Southern Link Inland Port

Geotechnical Desktop Study and Appraisal

Prepared for:
Southern Link Property Limited

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Prepared by:
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Executive Summary

This Geotechnical Desktop Study and Assessment has been prepared to support the resource consent application for the proposed Southern Link Inland Port (SLIP) at 270–292 Dukes Road North, Mosgiel. The assessment compiles existing geological, geomorphological, hydrological, and geotechnical information to identify preliminary ground-related risks and considerations relevant to the development.

The site is underlain by Late Pleistocene and Holocene alluvial deposits, comprising fine-grained clays and silts overlying sand and gravel mixtures, consistent with the regional Taieri Plain stratigraphy. Test pit information supplied by Southern Link Property Limited (SLPL) indicates approximately 0.3 m of topsoil overlying 0.8 to 1.6 m of fine-grained soils, with gravel and sand encountered at depths of 2.4 to 2.6 m below ground level (bgl). Available laboratory testing shows highly variable plasticity and California Bearing Ratio (CBR) characteristics across the Site, including low to high plasticity clays, high plasticity silts, and non-plastic silts, with soaked and unsoaked CBR values ranging from 1% to 25%. These results indicate that moisture-sensitive, low-strength, and potentially compressible soils may remain beneath proposed subgrade levels.

Historical aerial imagery and topographic data reveal areas of potential slope instability along the northern bank of Silver Stream, which forms the southeastern boundary of the Site. Evidence of subsidence or instability is first visible in historical aerial imagery from the late 1960s to the 1980s, with no clear indication of recent failures, though ongoing erosion or shallow instability cannot be ruled out. A site visit by other parties indicated that railway iron had been installed at the base of the stream bank along much of the Site boundary potentially as a form of slope protection. However, the timing, intent, and effectiveness of this installation are unknown. The steep and incised stream margins suggest that instability could extend beyond the bank crest into parts of the site.

Groundwater monitoring conducted in late 2025 indicates groundwater depths of approximately 7.5–8 m bgl. For preliminary geotechnical analysis, a maximum groundwater level of 5 m bgl has been adopted to allow for fluctuations. Given these groundwater levels and the nature of the fine-grained soils observed in the test pits, the upper soils are unlikely to be liquefiable. However, deeper sand/silt layers may still be susceptible. A preliminary liquefaction analysis based on representative NZGD CPTs (located up to 2.7 km from the Site) suggests a low liquefaction risk for the assumed conditions, with estimated settlements of less than 4 cm across a range of peak ground accelerations (PGAs) covering the expected Site seismic hazard. Some lateral spreading risk remains adjacent to Silver Stream due to the significant free face and potential presence of deeper liquefiable or strain-softening materials.

Earthworks for the project are expected to exceed 250,000 m^3, including excavations to subgrade depths of approximately 0.6–0.7 m. Suitability of excavated material for reuse, potential off-site disposal, and the interaction between earthworks and adjacent infrastructure will require further assessment during design. Where fine-grained soils remain beneath subgrades, additional excavation or ground improvement may be required to manage settlement and bearing capacity considerations.

Because the current geotechnical dataset is limited, the selection and design of mitigation measures for hazards such as slope instability, settlement, liquefaction, and lateral spreading will be developed following the completion of upcoming site-specific investigations. These investigations, expected to include a geological survey, boreholes, CPTs, and laboratory testing, will enable detailed slope stability modelling, refined bearing capacity and settlement analyses, and confirmation of seismic soil parameters. Mitigation measures may involve slope stabilisation works, setback requirements from stream banks, excavation of weak soils, ground improvement, or other engineering controls. However, the appropriate approach will depend on the geotechnical parameters and subsurface conditions confirmed during the design-phase investigations.

Key data limitations constrain the current assessment, including the absence of site-specific CPTs, boreholes, or geophysical data, as well as a reliance on aerial imagery and topography data for assessment of slope instability features. However, geotechnical investigations and geological surveys recommended to be required through conditions of consent are expected to significantly improve the Site characterisation via targeted data collection, enabling refined geotechnical analyses to address any identified risks.

Based on the collected information, preliminary findings indicate:

- Significant shallow soil variability, including moisture-sensitive and low-strength materials that may influence bearing capacity and settlement behaviour.
- Evidence of historical slope instability at isolated locations along Silver Stream with the potential for future instability at any point along the slope boundary that could impact proposed infrastructure near the stream.
- Low liquefaction risk for the adopted groundwater regime, with deeper layers requiring further verification.
- Overall suitability of the site for the proposed development with appropriate mitigation to be confirmed through design-phase investigations.

Overall, the available information is sufficient to support the resource consent application. Additional geotechnical data will also be obtained from ground investigations anticipated to be undertaken at the start of detailed design. These investigations will be essential to confirm subsurface conditions, refine the Site ground model, develop robust mitigation strategies, and support detailed engineering design.

Acronyms / Abbreviations

Acronym / Abbreviation	Full Name
AEP	Annual Exceedance Probability
ASCE	American Society of Civil Engineers
BH	Borehole
bgl	below ground level
CBR	California Bearing Ratio
CH	High-Plasticity Clay
CL	Low-Plasticity Clay
CPT	Cone Penetration Test
DCP	Dynamic Cone Penetrometer
DEM	Digital Elevation Model
GNS	GNS Science
HA	Hand Auger
IL	Importance Level
LiDAR	Light Detection and Ranging
LINZ	Land Information New Zealand
LL	Liquid Limit
LPI	Liquefaction Potential Index
MBIE	Ministry of Business, Innovation and Employment
MH	High-Plasticity Silt
NP	Non-Plastic
NZAFD	New Zealand Active Fault Database
NZGD	New Zealand Geotechnical Database
NZS	New Zealand Standard
ORC	Otago Regional Council
PGA	Peak Ground Acceleration
PI	Plasticity Index
SLIP	Southern Link Inland Port

1 General

1.1 Scope

This geotechnical desktop study and appraisal has been undertaken to gather, collate, and assess existing knowledge of the expected ground conditions relevant to the proposed Southern Link Inland Port (SLIP) site located at 270 – 292 Dukes Road North, Mosgiel (the Site). The intent of this report is to document the desktop findings, provide a preliminary assessment of the likely geotechnical-related risks at the Site and provide geotechnical considerations for resource consenting. This desktop study does not include a site walkover or site-specific ground investigation.

1.2 Contract Details

Stantec has been commissioned by Southern Link Property Limited (SLPL) to support a substantive application for the SLIP under the Fast-track Approvals Act 2024. The contract works have been undertaken to advance Stantec’s design approach for all stages of the SLIP development, with the intent to facilitate the granting of all necessary approvals.

1.3 Code of Conduct

The Independent Reviewer of this report is Ioannis Antonopoulos. Ioannis is a Chartered Geotechnical Engineer with over 30 years of experience in large infrastructure and development projects. His expertise includes working within interdisciplinary teams in both design and construction, with a particular focus on geotechnical earthquake engineering design, roading, ports, seawalls, water reservoirs, surface and deep foundations, cut-and-cover structures, tunnelling, slope stability, hydrogeology, and water resource management.

Ioannis confirms that he has read the Code of Conduct for expert witnesses contained in the Environment Court Practice Note 2023. This report has been prepared in compliance with that Code, as if it were expert evidence presented in proceedings before the Environment Court. Unless Ioannis states otherwise, this report is within his area of expertise, and he has not omitted to consider material facts known to him that might alter or detract from the opinions expressed in this report.

1.4 Project Overview

SLPL seeks to establish an Inland Port / Logistics Hub on an approximately 40-hectare site on the Taieri Plains near Mosgiel to enable freight transfer between trains and trucks. The SLIP Site is located at 270 – 292 Dukes Road North, as outlined in yellow and ancillary activities within the KiwiRail corridor marked blue in Figure 1. As the ancillary activities within the KiwiRail corridor do not impact on the content of this report, they are not discussed further in this report. This SLIP will deliver critical infrastructure to support regional economic growth, improve freight logistics, and enhance resilience to environmental risks.

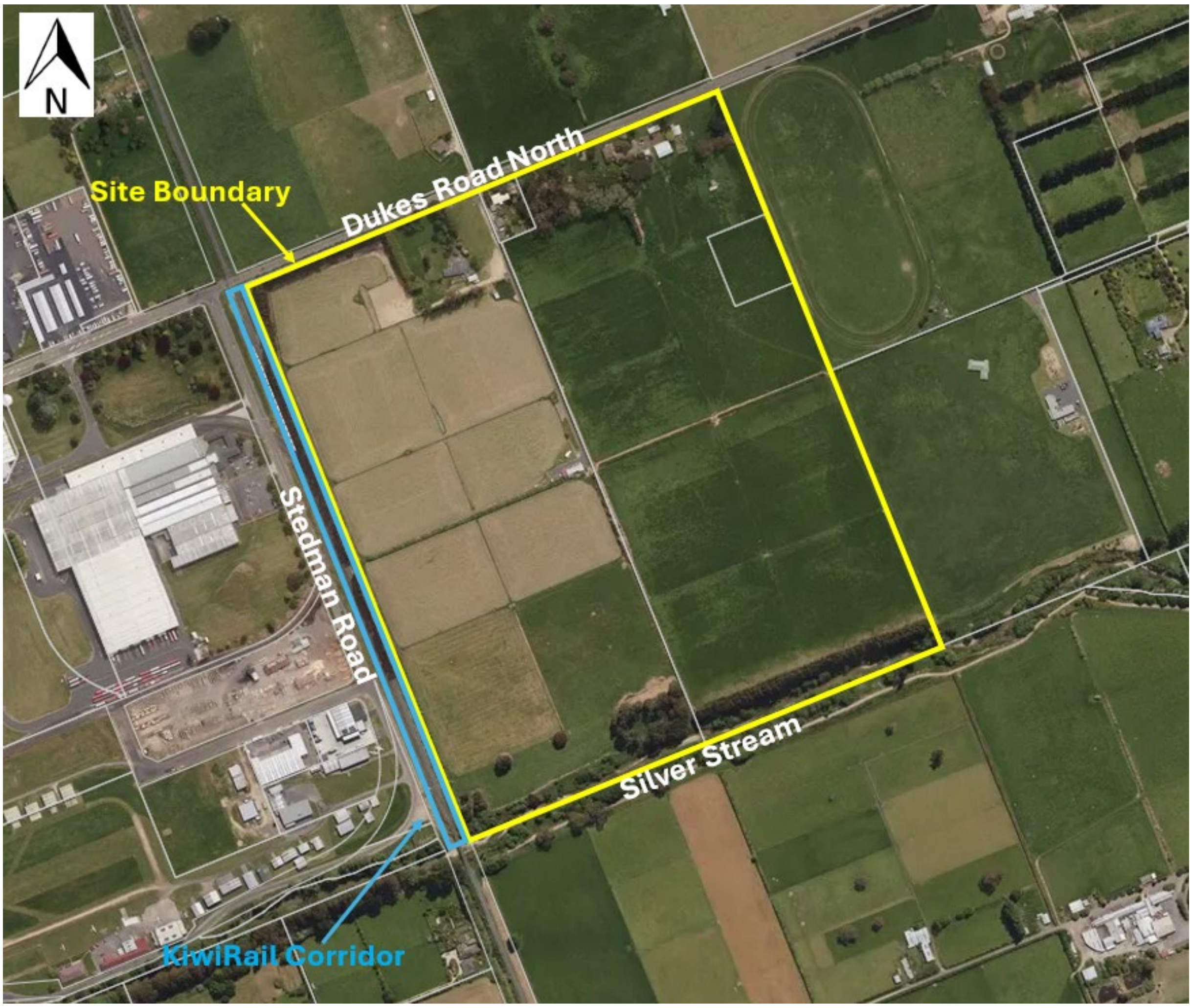

Figure 1. *Site overview*

The development will occur as described in the Project Description of SLPL's Substantive Fast-track Application, which should be referred to for a complete description of the Inland Port development. In summary, the Inland Port will include:

