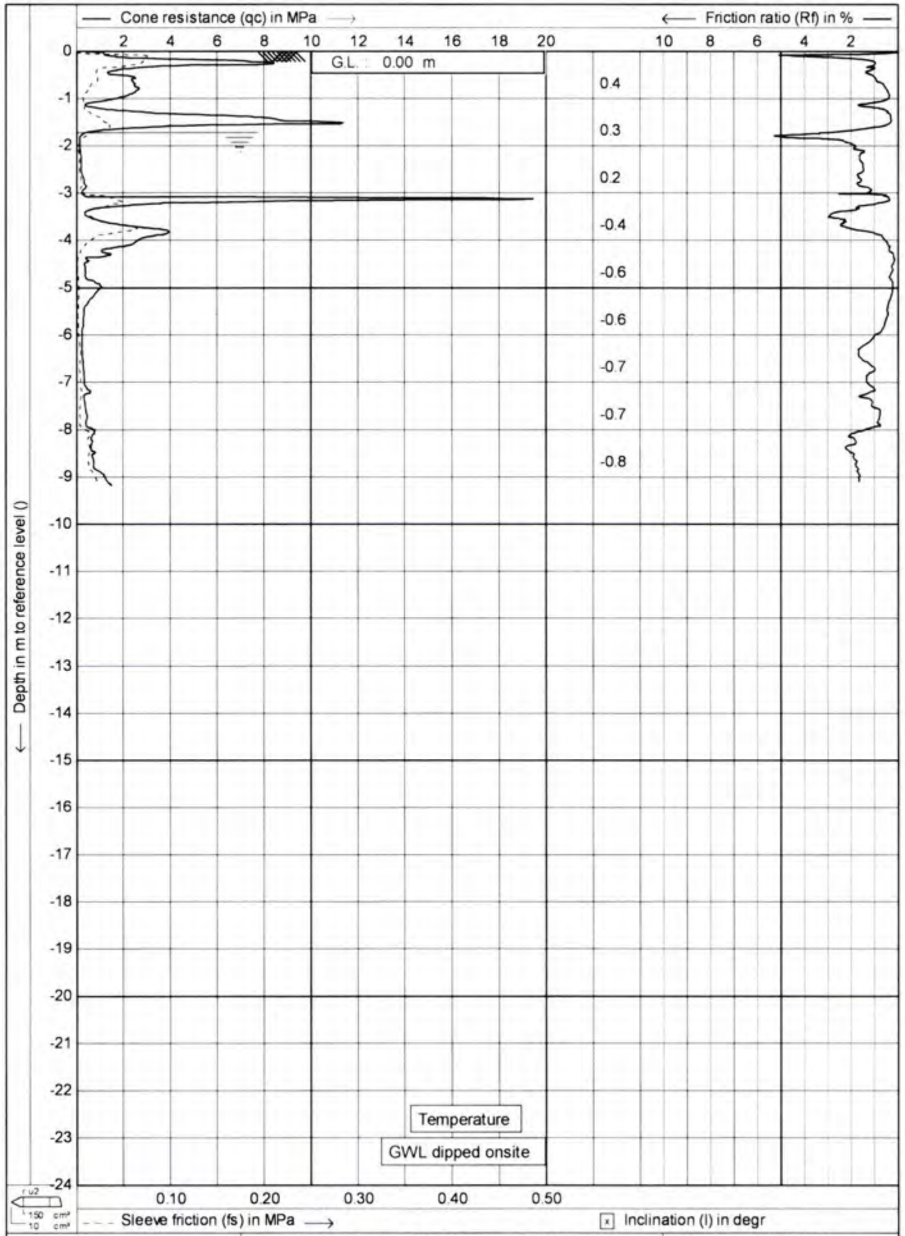
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CRPT.C09015**

Project no. : **05TT08**

CPT no. : **06** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **Rotorua Lakefront - Rotorua**

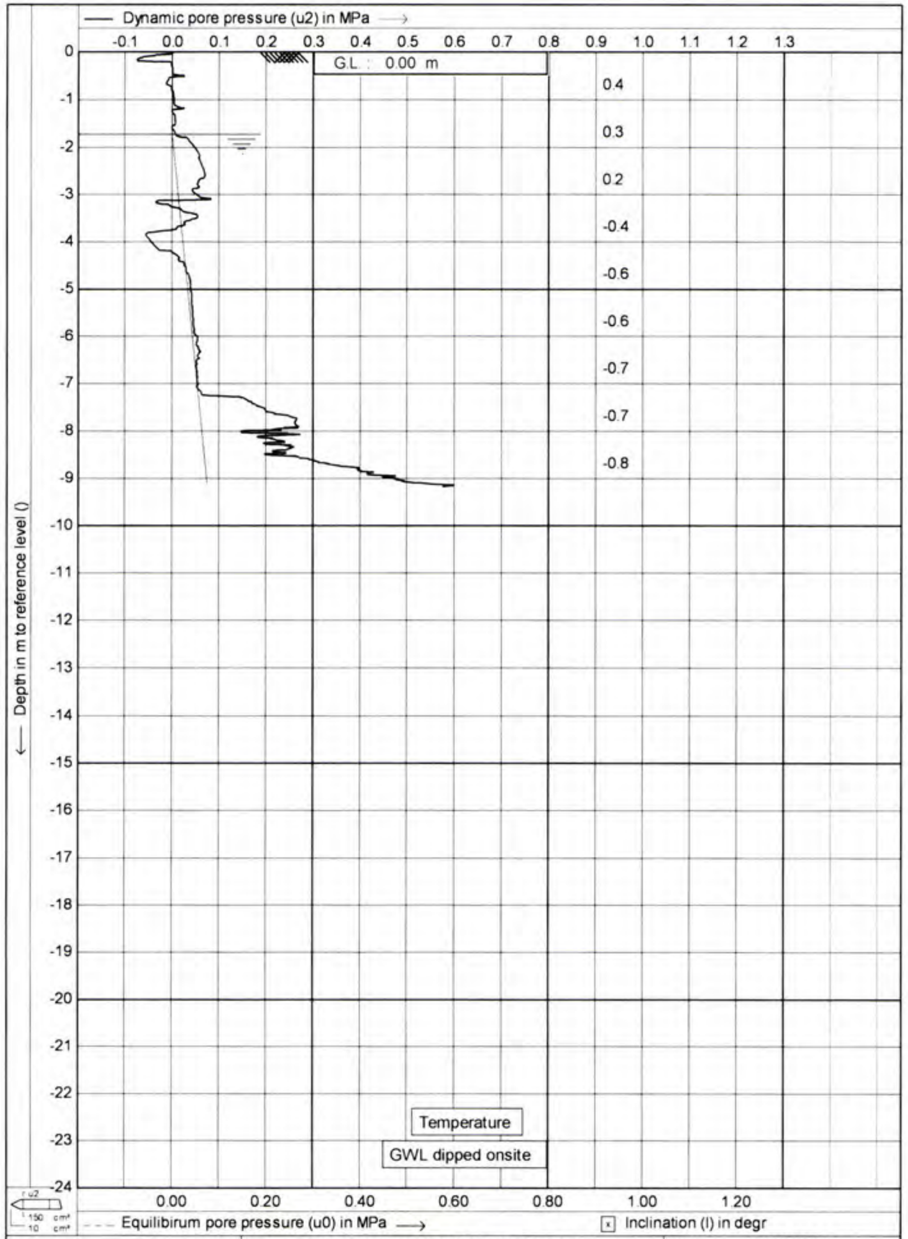
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CRPT.C09015**

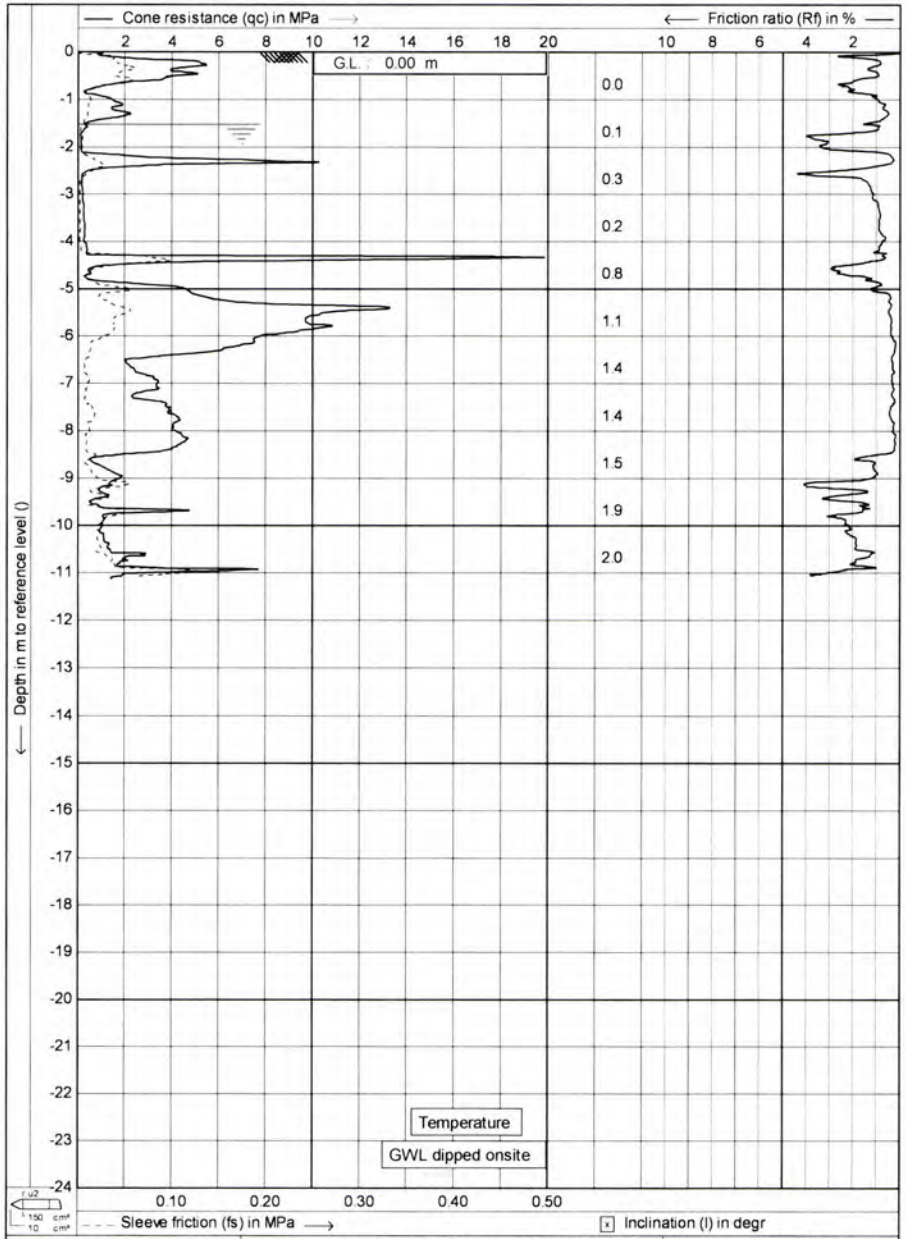
Project no. : **05TT08**

CPT no. : **07** **1/15**



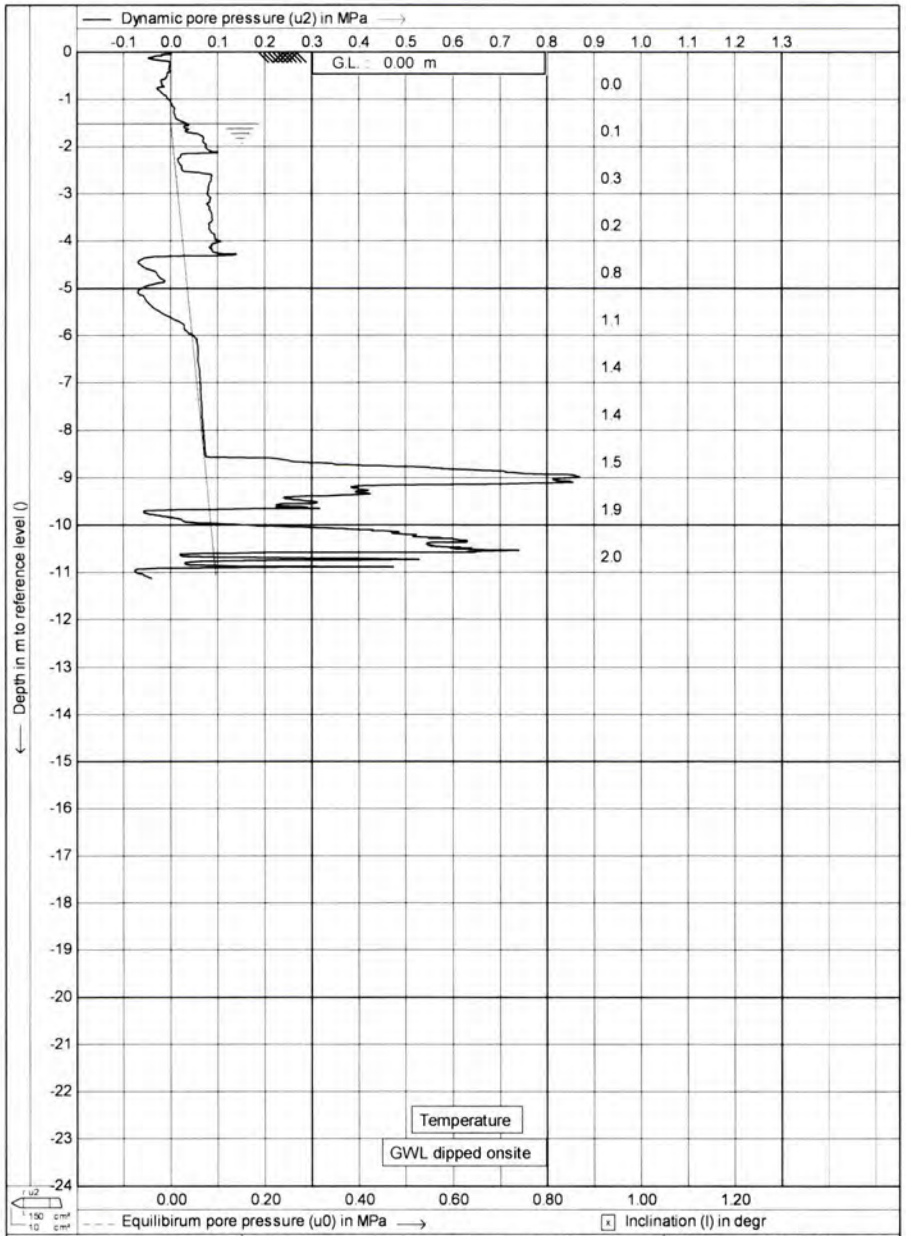
PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12	Date : **30/07/2018**
Project : **SIte Investigations**	Cone no. : **C10CFIPT.C09015**
Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
Position: **0, 0 RD**	CPT no. : **07** **2/15**