- A new rail siding off the Taieri Branch Line to enable loading, unloading and operation of a rail freight shuttle service to Port Chalmers and the wider rail network;
- Approximately 155,000 m^2 of high stud warehousing (chilled and ambient) and associated yard and canopy areas;
- Two road exchange areas for the loading and unloading of container trucks;
- A container depot facility enabling the inspection, cleaning, upgrading and repair of containers, including for food grade repacking;
- Approximately 9 ha of container terminal for storage and movement of empty and full containers, including refrigerated containers;
- Approximately 1000 m^2 of onsite offices ancillary to the Inland Port;
- Road widening and construction of a new intersection on Dukes Road North;
- 24/7 operation with flood and road lighting for nighttime operation;

- Ancillary activities to support the above including vehicle parking, truck waiting areas, onsite road network, three waters and power infrastructure, flood mitigation, landscaping, security measures, acoustic barriers and lighting; and
- Ongoing management and monitoring activities including ensuring establishment of landscaping, stream health monitoring, wildlife management and effects management.

Construction of the Inland Port is anticipated to be undertaken in three stages however the timing of the delivery of each stage, and discrete works within each stage, may change in response to demand for logistics capacity at the Inland Port. Each stage of works will involve site clearance, earthworks, construction of buildings, hardstanding and access, installation of infrastructure, landscaping and works and management activities necessary to manage environmental effects during construction including erosion and sediment controls and construction management activities:

- **Stage 1** is estimated to be completed 1 to 3 years following approval of the Project and will include clearance of the southern area of the site and construction of the ‘Stage 1’ container storage concrete pad, rail siding, container service area, warehouses, internal roading, parking and loading, road widening and construction of the new intersection on Dukes Road North, stormwater attenuation pond, Silver Stream stormwater outlets, servicing infrastructure, flood management measures, landscaping, acoustic barriers and eastern bund, and lighting.
- **Stage 2** is estimated to be completed 3 to 5 years following approval of the Project and will include clearance of the northern area of the site and construction of the ‘Stage 2’ container storage concrete pad, warehouses, ancillary offices, internal roading, parking and loading, emergency egress onto Dukes Road North, expansion of the stormwater attenuation pond, landscaping, extension of the servicing infrastructure and lighting.
- **Stage 3** is estimated to be completed 5 to 10 years following approval of the Project and will include clearance of the eastern area of the site, including the eastern acoustic bund, and construction of the ‘Stage 3’ warehouses, internal roading, parking and loading, landscaping, extension of the servicing infrastructure and lighting

2 Geomorphological and Geological Conditions

2.1 Regional Geomorphology

The Site is located on the Taieri Plain, situated within an extensive array of ranges and basins formed from the tectonic uplift, faulting and folding of schist basement rock. The basin containing the Taieri Plain was once a submerged marine inlet following post-glacial sea level rise, with subsequent infilling of Quaternary sediments deposited via alluvial and fluvial processes associated with the Taieri River and its tributaries. Poorly consolidated fine-grained sediments (“Waihola silt/sand”) carried in by the Taieri River dominate southwestern areas of the Taieri Plain, whereas in the northeastern reaches near the project site, Silver Stream has formed an alluvial plain extending over the Waihola silt/sand to the southwest (Barrell, Cox, Glassey, & Lyttle, 2014).

2.2 Site Geology

A review of the 1:250,000 GNS Science geology map (GNS, 2023) indicates that the site is underlain by two geological formations:

- OIS2 Late Pleistocene River Deposits
    - Comprising poorly consolidated, slightly weathered sandy quartz-, schist-, or volcaniclastic-derived gravel and sand.
- OIS1 Holocene Fan Deposits
    - Comprising poorly consolidated, often poorly sorted, fine to bouldery gravel with sand and mud.

Figure 2 shows the mapped geology for the area with the site boundary outlined in red. The Pleistocene River Deposits constitute the primary geological formation mapped as underlying the Site, with a small portion of the southeast corner of the Site comprising Holocene Fan Deposits.

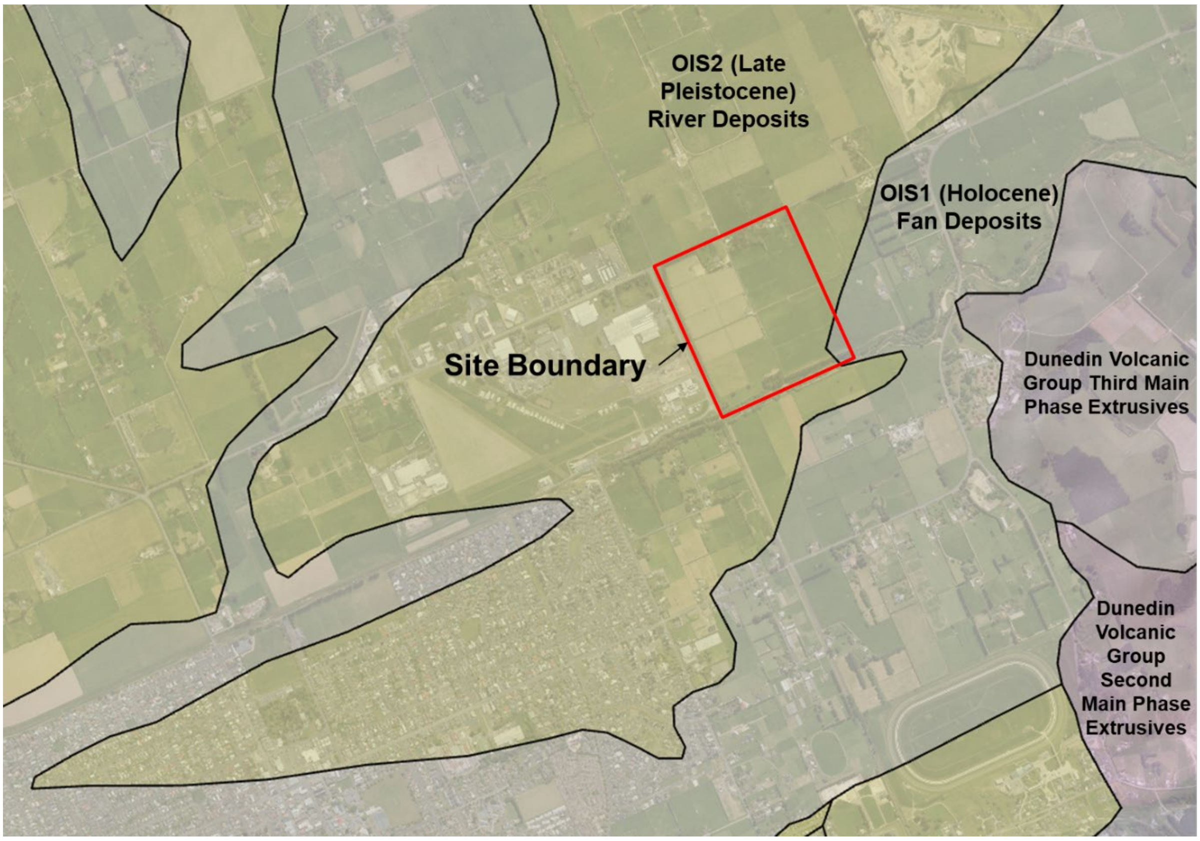

Figure 2. Mapped geology (GNS, 2023)

2.3 Site Topography

Figure 3 presents a hillshade map based on the New Zealand LiDAR 1m DEM (current as of 27 November 2025) obtained from Land Information New Zealand (LINZ). Site topography is generally flat, with an elevation of between approximately 29 to 25 m above mean sea level, sloping gently downward to the south and west across the Site.

Silver Stream forms the southeastern boundary of the Site (flowing to the southwest) and is incised by approximately 7-10 m relative to the adjacent site ground levels. Bank slopes are generally highest and steepest in upstream areas of the stream reach along the Site and decrease slightly in downstream areas. Based on LiDAR data and aerial imagery, areas of potential slope instability have been identified along the northern bank that forms the Site boundary, as shown in Figure 3. These areas are further discussed below in Section 3.1 and Section 7.5.

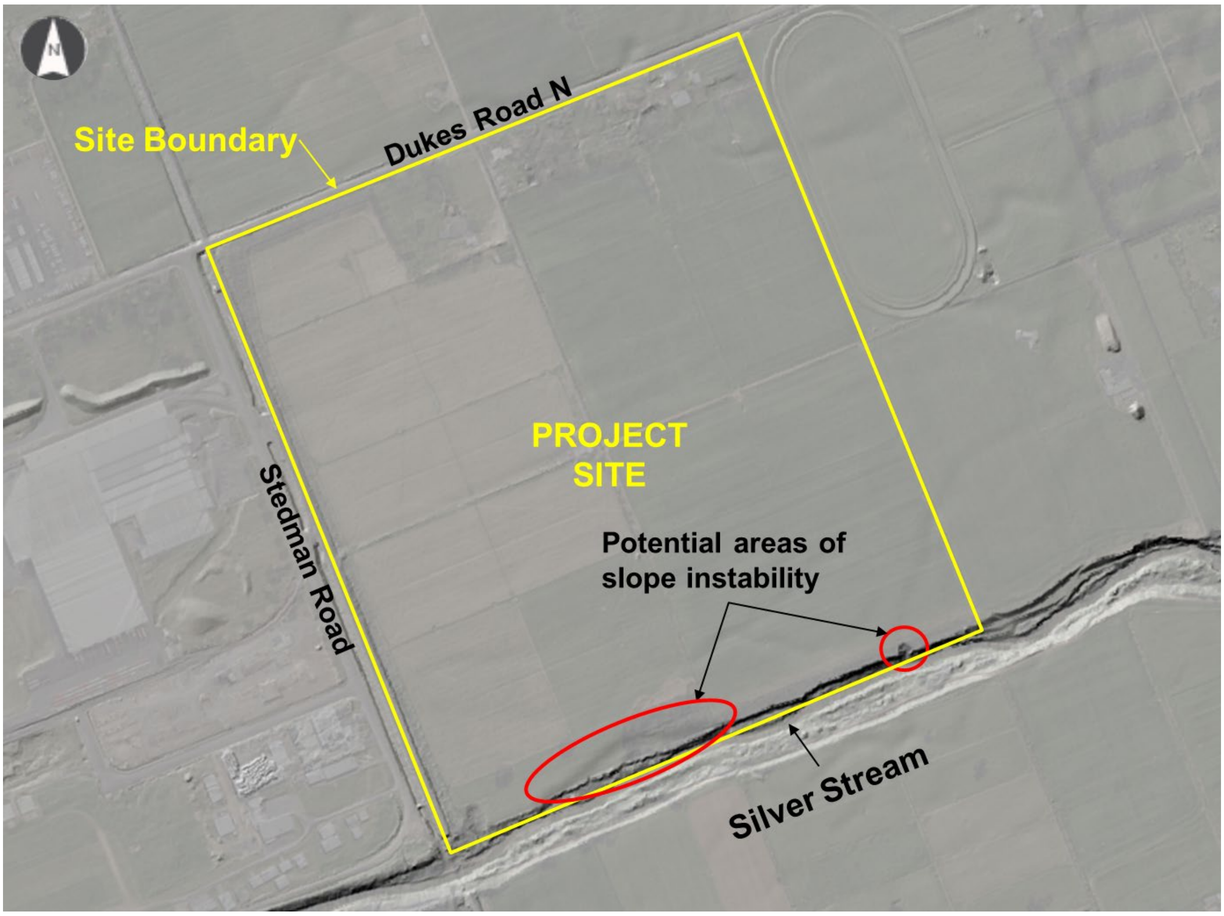

Figure 3. Hillshade map showing site topography

3 Land Use

3.1 Historical Aerial Imagery

A review of historical aerial imagery from Retrolens (Retrolens, 2023) and Google Earth indicates that land use at the Site has remained generally consistent since at least 1942. Photographs are included in Appendix A.

Photographs show that much of the area has been managed as farmland with a small strip of residential and/or farm-related development adjacent to Dukes Road North along the northern site boundary. A plot of development observed in the northwestern quadrant of the site area in 1942 appeared to have been removed by 1950, before reappearing in the same area by 1985 and generally persisting to the present day.

The historical aerial imagery indicates that the alignment of Silver Stream has undergone minor adjustments since 1942, with the channel migrating slightly within its established banks and observable shifts in depositional features (see sequence of aerial photographs in Appendix A). Vegetation cover has also varied considerably over the years, at times obscuring the aerial view of the channel banks.

A relatively unobstructed view from 1970 shows less vegetation and considerable progression of erosional and/or slope instability features in areas where the flow channel passed along the toe of the northern stream bank at that time. In particular, the western (downstream) potential instability area shown in Figure 3 appears to have experienced notable subsidence sometime between 1963 and 1970. The subsided area is still evident in the topography from the recent LiDAR data discussed above in Section 2.3. The eastern (upstream) potential instability area is also labelled in Figure 3 is first observed in aerial imagery from 1985, with little evidence of continued progression between then and the present day.

4 Historical Climatic and Seismic Events

4.1 Extreme Flood Events

As previously described, ground conditions within the Taieri Plain have historically been influenced by flooding of local waterways. The Site is situated in North Taieri, an area that has been characterised as susceptible to high flows and surface runoff from nearby upstream catchments that discharge onto the northern part of the Taieri Plain, as well as exposed to flood hazard from Mill Creek and Silver Stream (Otago Regional Council, 2015).

Otago Regional Council (ORC) developed flood-hazard maps which divide the geographical areas based on flood hazard characteristics. The North Taieri area is further subdivided into areas that are critical for the conveyance or storage of floodwater (‘North Taieri overland flow paths’ and ‘North Taieri floodways’), and the residual floodplain (‘North Taieri floodplain’), each of which have been mapped in proximity to the Site (see Figure 4).

As shown in Figure 4, the southwestern boundary of the Site is adjacent to an overland flow path along the Taieri Gorge Railway embankment, with much of the southwestern portion of the Site falling within the North Taieri flood ways area. The rest of the site falls within the North Taieri floodplain area, where runoff is slower and of shorter duration compared to the flood ways.

According to ORC (2015), the North Taieri floodways areas generally experienced flooding depths up to 1.5 m during an extreme flood event in April 2006, while flooding depths in the North Taieri floodplain areas were generally limited to 1 m or less, with the depth and extent of flooding influenced by local features (e.g., embankments, fences, buildings). Hydraulic models produced as part of Stantec’s stormwater assessment further suggest flows conveyed across the Site are typically less than 1 m deep (Stantec, 2026).

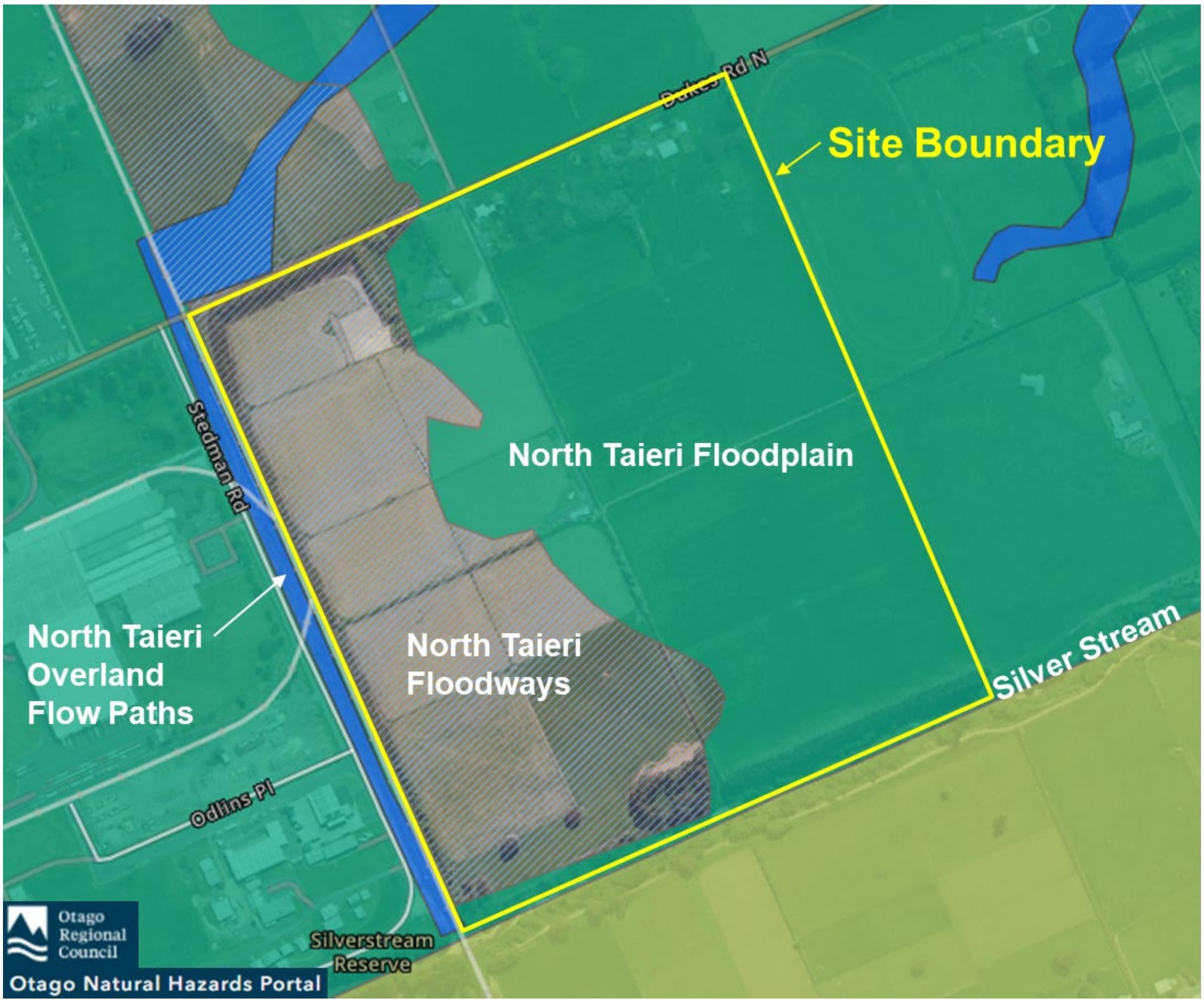

Figure 4. Flood hazard characteristics of the Site area, from Otago Regional Council's Natural Hazards Portal and based on ORC (2015)

4.2 Seismicity

Figure 5 shows nearby active faults mapped in the GNS New Zealand Active Fault Database (NZAFD). The nearest faults include the North Taieri Fault and Titri Fault, both reaching within approximately 5 km of the Site. The Akatore Fault is the best-known and has generated at least two large earthquakes with surface rupturing (likely magnitude 7) within the past 3,800 years (Barrell, Cox, Glassey, & Lyttle, 2014). According to QuakeCoRE, a research institute funded by the New Zealand Tertiary Education Commission, the seismic hazard corresponding to the Akatore Fault is expected to comprise a M_w 7.14 earthquake with a probability of occurrence of 1.43% in 50 years, as shown in Figure 6.

Possible risks associated with a large seismic event impacting the Site, to be further assessed and characterised during detailed design, may include:

- Soil shear strength reduction (e.g., liquefaction or cyclic softening) affecting the bearing capacity of structure foundations and pavements.
- Liquefaction-induced settlements impacting foundations and pavements.
- Slope instability and lateral spreading along Silver Stream.

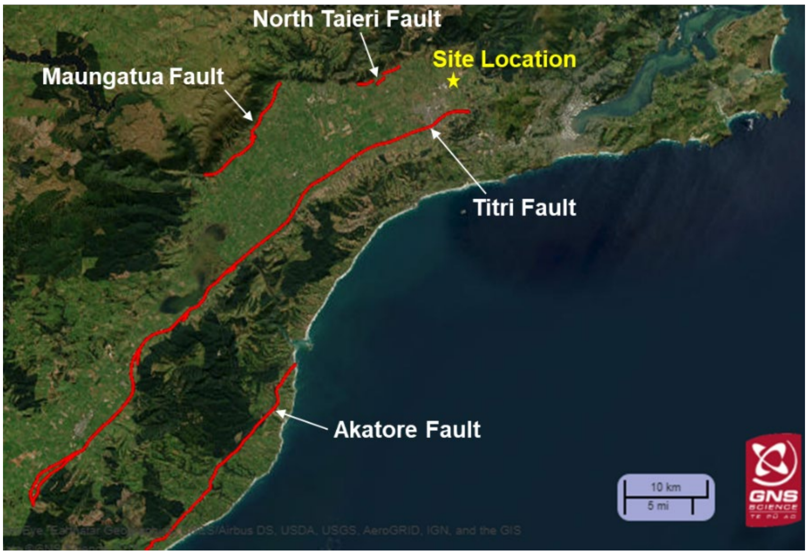

Figure 5. Nearby active faults according to GNS NZAFD

Fault Name: Akatore [Akatore]
Tectonic Type: Crustal
Magnitude: 7.14Mw
Avg. Rake: 90°
Avg. Dip: 45°
Rupture Top Depth: 0km
Probability in 50 years: 1.43%
Simulation Video

Figure 6. Characterisation of the Akatore Fault (QuakeCoRE)

5 Previous Ground Investigations

5.1 Site-Specific Investigations

SLPL provided Stantec with existing information from a test pit (TP) investigation program that was undertaken at the Site in August 2024. The test pit logs and associated CBR test results were completed by third parties, and Stantec was not involved in the planning, supervision, or logging of this work. The test pits were logged by an experienced technician (Grant Driver, NZCE [Mechanical]), and the CBR field testing was carried out by SouthRoads personnel (Jack Hadlans, NZ Dip Civil). While this information provides useful insight into ground conditions, Stantec is unable to independently verify the methods, observations, or interpretations made during these investigations. Accordingly, the data is considered indicative only and subject to the normal limitations associated with third-party investigation records.