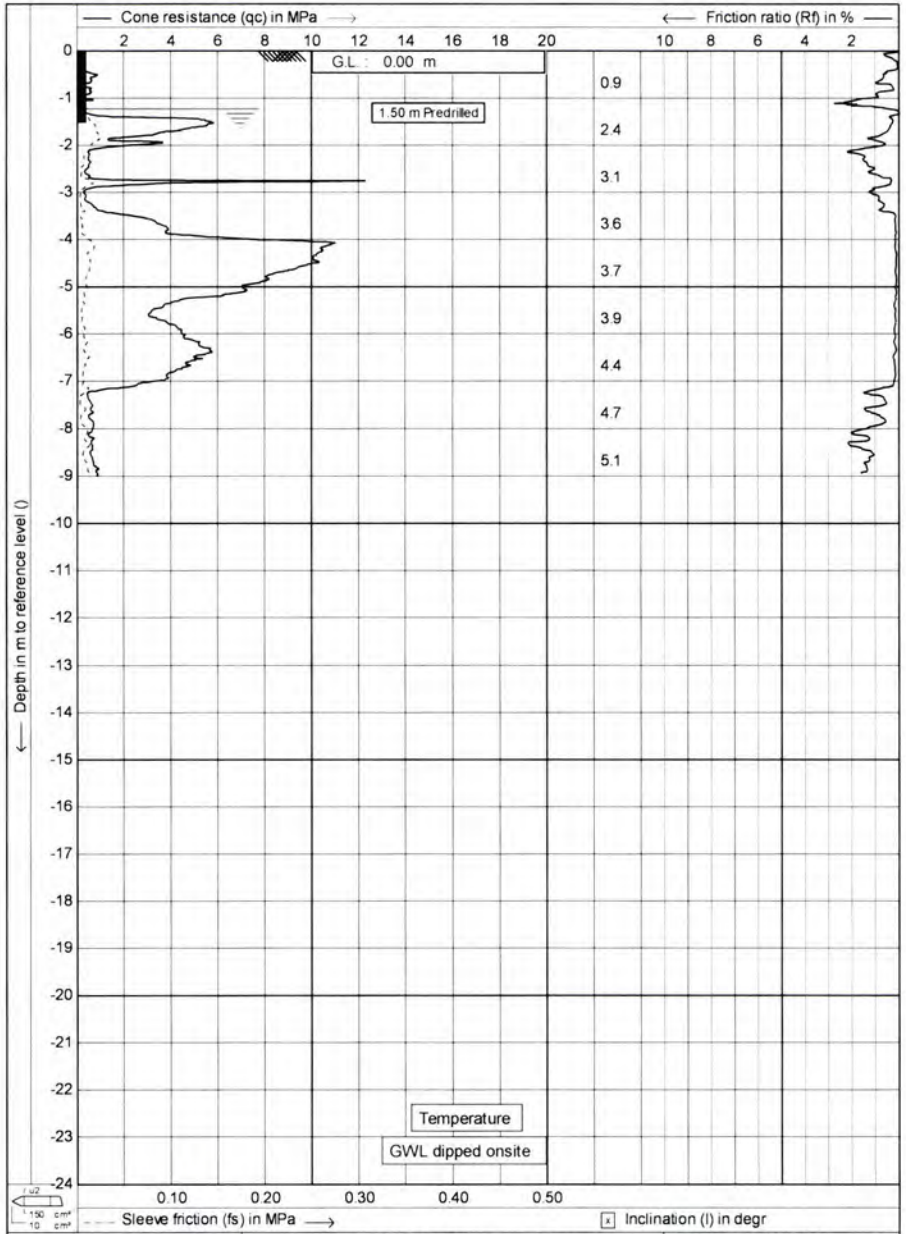


PERRY GEOTECH

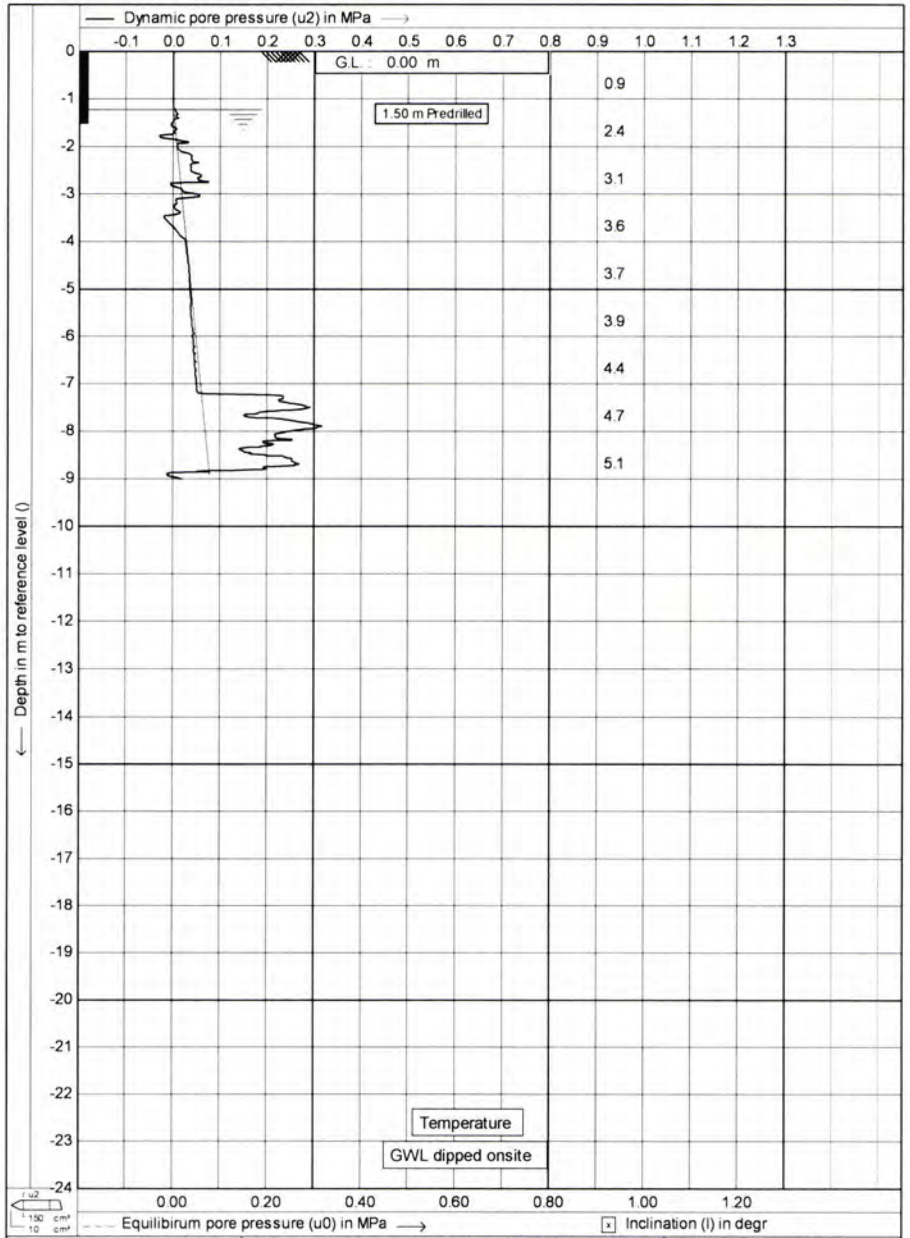
Test according A.S.T.M Standard D 5778-12	Date : **30/07/2018**
Project : **SIte Investigations**	Cone no. : **C10CFIPT.C09015**
Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
Position: **0, 0 RD**	CPT no. : **08** **1/15**



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **30/07/2018**
	Project : **Site Investigations**	Cone no. : **C10CFPT.C09015**
	Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
	Position: **0, 0 RD**	CPT no. : **08** **2/15**



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **1/08/2018**
	Project : **Site Investigations**	Cone no. : **C10CFIPT.C18265**
	Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
	Position: **0, 0 RD**	CPT no. : **09** **1/15**

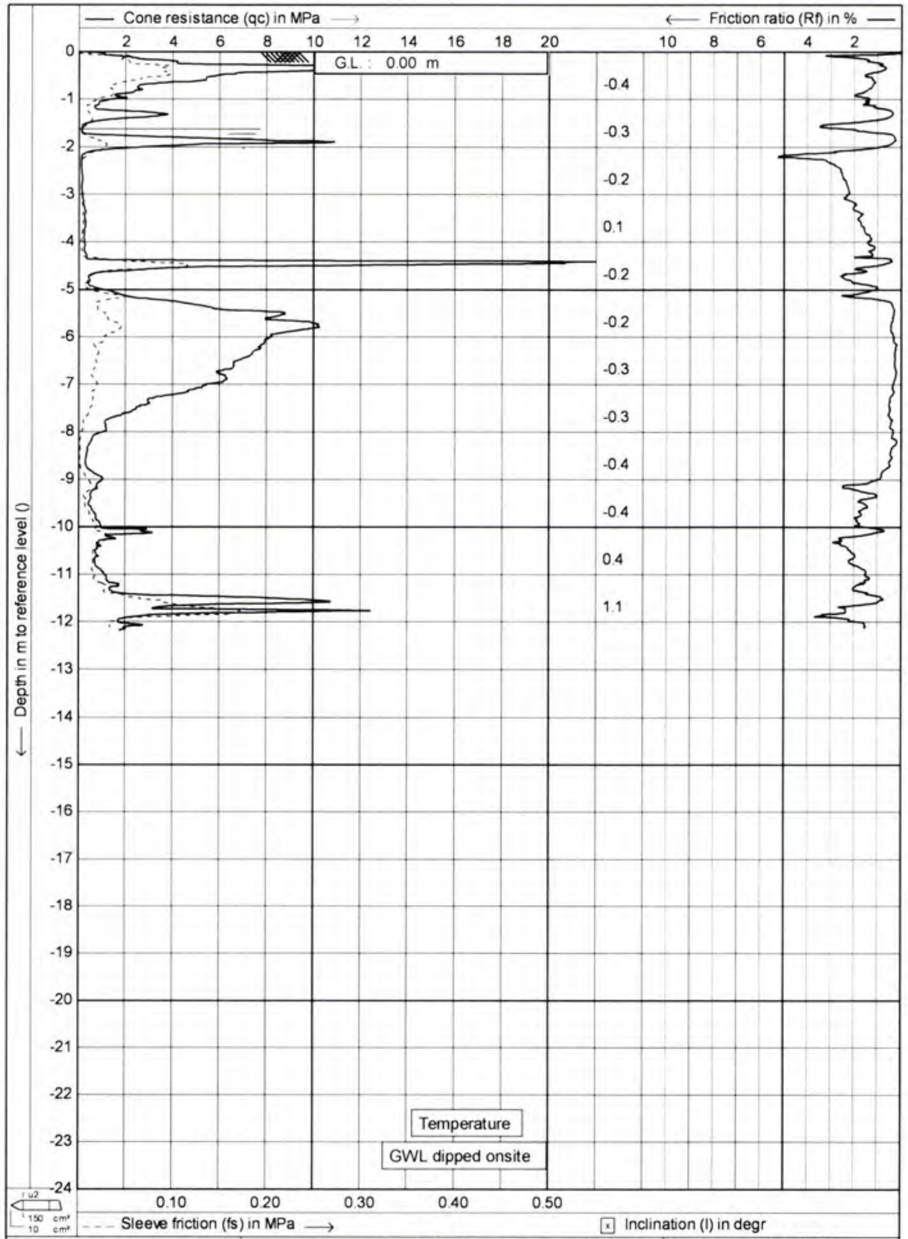


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **1/08/2018**
Cone no. : **C10CFIIPT.C18265**
Project no. : **05TT08**
CPT no. : **09** **2/15**