Seven (7) TPs were excavated across the Site at the approximate locations shown in Figure 7. The TP excavation depths ranged from 2.4 to 2.6 m below ground level (bgl).

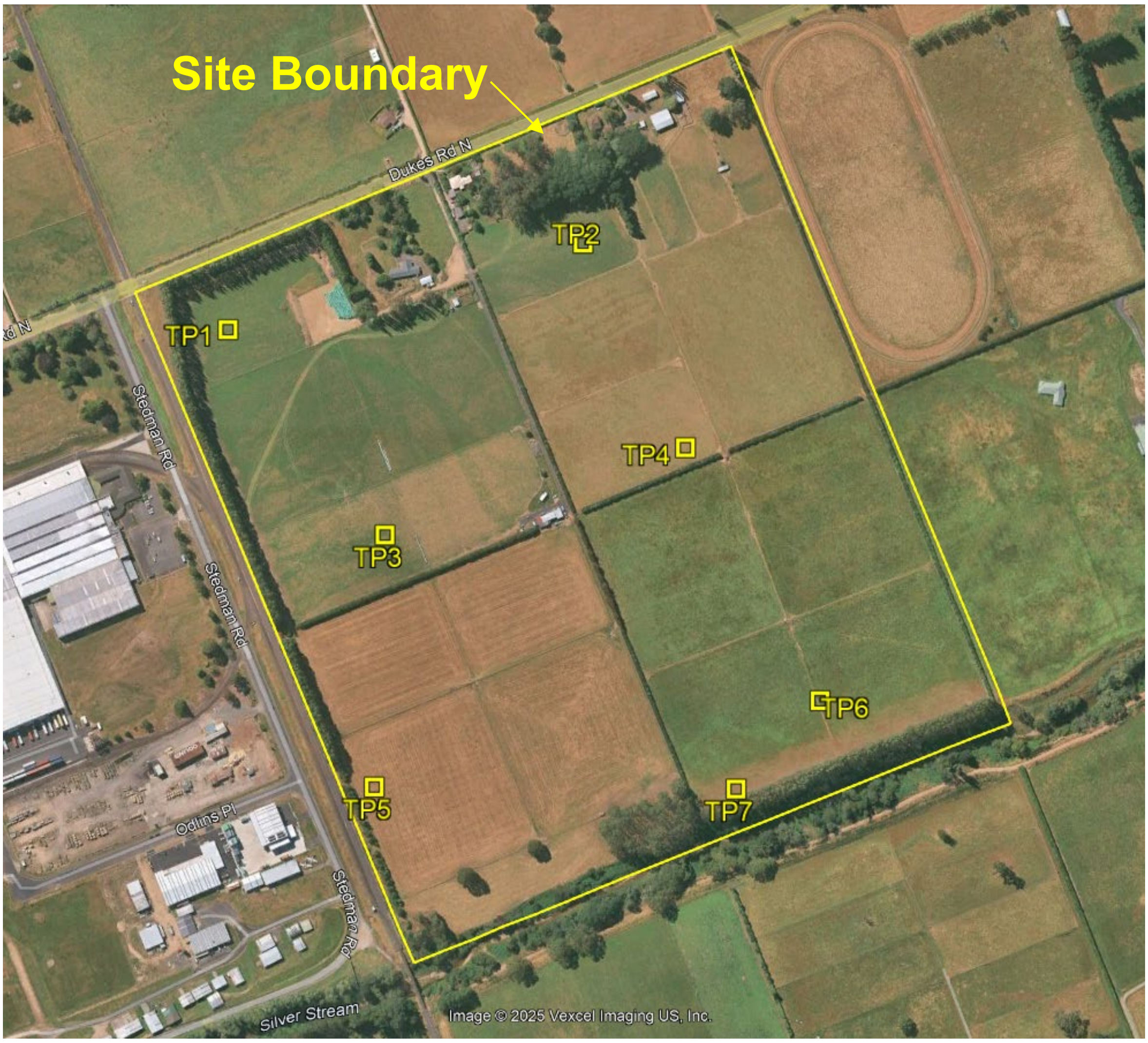

Figure 7. Approximate locations of Site test pit investigations

Ground conditions were generally consistent across the site, with the following materials encountered in the TPs (sequentially from top to bottom):

- Approximately 0.3 m of topsoil.
- Fine-grained soils (logged as firm clay with occasional sand/silt mixtures) to depths ranging from approximately 0.8 to 1.1 m bgl at TP1 through TP5, and from 1.5 to 1.6 m bgl at TP6 and TP7 near Silver Stream.
- Gravel and sand to the maximum investigation depths of 2.4 to 2.6 m bgl.

Additional spot Dynamic Cone Penetrometer (DCP) tests were undertaken near the corner of Dukes Road North and Stedman Road and used to estimate California Bearing Ratios (CBR) for the encountered soils based on correlations with the DCP blow counts.

As further discussed in Section 7 and Section 8, additional site-specific investigations will be undertaken at the start of detailed design to supplement the earlier test pits and confirm ground conditions at the Site.

5.1.1 Laboratory Testing

SLPL provided Stantec with a spreadsheet summary of laboratory testing results, including Atterberg limit and California Bearing Ratio (CBR) testing. Stantec has not been provided with laboratory reports. Based on the provided test summary, bulk samples were collected from each Site TP; however, Stantec is not aware of the specific sample depths as this information was not included in the summary.

5.1.1.1 Atterberg Limits

Atterberg limit testing was conducted on samples collected from the upper fine-grained soil layer at each TP (generally between 0 to 1.6 m bgl). We are advised by SLPL that the plasticity index (PI) tests were performed in accordance with NZS 4402:1986, Tests 2.3 and 2.4. Results are summarised in Table 1. Stantec has plotted the test results on the Casagrande Plasticity Chart in Figure 8.

Table 1. Atterberg limit testing results for Site test pit samples

Sample ID	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Plasticity Chart Classification
TP1	42	22	20	CL
TP2	55	28	27	CH
TP3	-	-	-	NP*
TP4	57	28	29	CH
TP5	66	34	32	MH
TP6	-	-	-	NP*
TP7	-	-	-	NP*

*NP = Non-Plastic

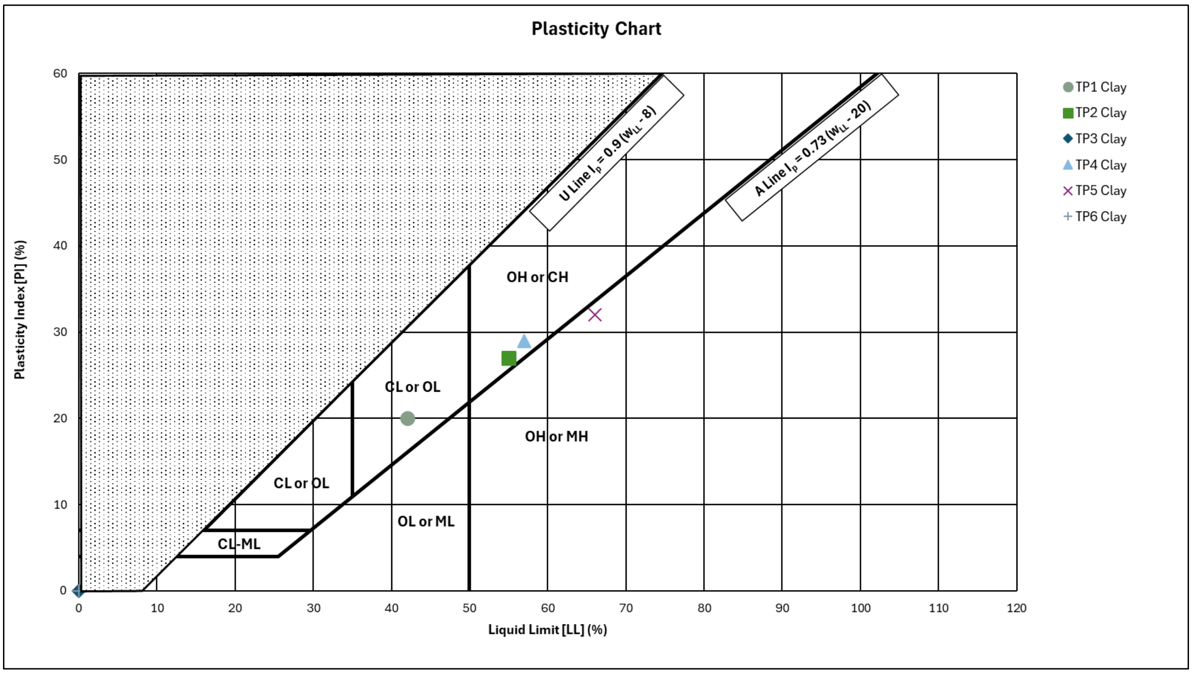

Figure 8. Plasticity chart for Site test pit samples

Based on these test results, the fine-grained soils appear to consist of low- and high-plasticity clays (CL and CH, respectively) and high-plasticity silt (MH), with likely non-plastic silts encountered in TP3, TP6 and TP7, though no additional test data (such as particle-size analysis) is available to confirm the differentiation between fine-grained and coarse-grained materials (e.g., silt versus silty sand) for these three samples.

Based on the test results, the upper fine-grained soil layer is highly variable, and a range of soil behaviours can be expected across the Site.

Higher values of LL and PI (such as those for the CH and MH soils) generally correspond to greater plasticity, compressibility, and moisture sensitivity. CL soils typically exhibit moderate cohesion and plasticity, remaining relatively firm when dry and showing limited volume change with water content variation, whereas CH soils are highly cohesive and exhibit pronounced plasticity, potentially becoming very soft when wet and undergoing shrink-swell cycles, which can lead to substantial changes in consistency and strength. MH soils display lower cohesion than clays but are highly sensitive to moisture, often losing structure and strength rapidly when saturated, and are prone to erosion. In contrast, non-plastic silts lack measurable plasticity and exhibit little cohesion, behaving more like granular soils. These soils can become unstable when saturated and may be highly susceptible to erosion.

Such risks can be managed by undertaking detailed site ground investigations, followed by geotechnical testing and interpretation of soil properties and parameters at the detailed design stage. These interpretations, as well as design calculations, will enable a refined assessment of the expected risks and possible mitigation strategies. Since these potentially challenging fine-grained soils are expected to be present at relatively shallow depths (generally less than 1.6 m bgl), excavation and replacement with suitable fill material may be a feasible option for critical Site facilities where the risk of unstable soils is not within the tolerable limits of a structure. Such limits will be confirmed during detailed design, and design calculations will be undertaken to quantify the expected risk.

5.1.1.2 California Bearing Ratio

Additional laboratory testing included CBR tests conducted on samples of the fine-grained material from each TP (generally between 0 and 1.6 m bgl). The provided test summary indicated the 4-day soaked CBR tests were conducted in accordance with NZS 4407:2015 3.1.5; however, no test standard was identified for the unsoaked CBR tests. Table 2 summarises the CBR results.

Table 2. CBR testing results for Site test pit samples

Sample ID	4-Day Soaked CBR (%)[1]	Unsoaked CBR (%)
TP1	3	-
TP2	4	5
TP3	1	1
TP4	-	7
TP5	4.5	6
TP6	7	10
TP7	2.5	25

[1] Based on NZS 4407:2015 3.1.5

CBR represents a strength index commonly used as an indication of material quality in terms of stiffness and resistance to permanent deformation. While not a fundamental material property, CBR has been found to be reasonably well correlated with more fundamental properties (e.g., resilient modulus) used for pavement design.

The CBR results show substantial variability across the tested samples based on differences in soil type and moisture condition:

- **Clayey soils (CL and CH)**
  - TP1 (CL) and TP2/TP4 (CH) samples exhibited soaked CBR values of 3–4%, with TP4 showing an unsoaked value of 7%, consistent with the low bearing capacity typical of plastic clays, especially under saturated conditions. Such soils are highly moisture-sensitive and require stabilisation or thicker pavement layers for adequate performance.
- **Highly plastic silt (MH)**
  - Soaked and unsoaked CBR values of 4.5% and 6%, respectively, were measured for the TP5 samples. This aligns with expectations for MH soils, which generally behave similarly to clays in terms of strength reduction when wet.
- **Non-plastic silty or sandy silt soils**
  - TP3, TP6, and TP7 sample results show contrasting behaviour. Extremely low CBR values (1% soaked and unsoaked) were measured for the TP3 sample, suggesting poor compaction or very fine silt with high sensitivity to saturation. TP6 material performs better (7% soaked, 10% unsoaked), indicating moderate bearing capacity. TP7 material is notable for its large difference between soaked (2.5%) and unsoaked (25%) CBR, suggesting severe moisture susceptibility, potentially due to loose structure or capillary effects that may occur in a non-plastic silt.

Overall, the fine-grained soil layer observed across the Site is likely to present bearing capacity issues for proposed pavement infrastructure if left in place, based on the laboratory test results presented above. Current plans for the project indicate this material will be partially excavated to achieve the proposed pavement subgrade levels (approximately 0.7 m bgl). Earthworks and bearing capacity considerations are further discussed below in Section 7.

5.2 New Zealand Geotechnical Database

Stantec searched the New Zealand Geotechnical Database (NZGD) for past investigations with data relevant to the Site assessment. The nearest investigations, including several Boreholes (BH), Cone Penetration Tests (CPT) and Hand Augers (HA), were located approximately 2.2 to 2.7 km away in the town of Mosgiel, as shown in Figure 9. BH logs, HA logs and CPT reports from representative NZGD investigations have been included in Appendix B.

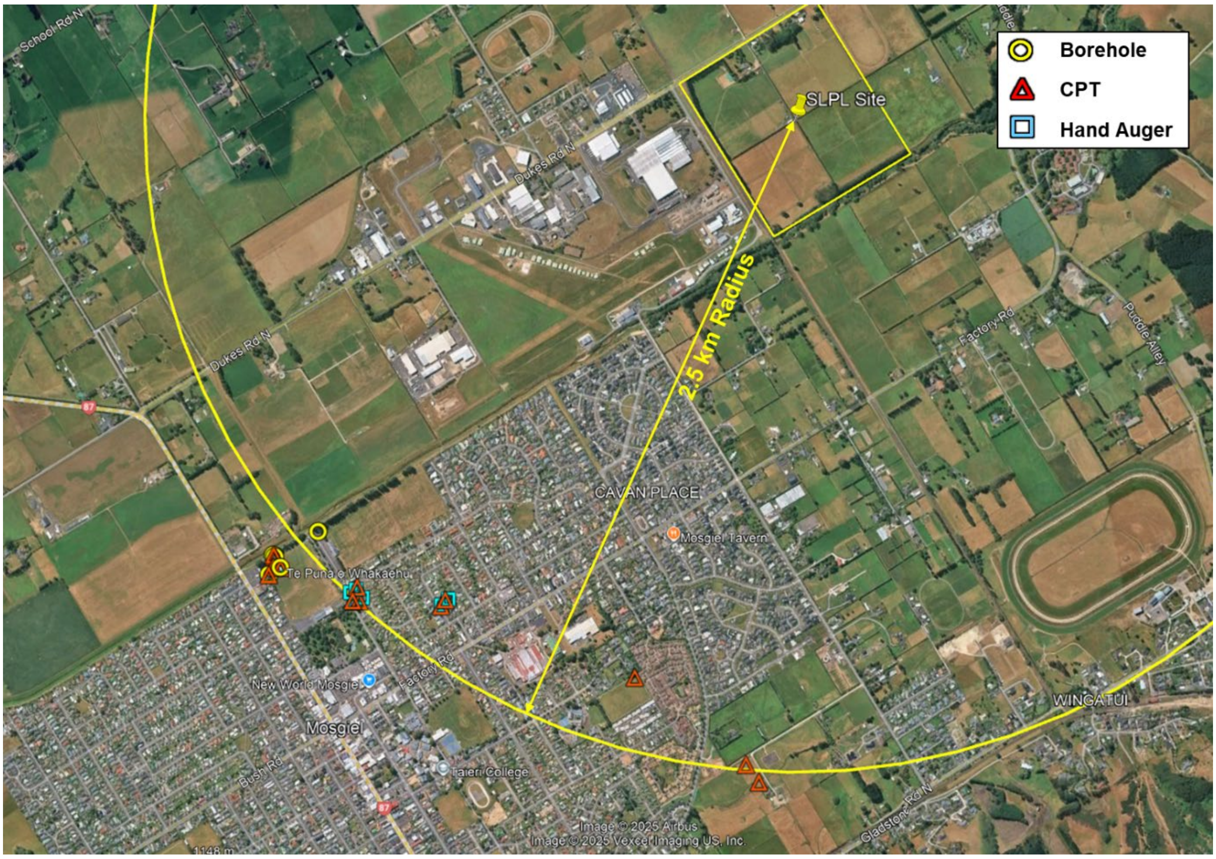

Figure 9. *NZGD investigation locations*

Relatively consistent ground conditions were observed in the various NZGD investigations across Mosgiel, with some local variation. Groundwater was typically encountered at depths between 1.5 to 4 m bgl. The subsurface profile can generally be described as follows (sequentially from top to bottom):

- Approximately 1 to 3 m of fine-grained soils, including soft to firm silts (clayey to sandy), loose sands and silty sands. Reported CPT soil behaviour types included Clays, Silt Mixtures, and Sand Mixtures.
- Approximately 1 to 4 m of sand and gravel mixtures.
- Additional interlayered fine-grained and coarse-grained layers to maximum investigation depths up to 15 m bgl.

In general, shallow subsurface conditions encountered in the NZGD investigations were consistent with those observed in the Site TPs. Based on the geology of the Taieri Plain, Site ground conditions at depths below the final TP investigation depths (2.4 to 2.5 m bgl) are expected to be similar to those encountered in the NZGD investigations, though perhaps with greater thicknesses of gravel deposits (and deeper groundwater; see Section 6.2) due to the Site's higher-elevation location near the northeastern end of the basin.

5.3 Wells Aotearoa New Zealand Database

Historical well lithology reports were available for a limited number of bore locations near the Site that are documented in the Wells Database. While not suitable for geotechnical assessment, the lithology reports nonetheless provide an indication of a deep soil profile. Drillers logs noted soil deposits and no bedrock to depths of around 30-40 m bgl.

6 Conceptual Ground Model

6.1 Ground Conditions

A conceptual ground model has been inferred for the site based on the assessment of site-specific and NZGD investigations described above.

Table 3. Conceptual Site ground model

Unit	Bottom Depth (m bgl)	Material Description
1	1.1 – 1.6	Fine-grained soils (silts, clays, and fine silty sands)
2	3 - 7	Gravel and sand mixtures
3	>7	Interlayered fine-grained soils (silts, clays and fine silty sands) and coarse-grained soils (sand/gravel mixtures)

Given the lack of site-specific data below depths of 2.4 to 2.6 m bgl, as well as no site-specific geotechnical data from in-situ tests such as CPTs or Standard Penetration Tests, the conceptual model has been developed based on the following key assumptions:

- Site lithology is assumed to be similar to that encountered at deeper NZGD investigations based on local geology and similar shallow ground conditions observed in the Site TPs.
- Additional fine-grained layers are likely present at greater depths underlying the gravel/sand layer in which the Site TPs terminated, similar to the profile observed at NZGD locations.

6.2 Hydrogeological Conditions

The Site is situated in a relatively high area of the Taieri Plain (approximately 25-30 m above sea level) described as a groundwater recharge area by the *Lower Taieri Groundwater Allocation Study* (Rekker & Houlbrooke, 2010). That study found that a segment of Silver Stream, including the length along the Site boundary, experiences net losses to groundwater.

Data loggers have recently been installed in on-site bores, and groundwater level monitoring is ongoing at the time of writing. Preliminary standpipe piezometer readings were taken at the time of instrumentation installation at the three well locations shown in Figure 10.

Figure 10. Site groundwater monitoring locations

Measured groundwater levels, likely representative of average levels considering no extreme rainfall events have occurred in recent months, were as follows:

- IP-S2 – 7.85 m bgl
- IP-P5 – 8.15 m bgl
- IP-P5b – 7.55 m bgl

Groundwater levels at IP-P5 and IP-P5b are similar to the base level of the Silver Stream channel in that area. As previously noted, Silver Stream is incised by approximately 7 to 10 m relative to the Site ground level (increasing from east to west).

The readings indicate groundwater is deeper at the Site compared to the NZGD locations (1.5-4 m bgl), consistent with the Site being more elevated above Silver Stream compared to those locations. Further downstream in Mosgiel, the base of the stream is typically about 3 to 4 m below the surrounding topography.