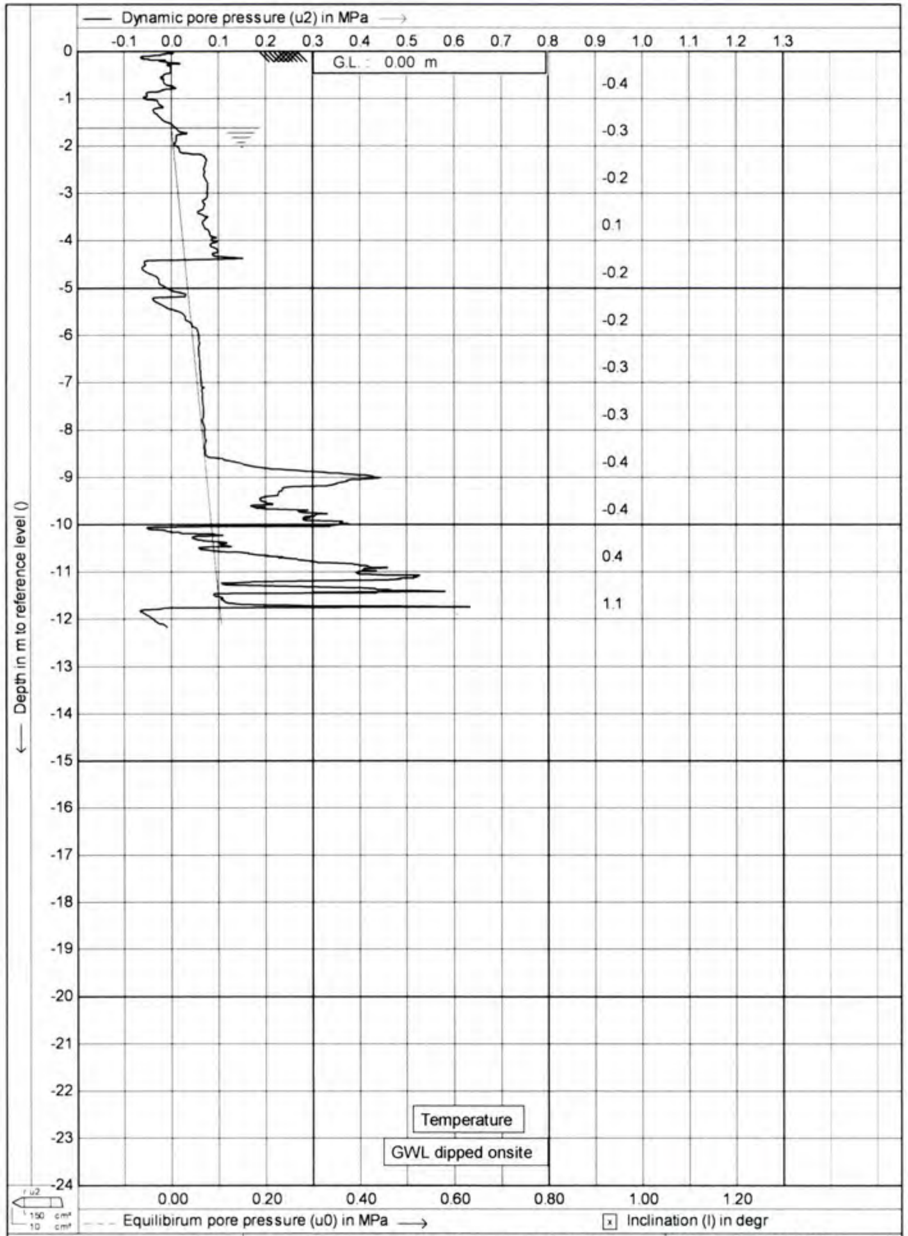


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **30/07/2018**
Cone no. : **C10CFIIPT.C09015**
Project no. : **05TT08**
CPT no. : **10** **1/15**

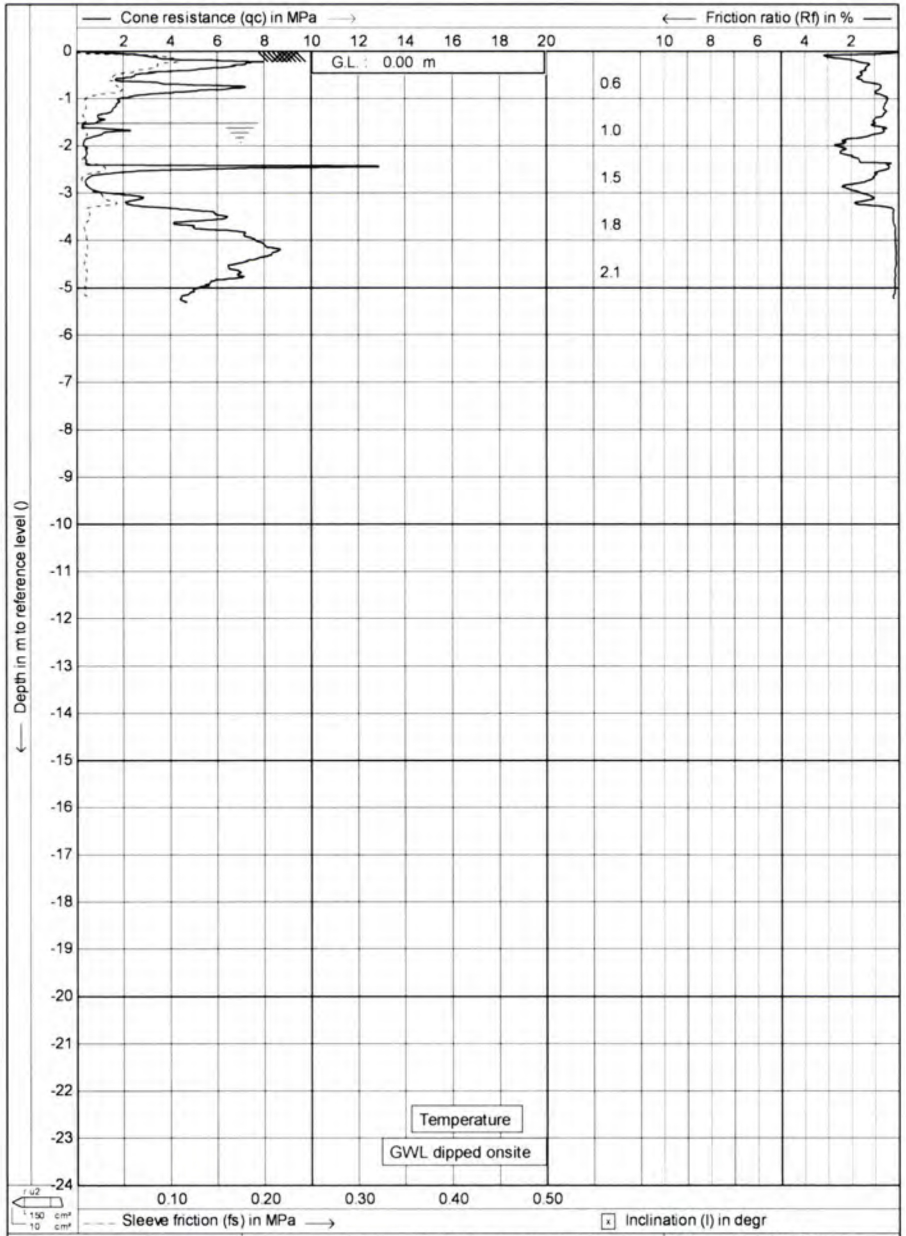


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date	: **30/07/2018**
Cone no.	: **C10CFIPT.C09015**
Project no.	: **05TT08**
CPT no.	: **10**
	2/15

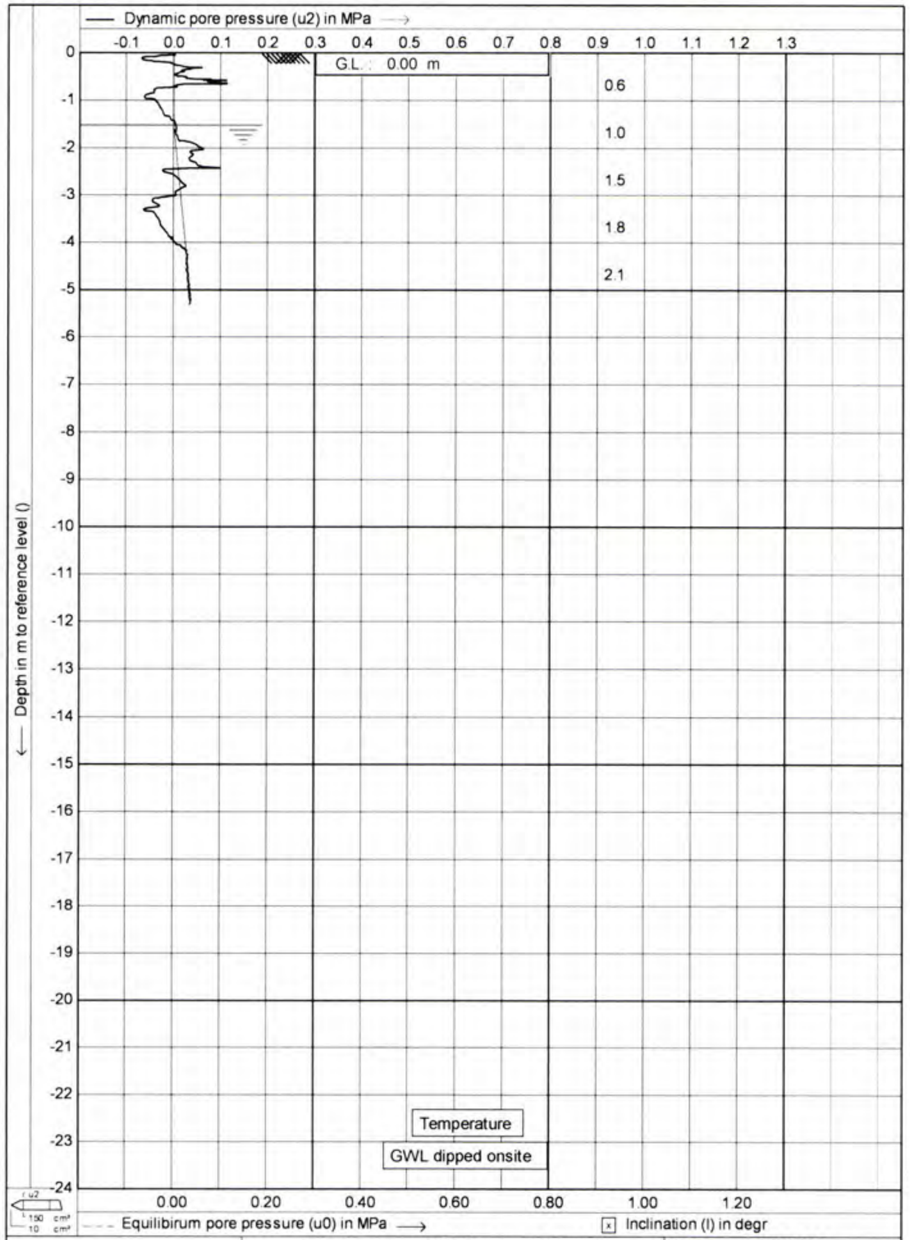


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date	: **1/08/2018**
Cone no.	: **C10CFIPT.C18265**
Project no.	: **05TT08**
CPT no.	: **11**
	1/15



Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

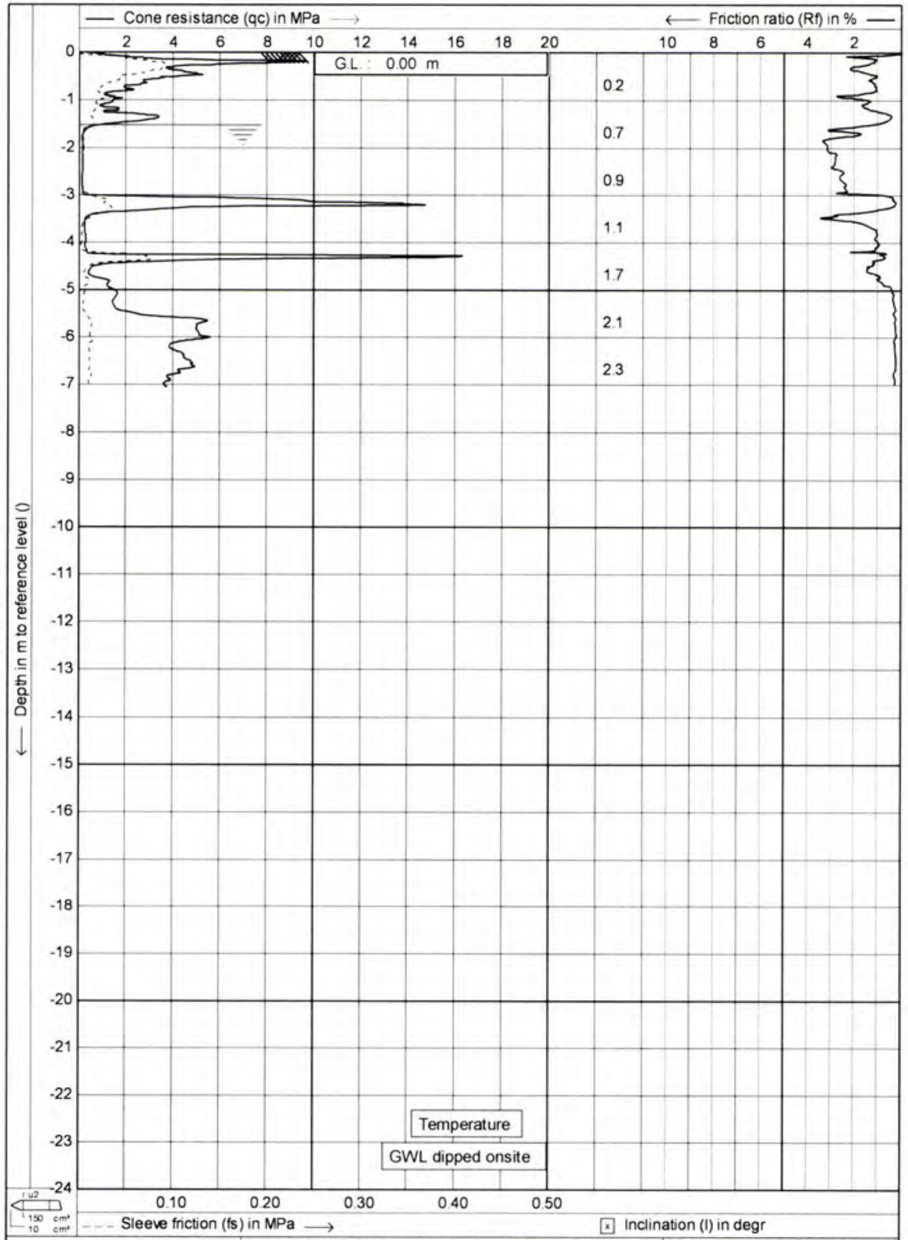
Position: **0, 0 RD**

Date : **1/08/2018**

Cone no. : **C10CRIPT.C18265**

Project no. : **05TT08**

CPT no. : **11** **2/15**



Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

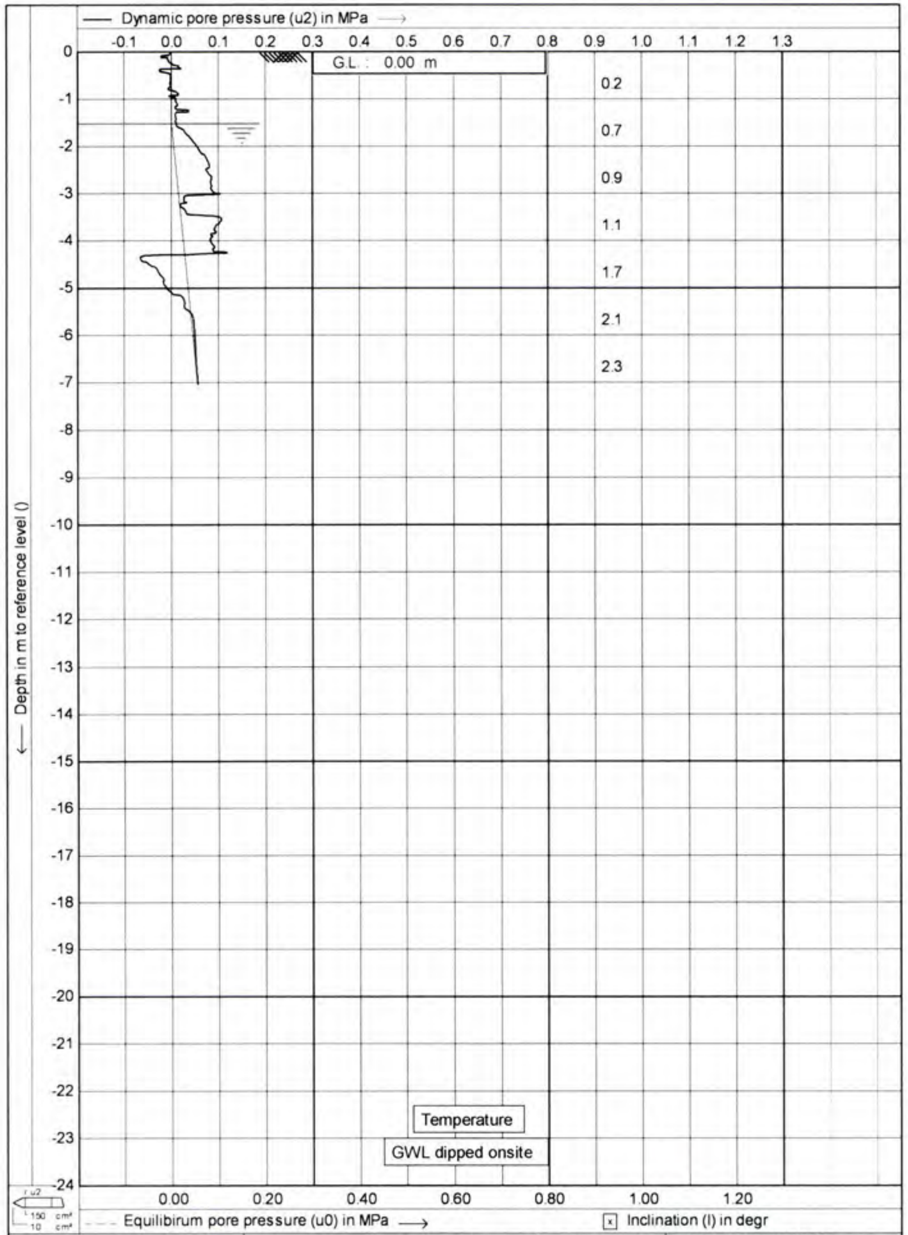
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CRPT.C09015**

Project no. : **05TT08**

CPT no. : **12** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

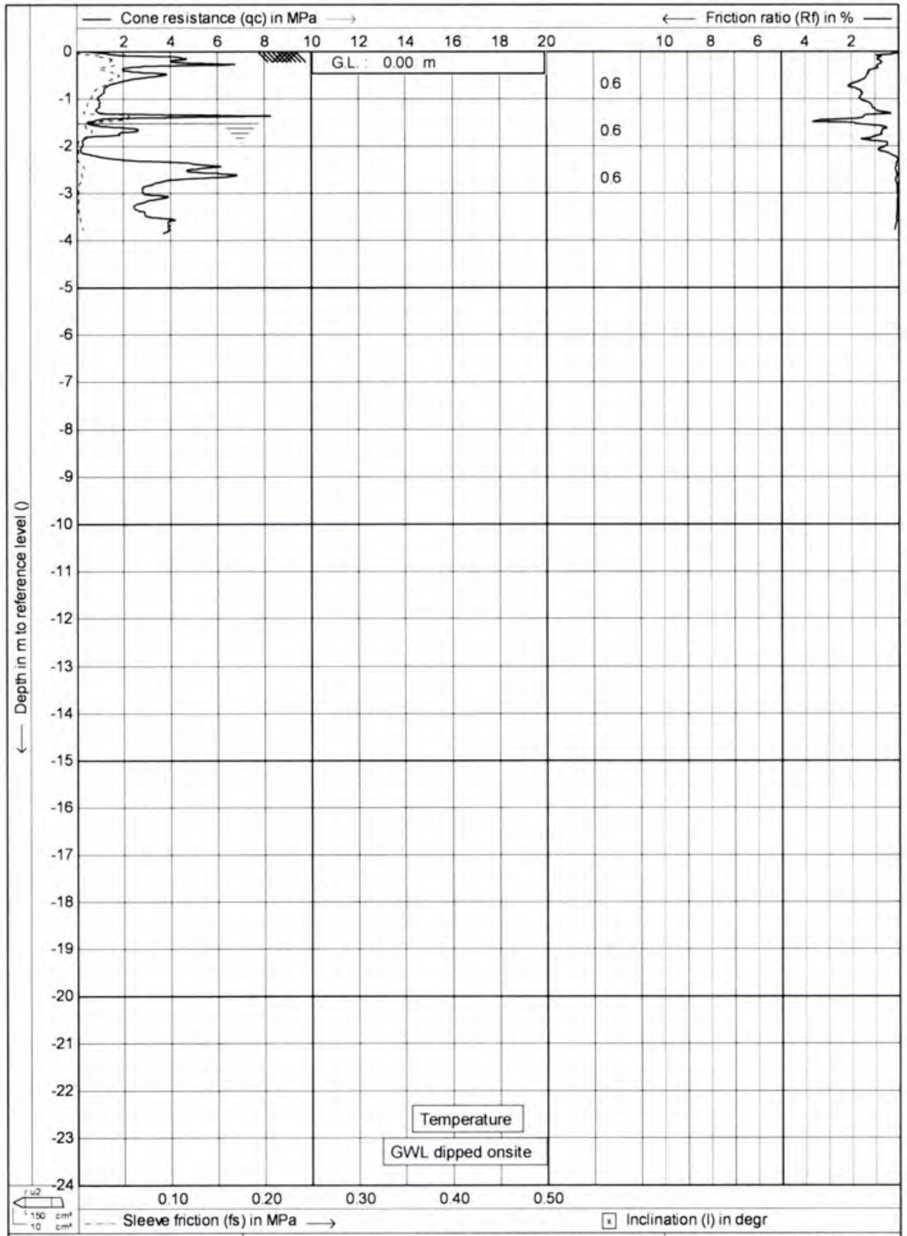
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CFIPT.C09015**