For geotechnical assessment purposes, Stantec has assumed a maximum groundwater level of approximately 5 m bgl at the Site, assuming rain events cause a rise in water levels from the measured values listed above.

7 Geotechnical Considerations

7.1 General Design Assumptions

Design is expected to proceed based on the following general assumptions:

- Structural seismic loads are to be determined via either:
  - **NZS 1170.5:2004** (incorporating amendment 2016) for easier consenting, or
  - **SNZ TS 1170.5:2025** for future proofing the project
- The Site and buildings are Importance level 2 (IL2) – typical commercial and industrial buildings – with a design life of 50 years.
- While inland "port" terminology is sometimes used operationally, the facility is not expected to function as a maritime structure or lifeline asset. Accordingly, port-specific seismic design criteria such as PIANC or ASCE 61 are potentially not warranted, and standard New Zealand industrial building design criteria may be appropriate.

7.1.1 Standards and Guidelines

Pertinent standards and guidelines for design are expected to include:

- Standards
  - NZS 1170.5:2004
  - ASCE 61-14 (Seismic Design of Piers and Wharves)
- Technical Specifications
  - SNZ TS 1170.5:2025
- Guidelines
  - PIANC *Working Group Report No. 165-2015, Design and Maintenance of Container Terminal Pavement*, or the upcoming update (PIANC WG243)
  - MBIE Earthquake Geotechnical Engineering Practice guidelines

7.2 Earthworks

Large earthworks volumes are anticipated (approximately 250,000 cubic metres of total cut material) to facilitate excavations to subgrade levels for the various Site facilities and structures. Expected subgrade depths for major Site facilities (excluding footpaths) are approximately 0.6 to 0.7 m bgl. The greatest depths (0.7 m bgl) are anticipated at the Container Depot where the heaviest applied loads are likely. The preliminary cut/fill balance has been assessed as part of the Geometrics and Grading report included in the Substantive Fast-Track Application package and indicates a net cut volume of approximately 171,000 m^3, including topsoil (some of which will be stockpiled on Site during construction). The cut/fill balance is expected to be further assessed during detailed design to evaluate stockpile requirements, the use of excavated materials as fill for noise barrier bunds or other appropriate applications, and/or the potential off-site disposal of some excavated material.

Potential quarries for source materials have also been identified for imported material, and laboratory testing has been performed for initial evaluation of material suitability. The intended material is AP65 Basecourse sourced from the Finlayson Road Quarry. Preliminary CBR test results from December

2025 for material sampled in November 2025 indicate a CBR at 5.0 mm of 70% for the AP65, with a fines percentage of 10%.

Initial planning indicates excavations will proceed in stages for different areas of the Site to avoid worst-case scenarios involving site-wide excavations, which exacerbate potential flooding and stormwater drainage issues.

The following items include key considerations to be further assessed and verified during the detailed design phase:

- Quality/suitability and available volume of source fill material shall be further verified during detailed design.
- Stormwater assessments will be conducted during the design phase, in addition to those already in progress and documented separately for resource consent purposes. These assessments will verify the potential worst-case impacts of staged excavations on flooding and water accumulation, ensuring the safe drainage or discharge of stormwater through the proposed collection pond and/or to Silver Stream.
- Slope stability assessments shall be undertaken during detailed design (following ground investigations and geotechnical interpretation), considering the effects of earthworks in proximity to the incised stream face (see Section 7.5) and adjacent infrastructure outside of the Site boundaries (e.g., the Taieri Branch railway line bridge and abutments at the Silver Stream crossing), as well as internal stability of excavation sidewalls.
- Impacts of Site earthworks on adjacent road and railway infrastructure (e.g., Taieri Branch Line, Stedman Road, Dukes Road North) and existing buried services along the Site perimeter shall be further evaluated during design.
- Appropriate excavation side slope angles shall be implemented during construction, with buffer zones for worker safety.

7.3 Bearing Capacity and Settlement

The upper fine-grained soil layer presents potential bearing capacity issues, based on the previous identification of potentially compressible soils (e.g., clays and highly plastic silts) in some areas of the Site. Clays and highly plastic silts were identified in test pits TP1, TP2, TP4, and TP5, spaced over an area covering more than half of the Site (see Figure 7), and could be present in other areas not yet investigated. The fine-grained layer, generally extending to depths of 0.8 to 1.1 m bgl (though up to 1.6 m bgl in southeastern areas near Silver Stream), is expected to be partially excavated across much of the Site based on the anticipated subgrade depths.

Where a substantial layer of fine-grained soil remains following subgrade excavations, particularly in the vicinity of Silver Stream where TPs indicated the depth to gravelly material is greatest, additional excavations or ground improvements may be required, depending on the specific properties (e.g., plasticity, compressibility, etc.) of the encountered soils. Such measures shall be further assessed and managed following interpretation of site-specific ground investigations and during detailed design using calculations for expected bearing capacities and settlements.

The following items include key assumptions and considerations to be further assessed and verified during later stages of the project:

- A maximum of four full shipping containers will be allowed to be stacked vertically, potentially inducing ground deformation in compressible fine-grained soils (possibly present throughout the container yard) due to container self-weights and the weight of contents.
  - Anticipated container loads will be further assessed during design based on PIANC *Working Group Report No. 165-2015, Design and Maintenance of Container Terminal Pavement*, or the upcoming update (PIANC WG243).
  - Up to six empty containers may be stacked vertically (based on height restrictions associated with the Taieri Aerodrome flight fan); potential settlement effects from container self-weights, much lighter than the full container weights, will be evaluated during the design.
- Settlement risk in shallow fine-grained soils, as well as deeper fine-grained layers that may be underlying the sand/gravel layer (i.e., below about 3-7 m bgl) shall be quantified and verified during the design, considering staged construction of pavement subbase and application of pavement loads.
- Potential settlement effects on adjacent infrastructure outside of the Site boundary (such as the Taieri Branch railway line, Stedman Road, Dukes Road N, or any other as yet unidentified services) shall be assessed during the design.

The potential bearing capacity and settlement risks described above are manageable given additional Site geotechnical data. Upcoming ground investigations at the Site are expected to result in an improved characterisation of ground conditions and geotechnical properties, as well as verification of the expected level of risk. This information, in conjunction with assessed tolerance limits for Site structures, will be used to develop specific mitigation strategies for potential bearing capacity or settlement issues, if necessary.

7.4 Liquefaction, Cyclic Softening and Lateral Spreading

7.4.1 Liquefaction Hazard Mapping

According to (Barrell, Cox, Glassey, & Lyttle, 2014), the Site is in an area mapped as Liquefaction Domain A, comprising little or no likelihood of liquefaction. Figure 11 shows the associated Dunedin City Council hazard mapping for the site area based on the 2014 GNS study.

Ground predominantly underlain by rock or firm sediments, having little or no likelihood of liquefaction occurring.

Source: *Assessment of liquefaction hazards in the Dunedin City district, GNS Science Consultancy Report 2014/068* (new window)

For more information visit the *Dunedin City Council website* (new window)

Figure 11. Dunedin City Council liquefaction hazard mapping for the Site area

The GNS study developed the Domains accounting mainly for geological factors (e.g., limited to coarse silt and fine sand material types) and not necessarily other geotechnical characteristics and behaviours. New studies since that study's publication in 2014 have shown that other soil types (such as gravels) may also be susceptible to liquefaction when assessed using shear-wave velocity profiles. Therefore, liquefaction cannot be ruled out for the Site based on this map alone.

7.4.2 Preliminary Assessment of Liquefaction Susceptibility

Since Site investigations only included test pits, no other site-specific geotechnical data is currently available to quantify liquefaction susceptibility. Key information required for liquefaction assessments includes the site subsoil class and time-averaged shear-wave velocity in the upper 30 metres bgl (V_{s30}). The currently available data is insufficient to assess these parameters, since no geophysics data is available, NZGD CPTs are too shallow to estimate V_{s30}, and NZGD boreholes do not have downhole shear-wave velocity data. Additional site-specific data will be required to confirm liquefaction susceptibility and triggering potential of site soils during detailed design and prior to construction.

Based on Site TP findings and the expected Site groundwater levels, the upper fine-grained layer identified at the Site is likely non-liquefiable due to:

- The presence of plastic, clayey materials based on Atterberg limits testing (CL: low plasticity clay and CH: high plasticity clay); however, these materials could potentially be susceptible to cyclic softening without groundwater present, i.e., structural degradation of the soil fabric under cyclic strain (less severe effect than in saturated conditions).
- Non-plastic silts are likely to be located above the water table and unsaturated.

Stantec undertook a preliminary liquefaction assessment and parametric analysis of select NZGD CPTs to further evaluate deeper materials expected to be encountered at the Site. The liquefaction triggering assessment indicated that saturated sand/silt mixtures in the upper 10 m bgl are potentially liquefiable for a range of potential Ultimate Limit State (ULS) Peak Ground Accelerations (PGA), as described below.

To determine seismic demand for the Site, two different New Zealand earthquake loading frameworks are considered: NZS 1170.5:2004, the Building Code–referenced standard that derives design actions using return-period-based AEP values, and the more recent SNZ TS 1170.5:2025, a technical specification incorporating the updated 2022 National Seismic Hazard Model. While NZS 1170.5 provides PGA values consistent with traditional hazard zoning and Importance Level requirements, TS 1170.5 introduces revised hazard estimates and updated geotechnical parameters, resulting in higher and more contemporary representations of seismic demand. The details are shown in Table 4.

Table 4 Seismic Demand Comparison

Standard	Basis	Underlying Seismic Hazard Model	Relevant PGA (g)
NZS 1170.5 2004	Importance Level 2 50-year Design Life 1/500 Annual Exceedance Probability (AEP) - ULS Soil Class C (assumed)	Uses 2002 National Seismic Hazard Model (NSHM).	0.17
SNZ TS1170.5:2025	1/500 AEP - ULS V_{s30} of 300-400 m/s	Uses 2022 NSHM, incorporating 20 years of updated science and fault data.	0.25 – 0.26

Five selected NZGD CPTs were evaluated, all located in Liquefaction Domain B – low to moderate likelihood of liquefaction (yellow zone shown in Figure 11). The assessment considered a groundwater level at 5 m bgl, corresponding to the assumed maximum level expected for the Site.

Parametric analyses are presented below for Liquefaction Potential Index (LPI) and liquefaction-induced settlements versus PGA across a range of PGAs. Figure 12 and Figure 13 present the analyses of LPI and expected free-field settlements, respectively.