Project no. : **05TT08**

CPT no. : **12** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

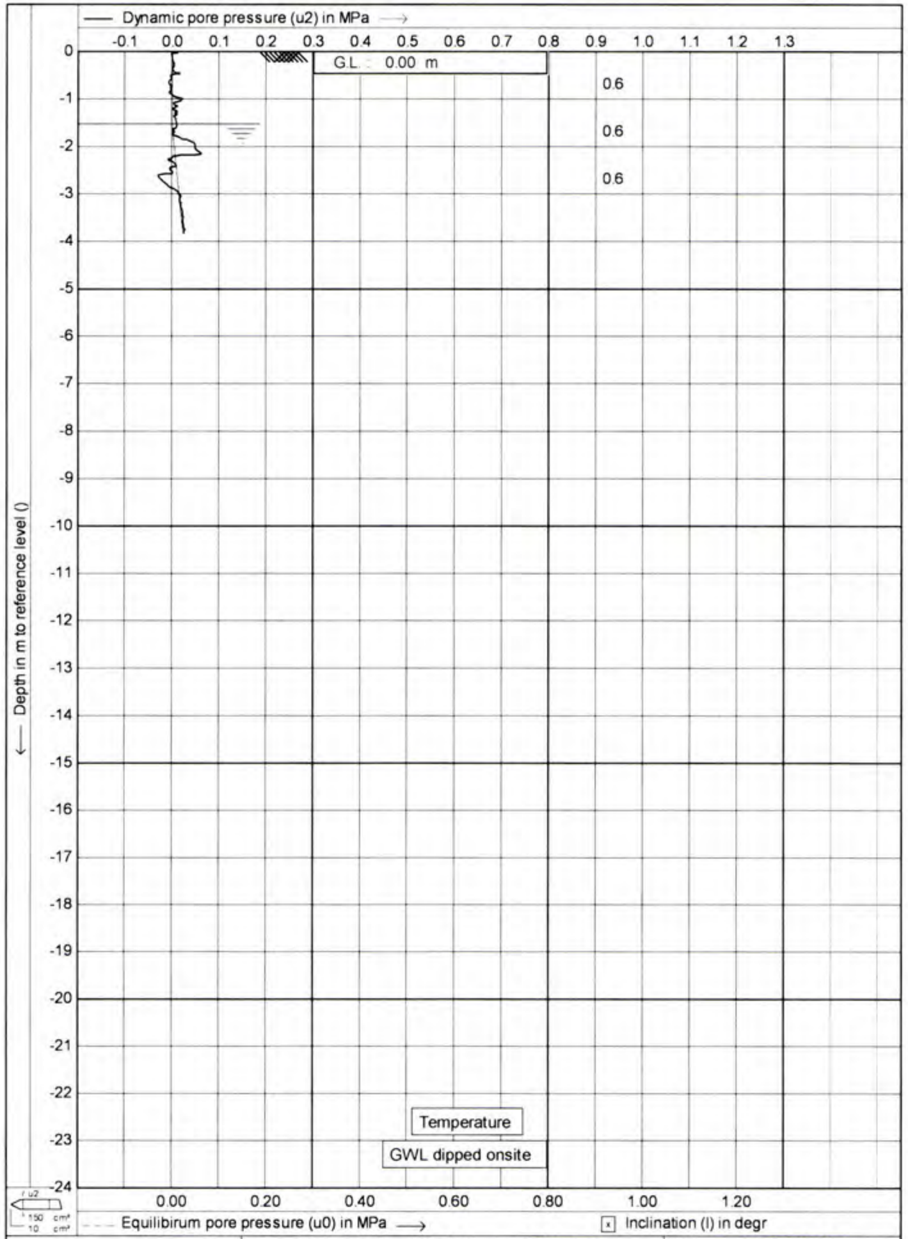
Position: **0, 0 RD**

Date : **31/07/2018**

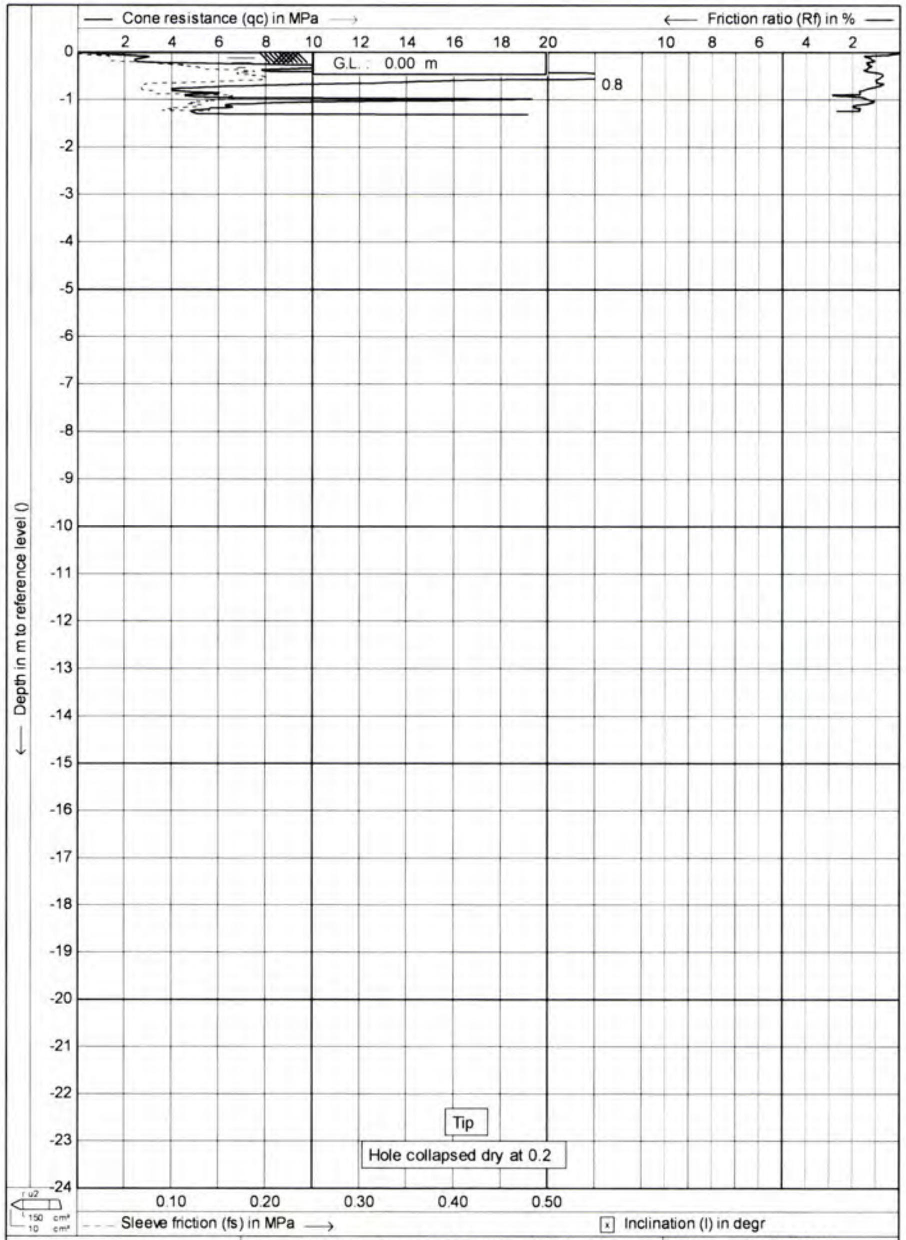
Cone no. : **C10CFIIPT.C18265**

Project no. : **05TT08**

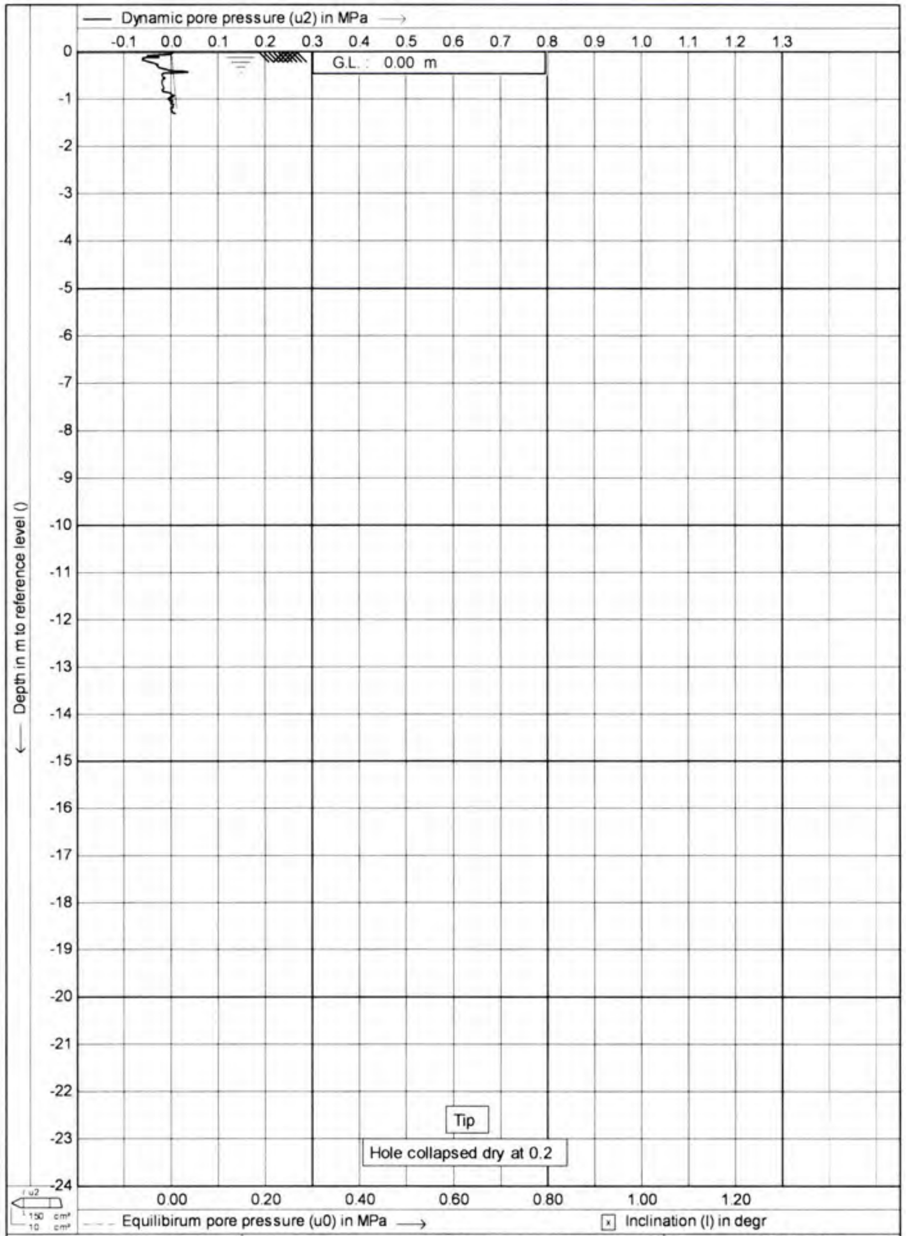
CPT no. : **13** **1/15**



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **31/07/2018**
	Project : **SIte Investigations**	Cone no. : **C10CFIIPT.C18265**
	Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
	Position: **0, 0 RD**	CPT no. : **13** — **2/15**



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **30/07/2018**
	Project : **SIte Investigations**	Cone no. : **C10CFIPT.C09015**
	Location: **Rotorua Lakefront - Rotorua**	Project no. : **05TT08**
	Position: **0, 0 RD**	CPT no. : **14 a** — **1/15**

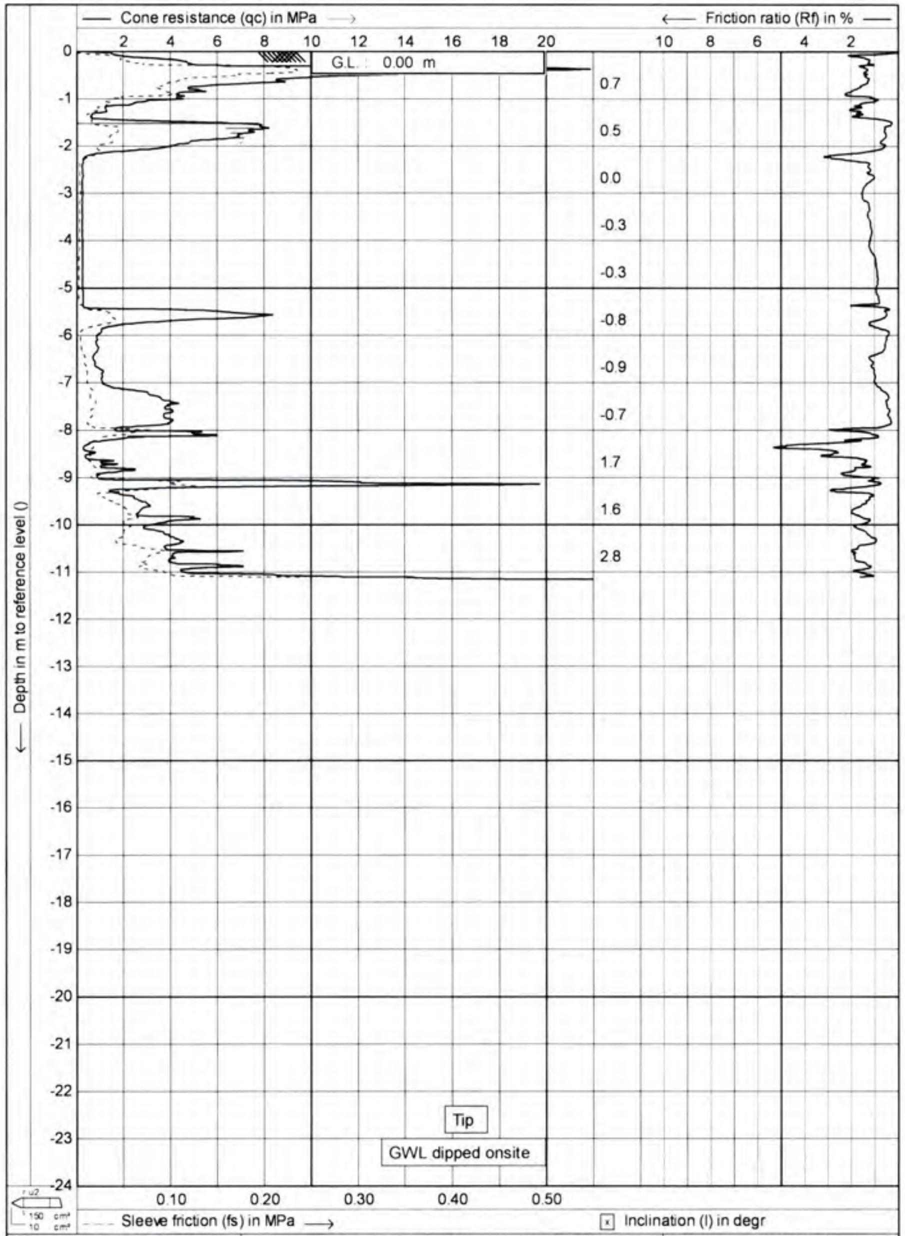


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **30/07/2018**
Cone no. : **C10CFPT.C09015**
Project no. : **05TT08**
CPT no. : **14 a** **2/15**

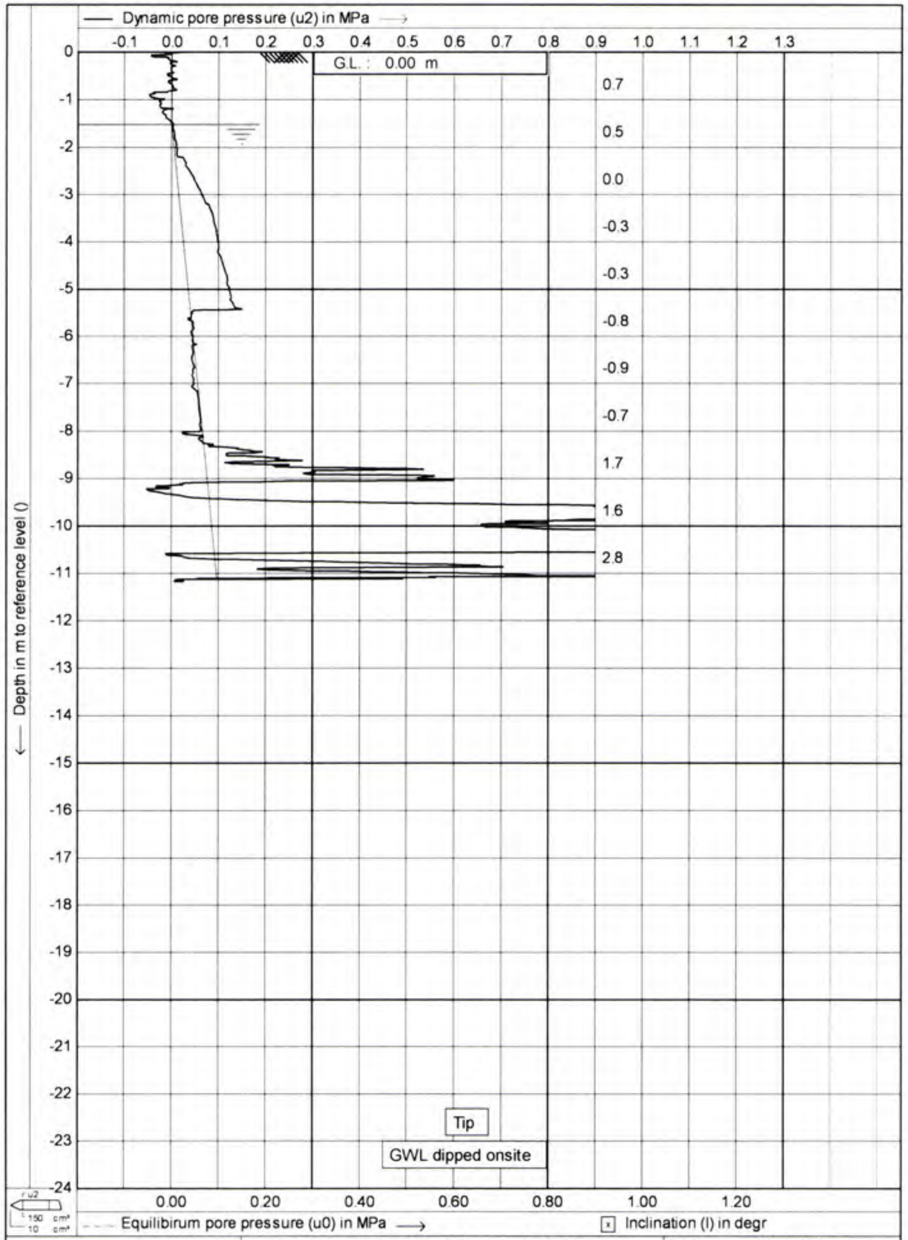


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **Site Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **30/07/2018**
Cone no. : **C10CFPT.C09015**
Project no. : **05TT08**
CPT no. : **15** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