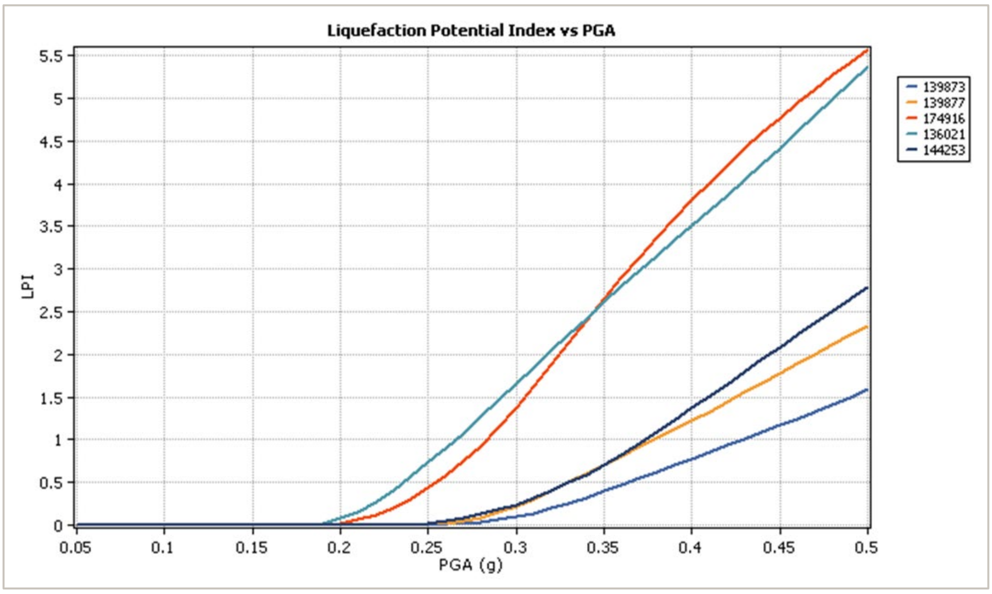

Figure 12. *Liquefaction Potential Index versus PGA (expected Site groundwater – 5 m bgl)*

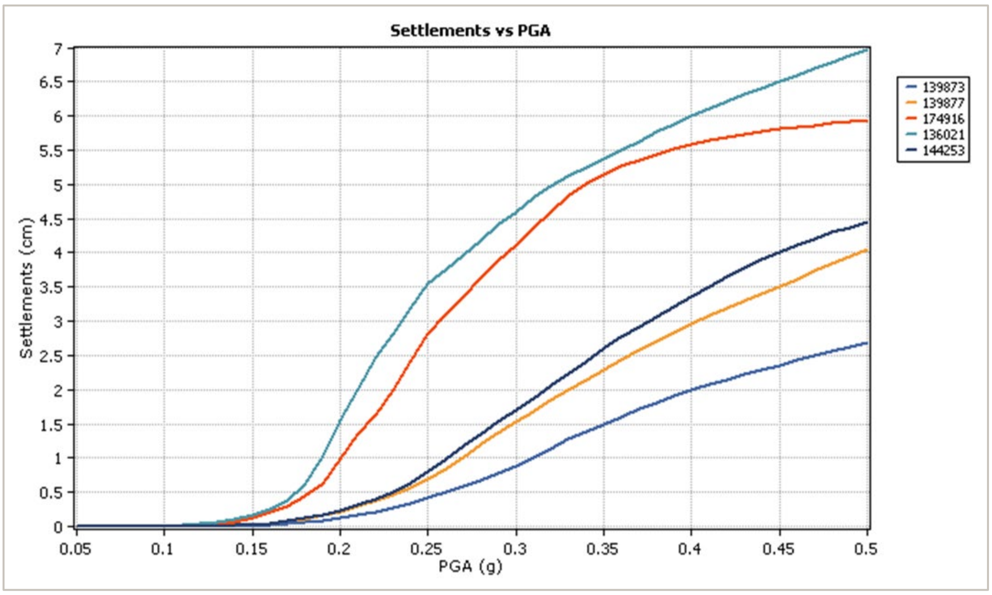

Figure 13. *Liquefaction-induced settlements versus PGA (expected Site groundwater – 5 m bgl)*

LPI can be correlated to expected liquefaction risk based on categories developed by Iwasaki et al. (1978), as defined in Table 5.

Table 5. *LPI-based liquefaction risk categories (Iwasaki et al., 1978)*

Liquefaction Potential Index	Liquefaction Risk
0	Very Low
0-5	Low
5-15	High
≥15	Very High

Although the CPT profiles are from Domain B locations (see Figure 9 and Figure 11), LPI results from the parametric analyses suggest a Low risk of liquefaction when the expected Site (Domain A) groundwater level (5.0 m bgl) is considered. The analyses further indicate that liquefaction-induced settlements less than 4 cm are expected for PGA up to 0.26 g.

The following key assumptions and considerations have been identified for further assessment and verification during detailed design of the project:

- **Possible (as yet unidentified) silt/sand mixtures underlying sand/gravel layer** (i.e., below about 3-7 m bgl) may be below the groundwater table (expected to be approximately 5.0 m bgl) and potentially liquefiable, though expected liquefaction-induced settlements are small and considered manageable for design.
- **Existing NZGD investigation data** have been assumed to be appropriate for the initial estimation of liquefaction susceptibility for similar material types anticipated at the Site. Additional ground investigations will be required prior to construction to obtain site-specific parameters for confirmation of the liquefaction hazard.
- **Lateral spreading risk** exists adjacent to Silver Stream, associated with the presence of a significant free face formed by steep, high banks, and potential layers susceptible to liquefaction or strain softening underlying a sand/gravel crust above the stream base elevation.
- **Based on NZS 1170.5 2012, the site subsoil class is not A, B, or E**, considering historical well lithology reports showing potential depths of soil deposits. Whether the site subsoil class is C or D shall be confirmed during design, following additional Site ground investigations.
- **Site-specific liquefaction, cyclic softening and lateral spreading hazards** shall be quantified during preliminary/detailed design stages using new Site ground investigation data, and the risk appropriately managed during detailed design and construction.

7.5 Slope Stability and Erosion

As discussed in Section 3.1, areas of potential slope instability and erosion along Silver Stream have been identified from historical aerial photographs and recent LiDAR topography data. Although these features (identified in Figure 3) appear to have initiated prior to the year 1985, and no other recent instabilities or slope failures are evident in the LiDAR data, it is possible that ongoing erosion has occurred along the stream bank.

These observations, particularly the possible subsidence area, suggest a potential risk of shallow transitional slope failures affecting the stream bank and potentially a sizable area extending beyond the top of the bank and into the Site. Based on the groundwater characterisation, the toe of the slope is expected to remain saturated as groundwater is recharged from Silver Stream, which may contribute to the potential for future slope instability.

A site visit was undertaken by **e3 Scientific** (e3) as part of a Site freshwater ecological assessment focusing on Silver Stream. During their site visit, e3 observed railway iron installed at the base of the stream banks that appears to have been placed as a form of slope protection, though the original intent is unknown. Figure 14 presents a photograph from the e3 site visit showing the iron inserted vertically into the ground at the base of the slope.

Figure 14. Railway iron (highlighted by a yellow circle) placed along the Silver Stream bank (photograph provided by e3 Scientific)

According to e3, the iron rails appeared to have been placed along most of the stream length along the Site boundary, presumably at a similar spacing as shown in the photo above. The timing of installation is unknown since they are not visible in historical aerial imagery. Nonetheless, their presence suggests that bank erosion has been a concern in the past. Whether the measures have been effective at stabilising the bank remains unknown at this stage, though they will be further assessed during and after the upcoming site walkover and geotechnical investigations, as described below.

The following activities will be undertaken prior to detailed design to inform the assessment of slope stability along Silver Stream:

- Site walkover / geological survey by a qualified Engineering Geologist to visually assess existing conditions along the length of the stream bank forming the Site boundary, and to confirm the preliminary findings described herein, which have been developed from aerial imagery and LiDAR data. The condition of the existing railway iron pieces will also be observed during the walkover to inform geotechnical assessments of their effectiveness.
- Geotechnical investigations and laboratory testing to characterise the ground profile, identify potential layers contributing to slope instability, develop geotechnical material parameters, and perform slope stability analyses.

The contributing factors to slope instability appear to primarily be a result of natural processes along Silver Stream and not necessarily associated with adjacent land development activities (since the area has persisted as farmland since at least 1942). However, vegetation removal in past decades may have also contributed to erosion and instability along the stream banks.

Findings from upcoming geotechnical investigations and analyses will be incorporated into the detailed design of Site infrastructure near Silver Stream to reduce the potential for Site activities to exacerbate existing instability. Mitigation measures will depend on the findings but could include slope stabilisation measures (to improve the resistance of slopes to failure) and/or the implementation of minimum setback distance requirements between the stream bank and new Site infrastructure (to reduce or eliminate additional loadings on slope crests).

Potential stabilisation measures, if required, would be assessed with the intent to avoid impacts to the stream itself, and could include the installation of shear piles along the top of the slope to cut off potential slope failure planes. However, additional investigations will inform whether potential slope instability is more likely to be a result of subsurface ground conditions in the slope or from progressive erosion of the stream bank at the toe of the slope (causing undercutting). If the latter case is true, then erosion control measures on the slope face and/or at the bottom of the bank may be required (e.g., rock armouring, geotextile placement, or additional revegetation).

8 Conclusions

Based on the review of available information and preliminary site assessments, the available data is considered sufficient to support the resource consenting process. Upcoming ground investigations will further improve the characterisation of Site ground conditions and groundwater levels at the detailed design stage, allowing for in-depth assessments of risks and potential mitigation methods.

- Test pits and NZGD data provide a consistent subsurface profile supported by regional geology, with fine-grained soils overlying a sand/gravel layer, followed by additional interlayering of fine-grained and sand/gravel layers. Thicker deposits of the uppermost fine-grained soil layer are expected in southeastern areas of the Site near Silver Stream.
- On-site groundwater level measurements, as well as assumptions based on hydrogeological assessment of topography and regional datasets, are considered reasonable for preliminary design.
- The anticipated excavation to subgrade level will partially remove weaker soils, reducing the immediate large settlement risk, though manageable risks remain in areas potentially containing clay or highly plastic silts. Long-term settlement risk relating to Site operations can be assessed during detailed design, with improved identification and assessment of potentially compressible soils following additional Site geotechnical investigations.
- Preliminary earthworks volumes and material sourcing strategies have been identified, with the client's commitment to manage fill requirements.
- Regional mapping and preliminary assessment of NZGD data indicate relatively low liquefaction risk overall for the expected site conditions (based on estimated LPI and settlements), with manageable lateral spreading risk near Silver Stream. Site-specific seismic hazard and liquefaction susceptibility shall be further quantified during detailed design to reduce uncertainties.
- Slope stability risks are anticipated along Silver Stream based on the identification of potential instability features from aerial imagery and LiDAR topography data. Given the relatively consistent geology across the Site, instability is possible (assuming some triggering mechanism resulting from a change in ground conditions) at any point along the stream boundary, including locations outside of the already identified instability areas. Field surveys and ground investigations shall be undertaken prior to design to characterise the existing slope conditions and obtain geotechnical data for stability assessments. Field findings will be used to inform mitigation methods for design.
- Targeted geotechnical investigations are strongly recommended (and as stated above, expected to proceed) at the start of the design phase to confirm site ground conditions, establish the V_{s30} in case the SNZ TS 1170.5:2025 pathway is adopted, and acquire data to quantify the hazards identified above.