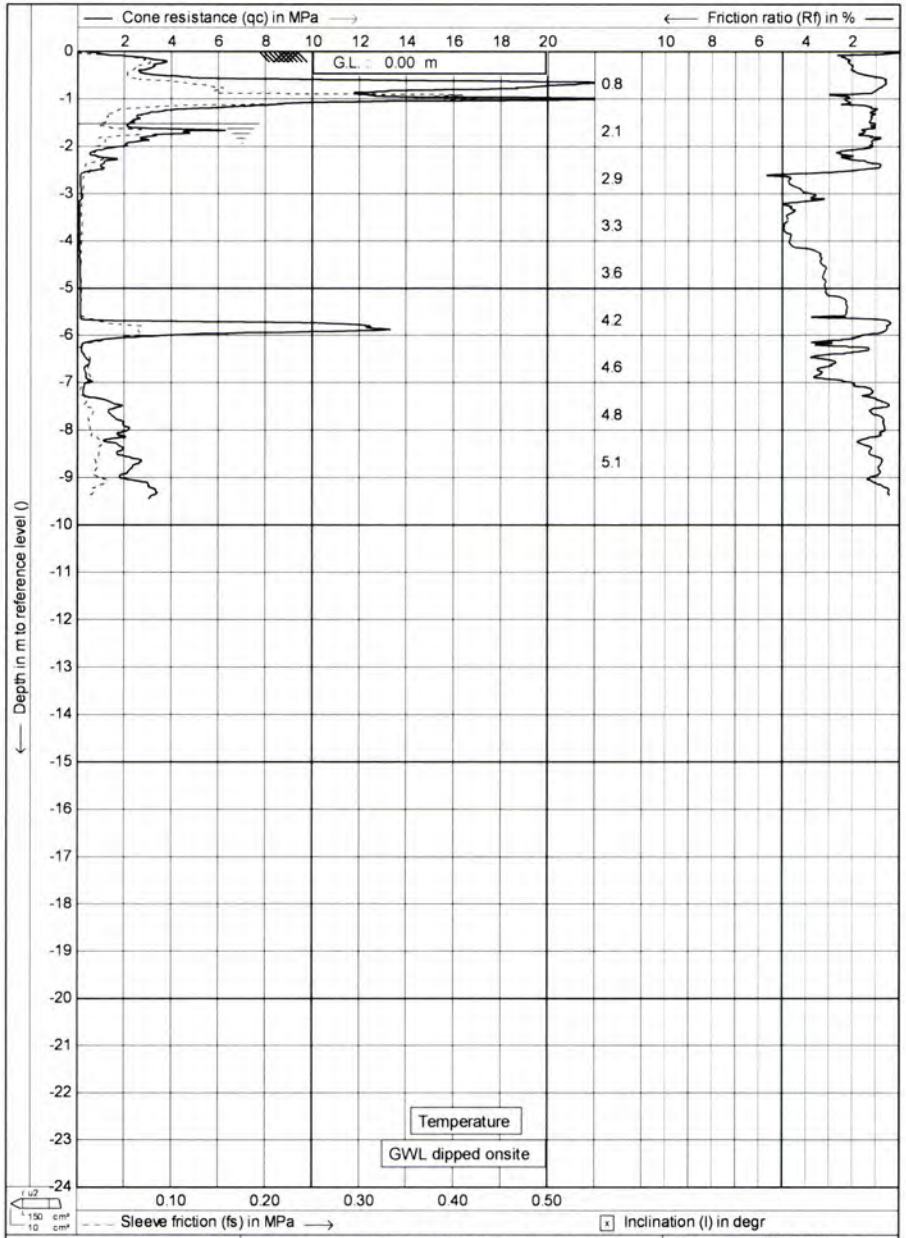
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CFIPT.C09015**

Project no. : **05TT08**

CPT no. : **15** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

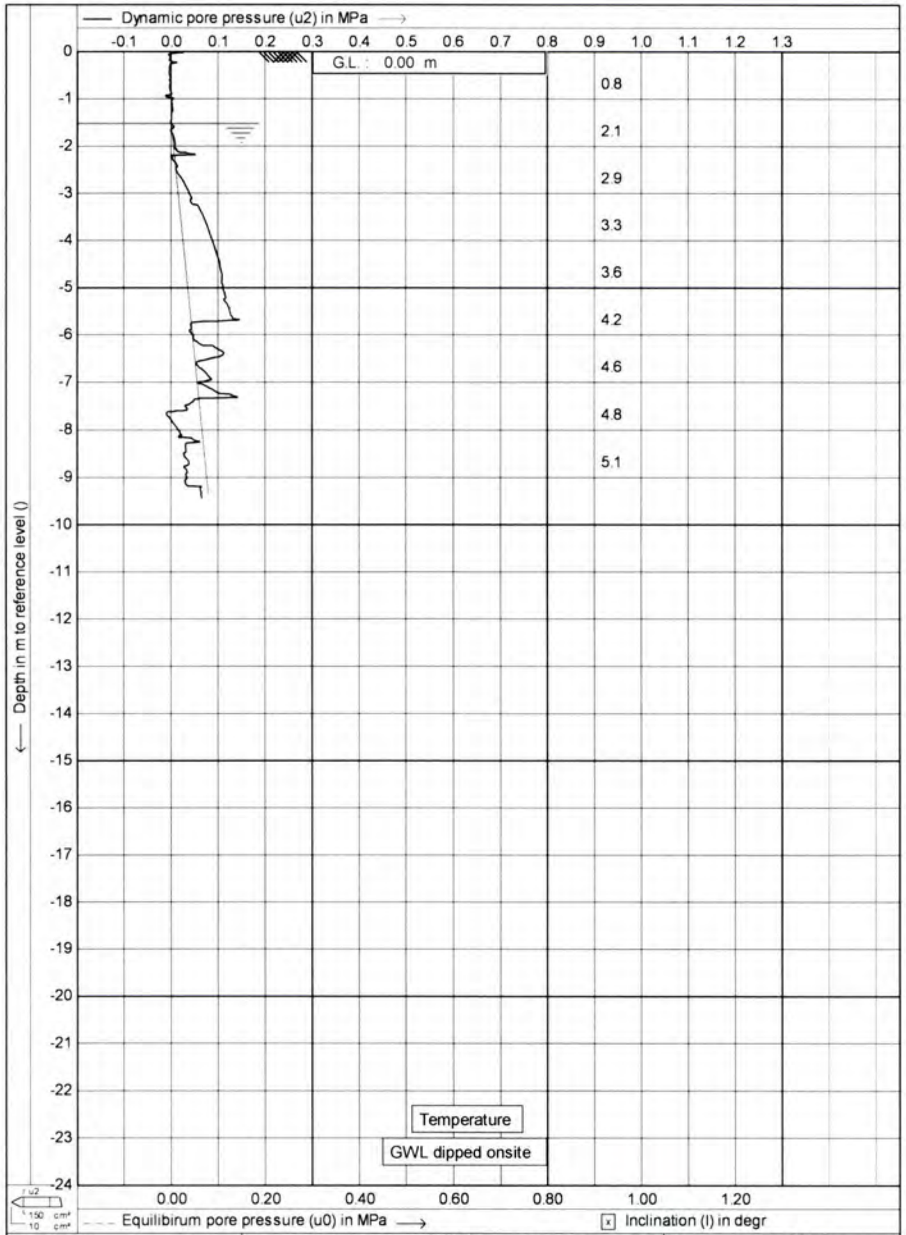
Position: **0, 0 RD**

Date : **30/07/2018**

Cone no. : **C10CFIPT.C09015**

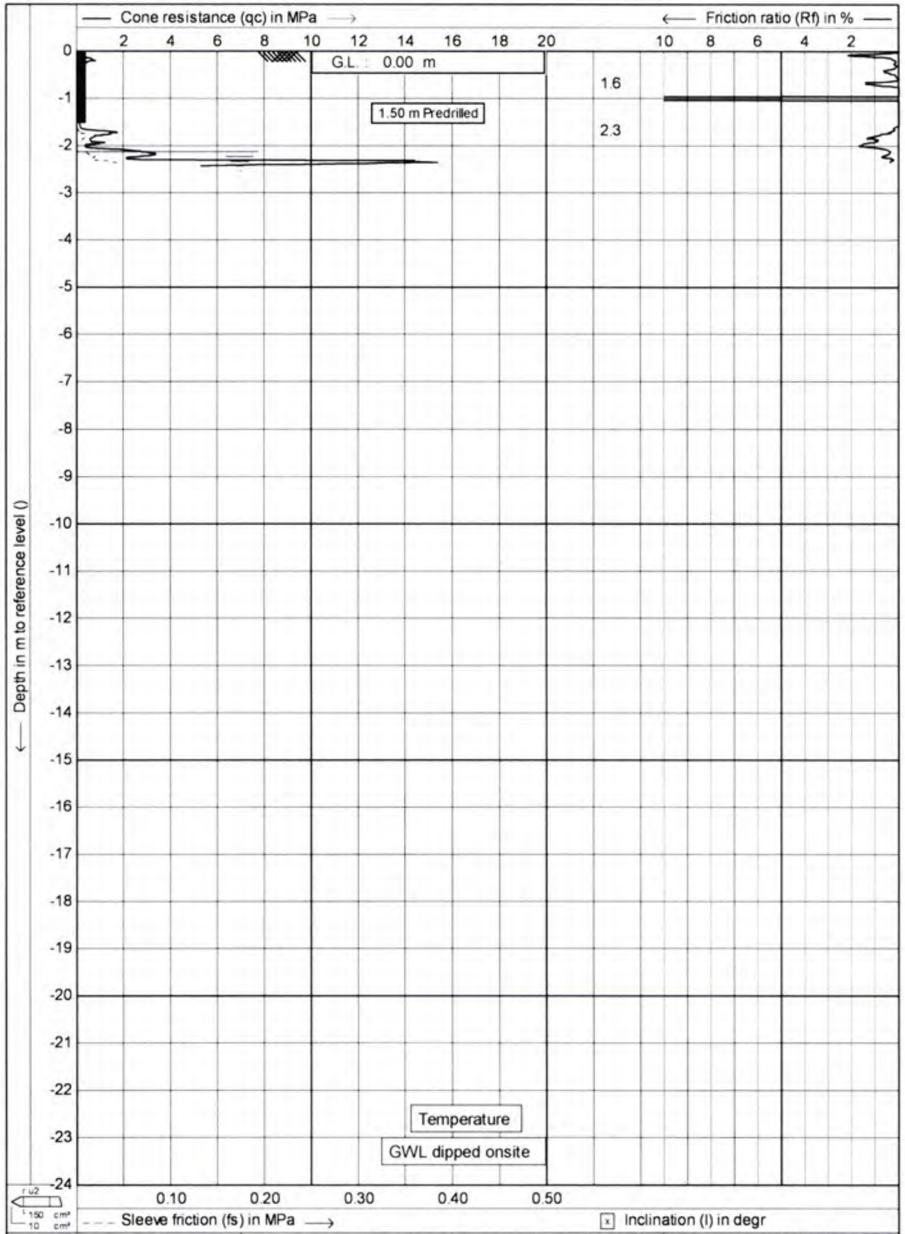
Project no. : **05TT08**

CPT no. : **16** **1/15**



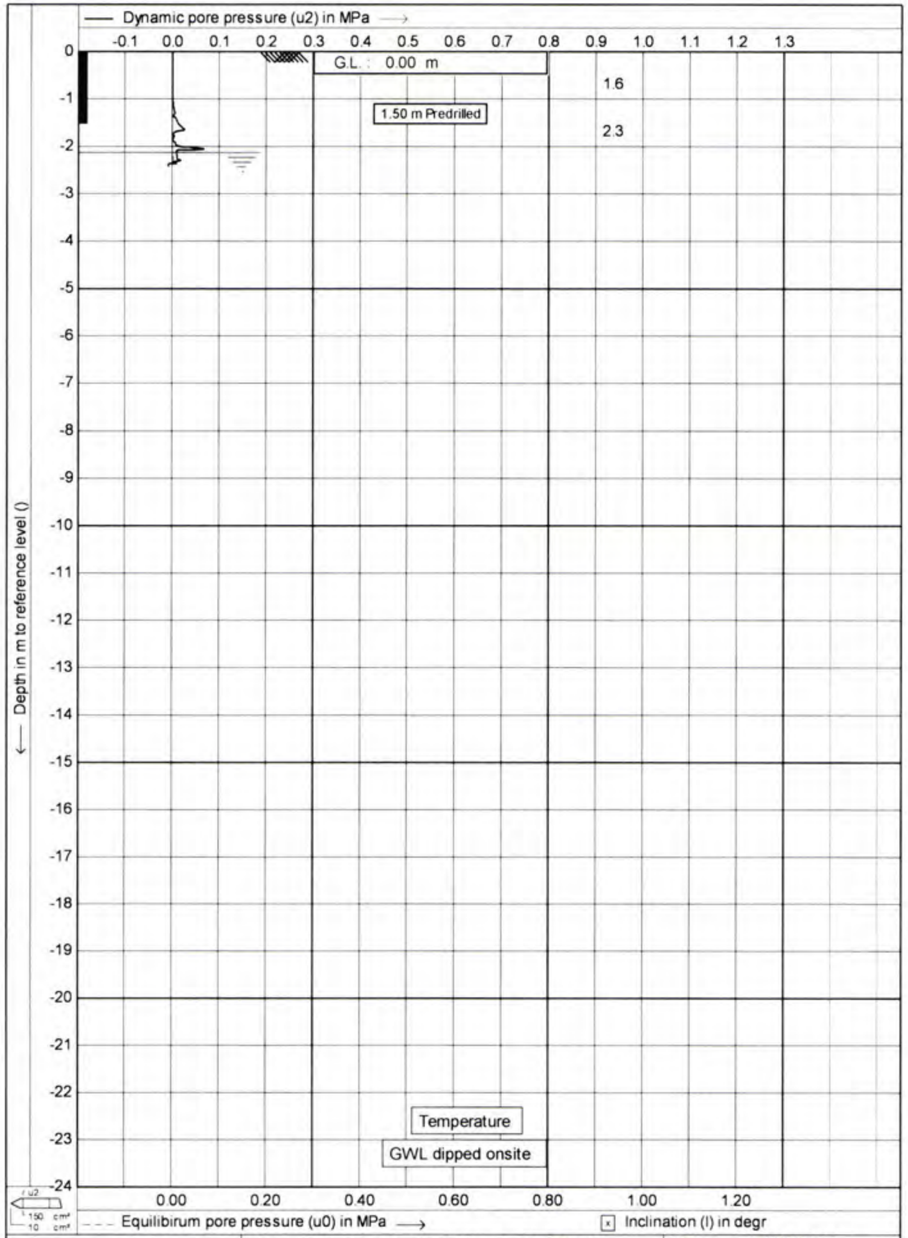
PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12	Date	30/07/2018	
Project : **Site Investigations**	Cone no.	**C10CRPT.C09015**	
Location: **Rotorua Lakefront - Rotorua**	Project no.	**05TT08**	
Position: **0, 0 RD**	CPT no.	**16**	**2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12	Date	31/07/2018	
Project : **Site Investigations**	Cone no.	**C10CRIPT.C18265**	
Location: **Rotorua Lakefront - Rotorua**	Project no.	**05TT08**	
Position: **0, 0 RD**	CPT no.	**17 a**	**1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

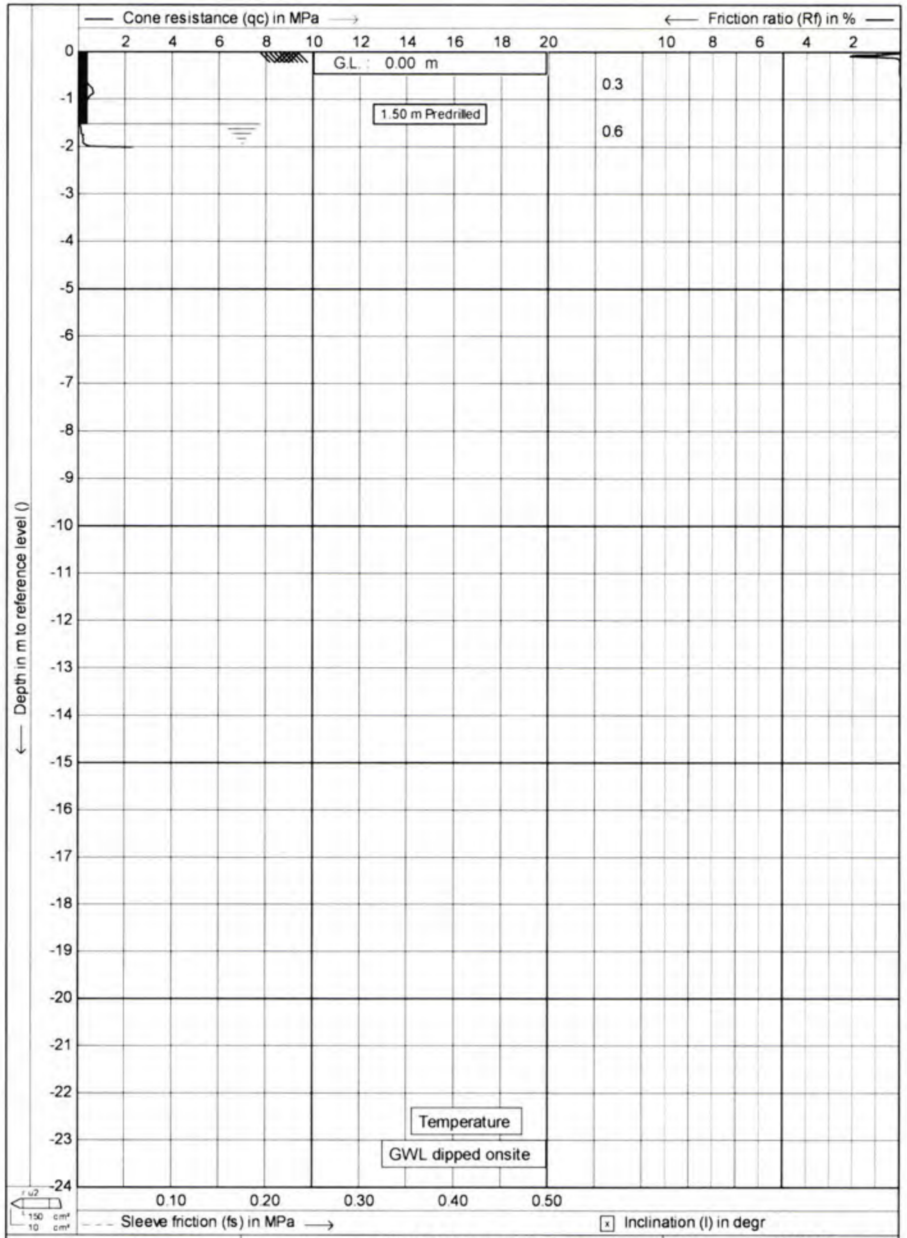
Position: **0, 0 RD**

Date : **31/07/2018**

Cone no. : **C10CFIIPT.C18265**

Project no. : **05TT08**

CPT no. : **17 a** **2/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**

Location: **Rotorua Lakefront - Rotorua**

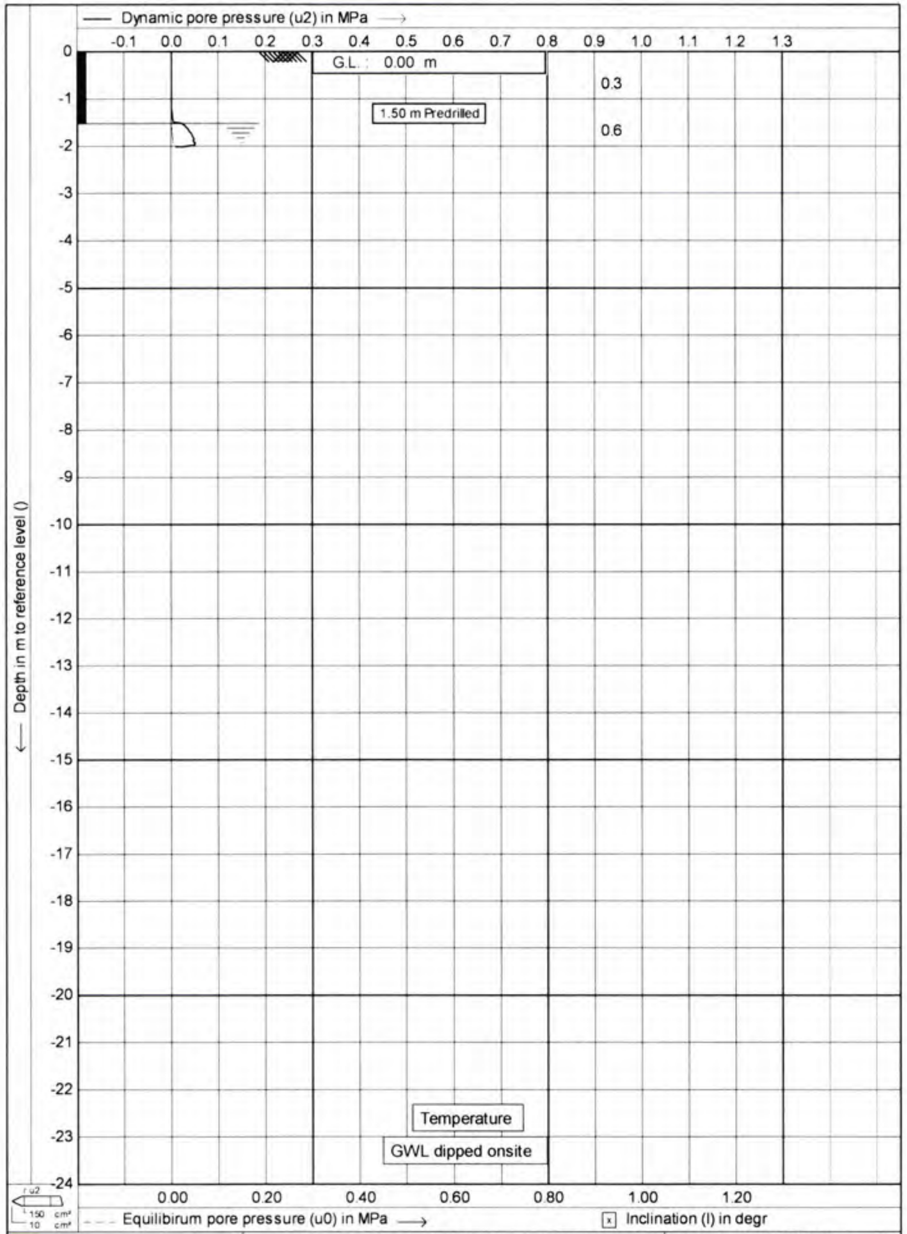
Position: **0, 0 RD**

Date : **31/07/2018**

Cone no. : **C10CFIIPT.C18265**

Project no. : **05TT08**

CPT no. : **18 a** **1/15**

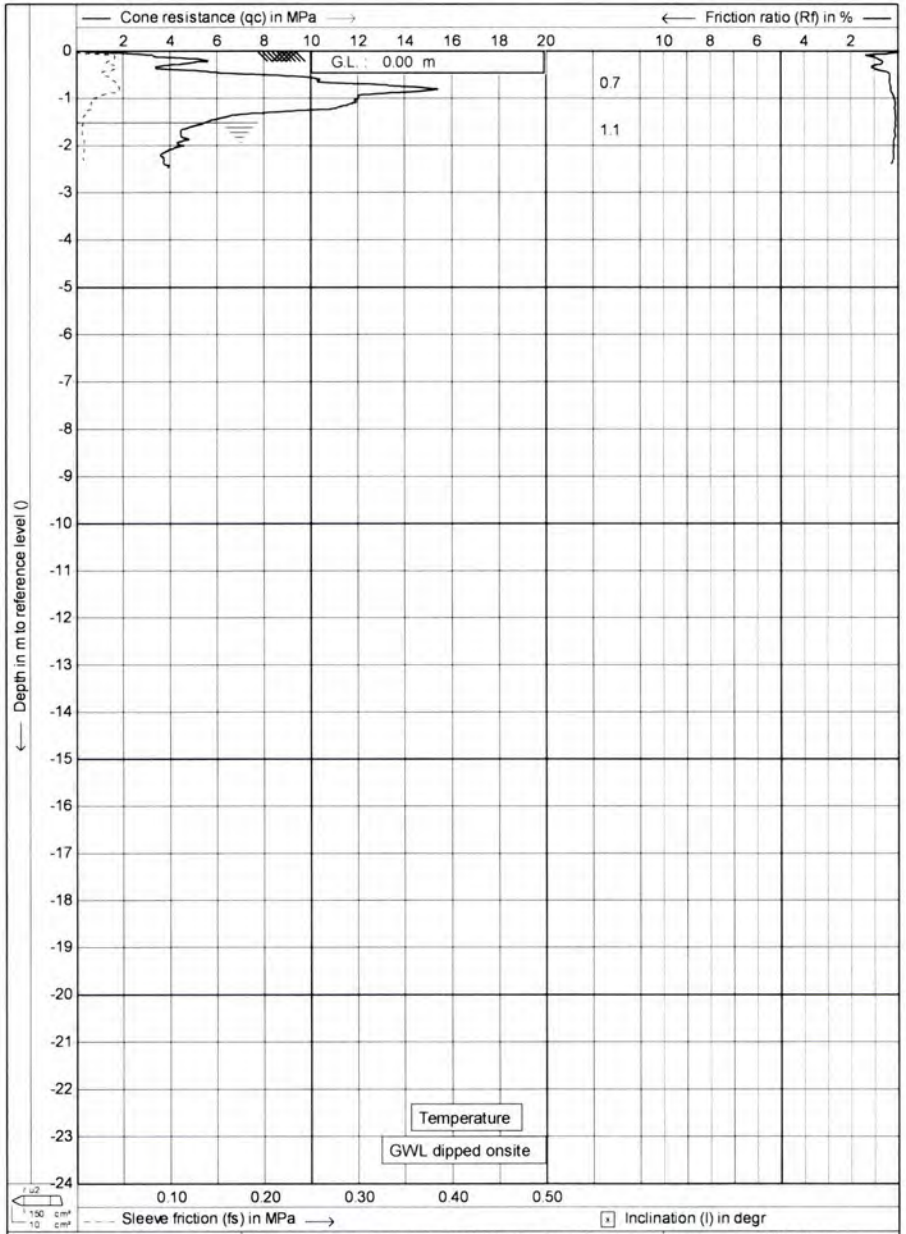


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIIPT.C18265**
Project no. : **05TT08**
CPT no. : **18 a** **2/15**

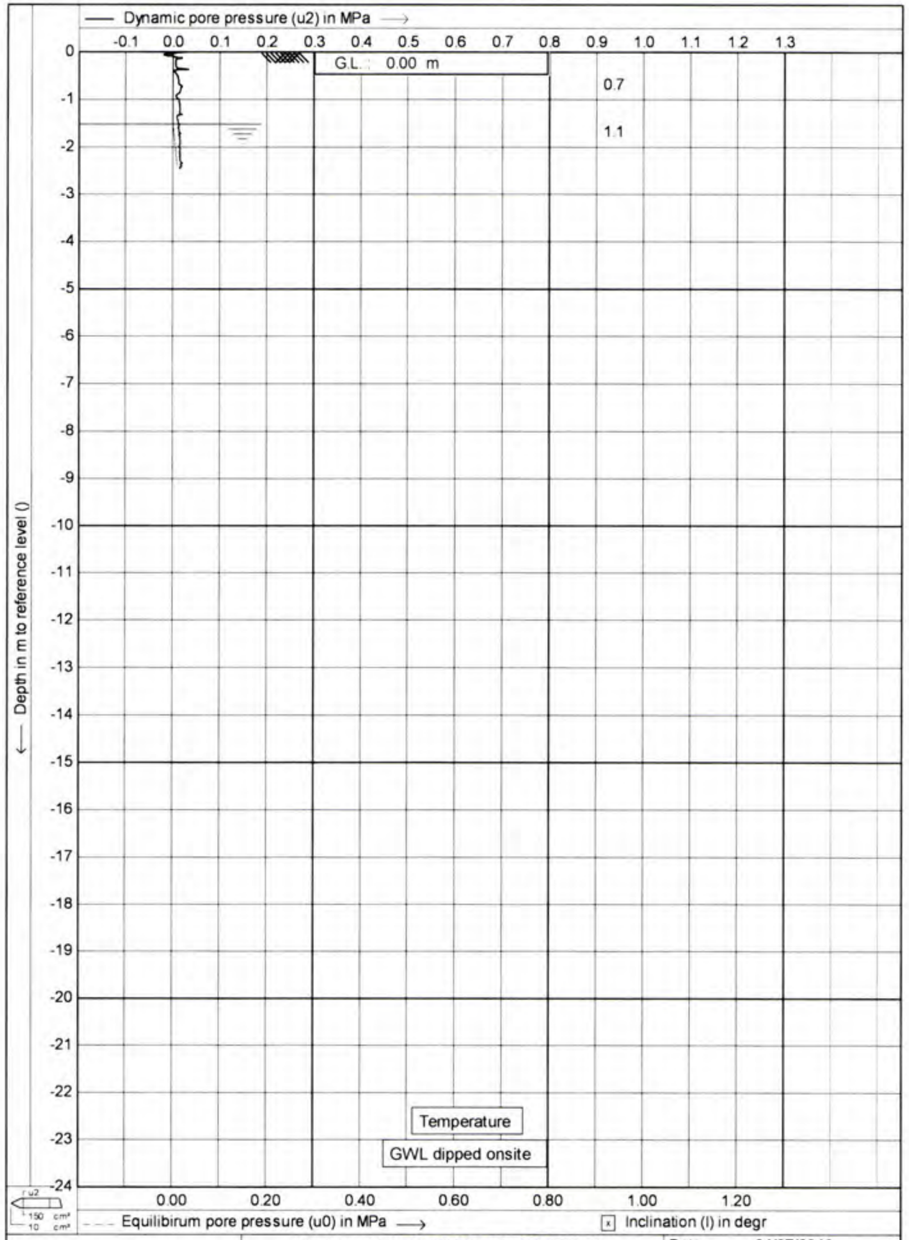


PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIIPT.C18265**
Project no. : **05TT08**
CPT no. : **19** **1/15**

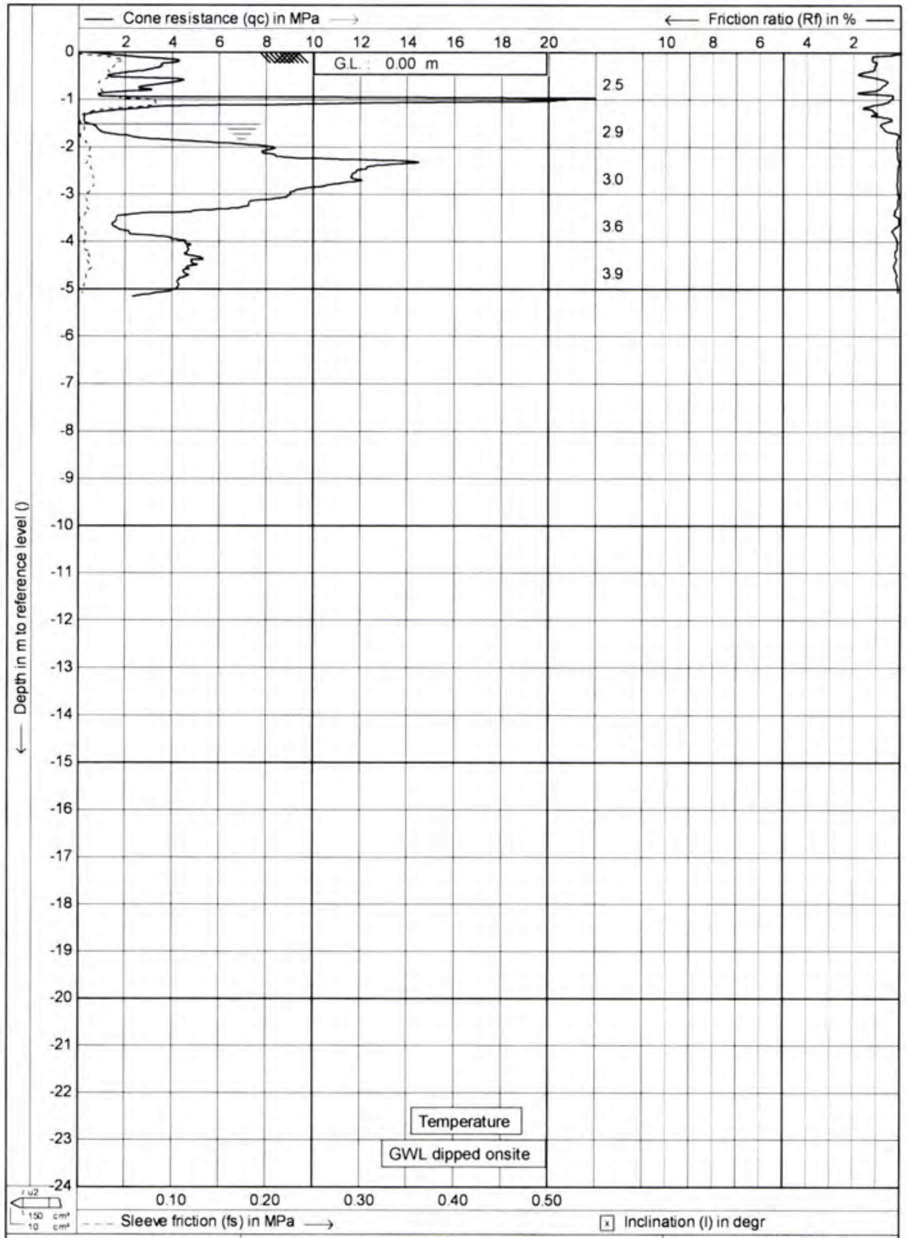


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIIPT.C18265**
Project no. : **05TT08**
CPT no. : **19** **2/15**

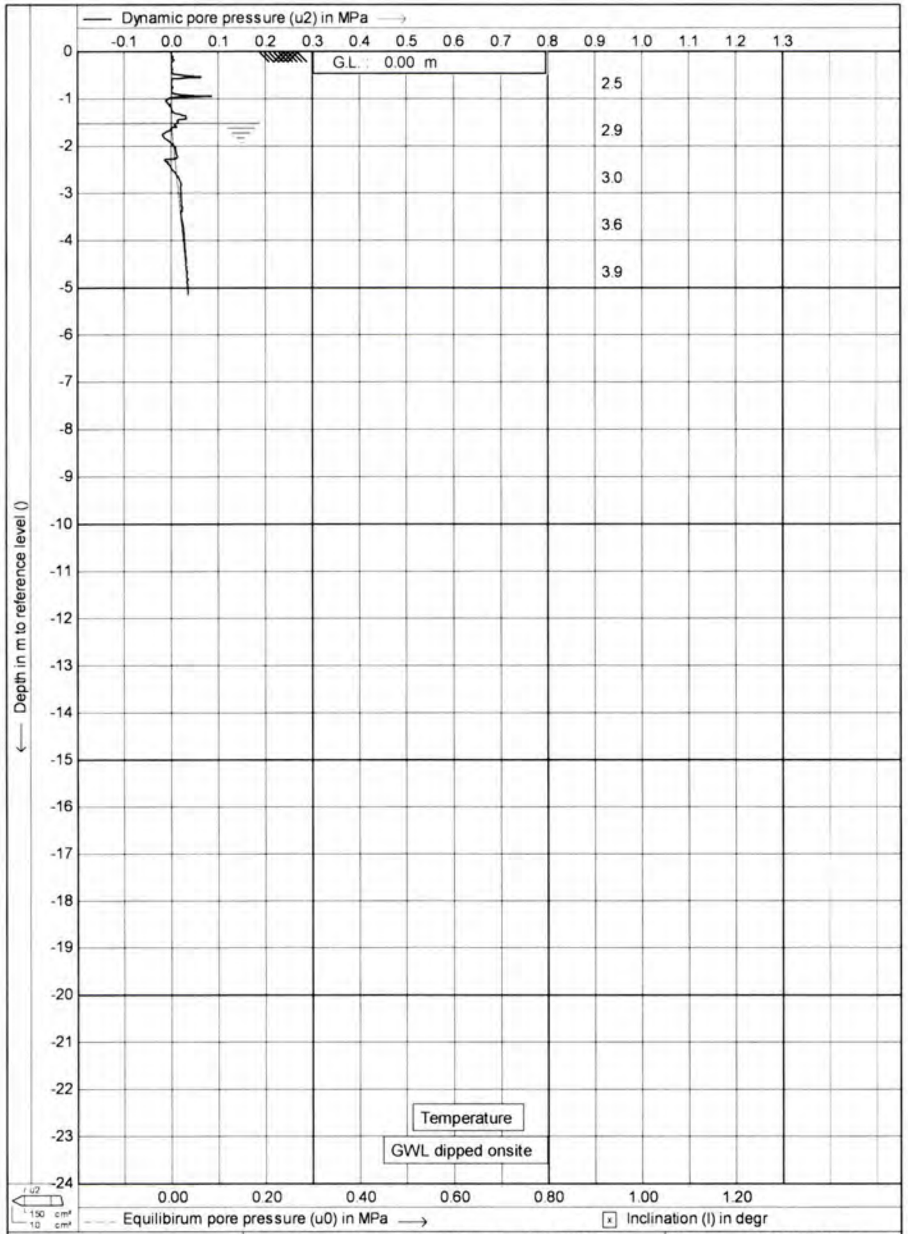


Test according A.S.T.M Standard D 5778-12

PERRY GEOTECH

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIIPT.C18265**
Project no. : **05TT08**
CPT no. : **20** **1/15**



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **SIte Investigations**
Location: **Rotorua Lakefront - Rotorua**
Position: **0, 0 RD**

Date : **31/07/2018**
Cone no. : **C10CFIPT.C18265**
Project no. : **05TT08**
CPT no. : **20** **2/15**

Appendix C: Laboratory test certificates

Hill Laboratories
TRIED, TESTED **AND TRUSTED**

R J Hill Laboratories Limited
28 Duke Street Frankton 3204
Private Bag 3205
Hamilton 3240 New Zealand

T **0508 HILL LAB** (44 555 22)
T +64 7 858 2000
E mail@hill-labs.co.nz
W www.hill-laboratories.com

Certificate of Analysis

Client:	Tonkin & Taylor	**Lab No:**	2027213 SPv1
Contact:	Adam Nally C/- Tonkin & Taylor PO Box 317 Tauranga 3140	**Date Received:**	07-Aug-2018
		Date Reported:	13-Aug-2018
		Quote No:	80842
		Order No:	001
		Client Reference:	1007467.1000
		Submitted By:	Adam Nally

Sample Type: Aqueous

	Sample Name:	BH01 30-Jul-2018 3:10 pm	BH02 31-Jul-2018 2:25 pm	BH03 01-Aug-2018 3:00 pm	BH04 02-Aug-2018 8:30 am	
	Lab Number:	2027213.1	2027213.2	2027213.3	2027213.4	
pH	pH Units	6.5	6.7	6.4	6.4	-
Sulphate	g/m³	30	19.9	35	14.3	-

Summary of Methods

The following table(s) gives a brief description of the methods used to conduct the analyses for this job. The detection limits given below are those attainable in a relatively clean matrix. Detection limits may be higher for individual samples should insufficient sample be available, or if the matrix requires that dilutions be performed during analysis.

Sample Type: Aqueous

Test	**Method Description**	**Default Detection Limit**	**Sample No**
Filtration, Unpreserved	Sample filtration through 0.45µm membrane filter.	-	1-4
pH	pH meter. APHA 4500-H^+ B 22nd ed. 2012. Note: It is not possible to achieve the APHA Maximum Storage Recommendation for this test (15 min) when samples are analysed upon receipt at the laboratory, and not in the field. Samples and Standards are analysed at an equivalent laboratory temperature (typically 18 to 22 °C). Temperature compensation is used.	0.1 pH Units	1-4
Sulphate	Filtered sample. Ion Chromatography. APHA 4110 B (modified) 22nd ed. 2012.	0.5 g/m³	1-4

These samples were collected by yourselves (or your agent) and analysed as received at the laboratory.

Samples are held at the laboratory after reporting for a length of time depending on the preservation used and the stability of the analytes being tested. Once the storage period is completed the samples are discarded unless otherwise advised by the client.

This certificate of analysis must not be reproduced, except in full, without the written consent of the signatory.

Ara Heron BSc (Tech)
Client Services Manager - Environmental

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www.tonkintaylor.co.nz