Overall, the assumptions and supporting data provide a reasonable basis for resource consent, with residual geotechnical risks clearly identified and mitigation strategies outlined for the coming detailed design stages.

9 References

Barrell, D. J., Cox, S. C., Glassey, P. J., & Lyttle, B. S. (2014). *Assessment of liquefaction hazards in the Dunedin City district.* GNS Science Consultancy Report 2014/068.

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10 Geotechnical Report Limitations

This geotechnical report (the report) has been prepared in accordance with the scope of services set out in the contract based on your project-specific requirements and criteria. In some circumstances the scope of the report may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

Stantec may have relied upon data, surveys, analyses, designs, plans and other information provided by the client and other individuals and organisations, (the data). Except as otherwise stated in the report, Stantec has not verified the accuracy or completeness of the data. Stantec will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to Stantec.

This report was prepared expressly for the client and expressly for purposes indicated by the client or its representative. This report may not be relied upon by any other persons for any purpose. The client should not use this report for other than its intended purpose without seeking additional geotechnical advice. The report applies only to the site investigated as outlined within. This report should not be relied upon if there are any changes to the project without first asking Stantec to review our recommendations and design.

Our ground conditions assessment is based on publicly available information and recorded subsurface conditions at the selective discrete test and sampled locations, the type, spacing and frequency of which were selected to meet the project requirements agreed by the Client. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. Our overall interpretation is based on inferred soil, rock, and groundwater conditions between discrete points; actual conditions may differ from those inferred. Stantec accepts no liability for any unknown or adverse ground conditions that would have been identified had further investigations, sampling, and testing been undertaken. No warranty is expressed or implied that the conditions encountered following investigation or during construction will conform to the conditions described herein.

Subsurface conditions are created by natural processes and human activities that evolve and change over time and can result in changes to ground conditions. Groundwater levels presented in this report may vary over time due to diurnal, tidal and seasonal influences. Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events and should be consulted to determine if additional tests are necessary.

Ground conditions cannot be fully substantiated until project implementation has commenced and therefore this report's recommendations require confirmation onsite during construction. As such, uncertainty in ground conditions should be verified by Stantec geotechnical professionals required at nominated design milestones and during construction. Only Stantec, who prepared this report, is fully familiar with the background information needed to assess whether this report's recommendations are valid and whether changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report, there is risk of misinterpretation and Stantec cannot be held responsible for such misinterpretation. Stantec should be notified and be given an opportunity to review the report recommendations made in this report where conditions encountered at the site differ from those inferred; and if there are changes to design or construction methodologies.

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

Appendices

Appendix A Historical Aerial Photographs

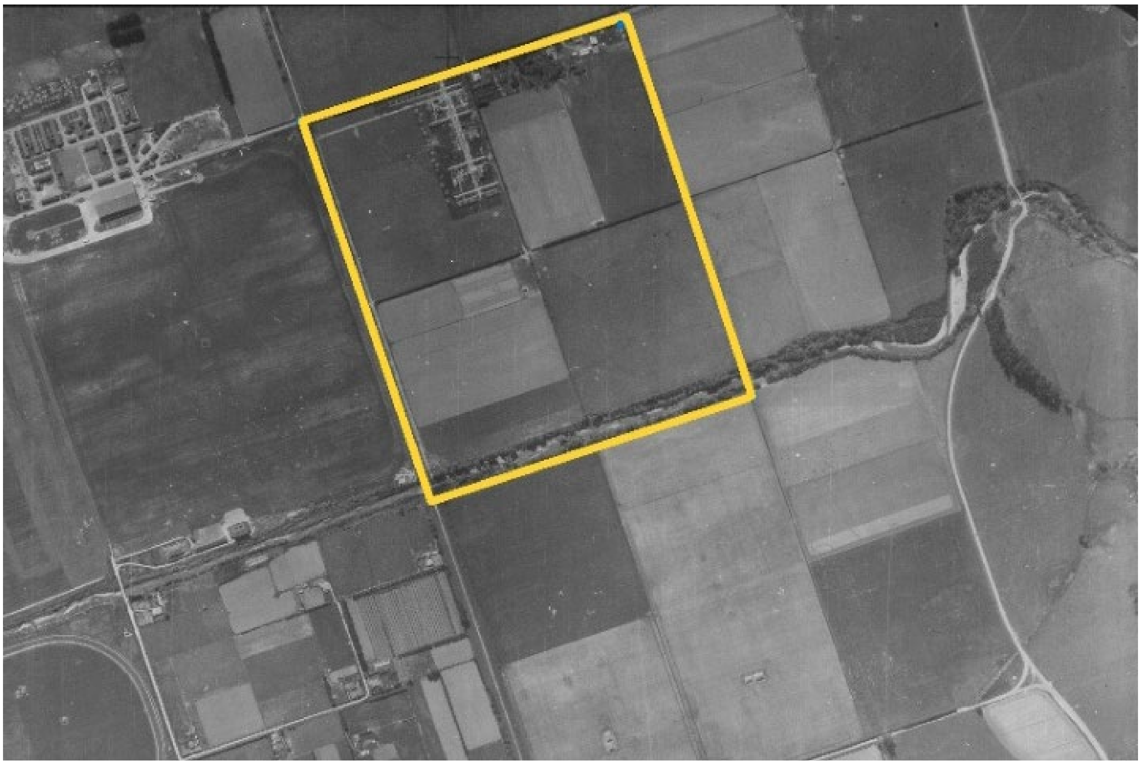


Figure A1. 1942 aerial imagery (Retrolens)

Figure A2. 1950 aerial imagery (Retrolens)

Figure A3. 1963 aerial imagery (Retrolens)

Figure A4. 1985 aerial imagery (Retrolens)

Figure A5. 2005 aerial imagery (Google Earth)

Figure A6. 1963 aerial imagery of Silver Stream (Retrolens)

Figure A7. 1970 aerial imagery of Silver Stream (Retrolens)

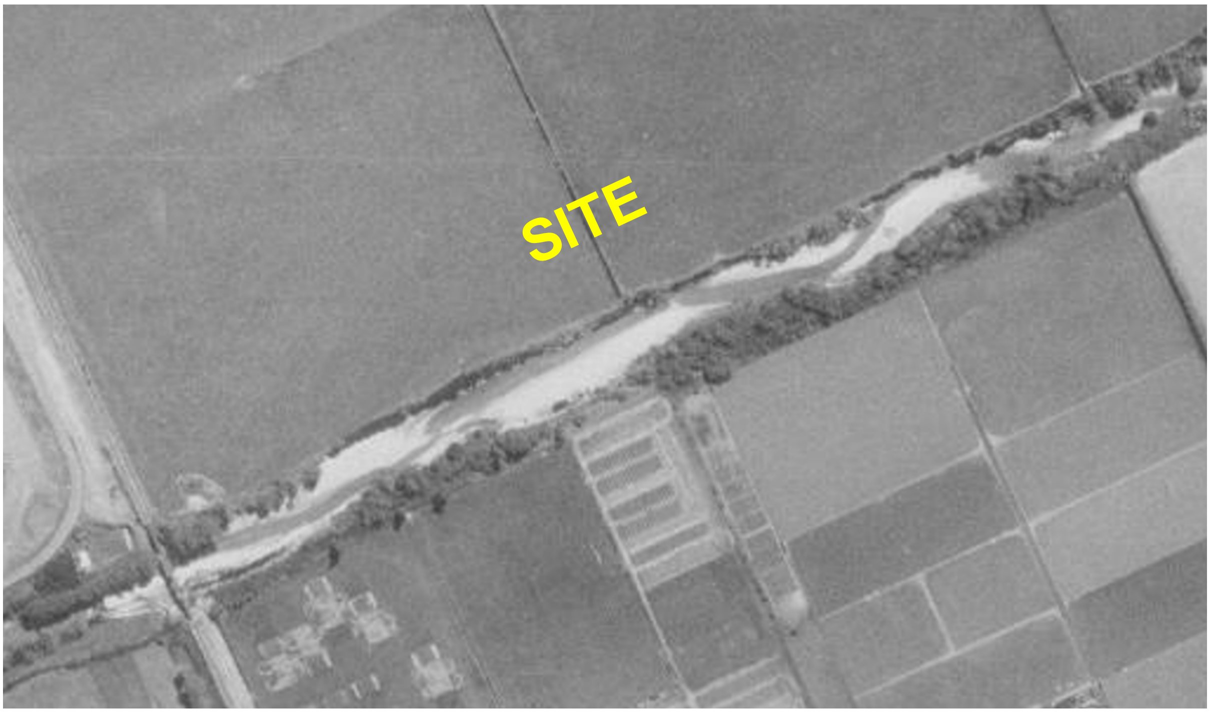


Figure A8. 1975 aerial imagery of Silver Stream (Retrolens)

Figure A9. 1985 aerial imagery of Silver Stream (Retrolens), showing first noted occurrence of potential slope instability area (circled in red) near southeastern corner of Site

Appendix B NZGD Logs

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION	**BH01**

Client	Dunedin City Council
Project	Mosgiel Pump Station Upgrade
Project number	60555639

Co-ordinates	1394188mE 4917296mN
Orientation	-90° Elevation 22m (Approx)
Location	5 Reid Avenue, Mosgiel
Feature	Carpark near stopbank.

GEOLOGICAL DESCRIPTION | Test Records (Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50) | Drilling Method Casing remarks | Core Loss/Lift 0 - 100% | Depth | Graphic Log | Instrumentation

SOIL PROPERTIES
Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

Geological Description	Soil Properties
0m: Asphalt.	0m: Asphalt.
0.1m: Subbase.	0.1m: Silty, sandy fine to medium GRAVEL; dark brown. Tightly packed; wet; well graded; angular to subangular; schist; sand is fine to coarse.
0.25m: Non-Engineered Fill.	0.25m: Sandy SILT; dark brown. Firm to stiff; wet; non-plastic.
	0.4m: Sandy fine to coarse GRAVEL with some silt; brown. Wet; well graded; subrounded to subangular; sand is fine to coarse.
	0.6m: SILT with some sand and gravel; brown, grey and black. Firm; moist; sand is fine to medium; gravel is medium; black material is odourless, possibly reworked bituminous material.
	1m: Sandy SILT; greyish brown. Soft; wet; non-plastic; sand is fine.
	1.25m: CLAY with minor silt and trace organic material; grey with brown mottles. Soft; moderate plasticity; organic material is rootlets.
	1.6m: SILT with minor clay and trace organic material; grey with brown mottles. Soft; wet; non-plastic; organic material is rootlets.
	1.95m: Silty fine SAND; grey. Loose; wet; uniformly graded.
	2.1m: Sandy fine to medium GRAVEL; light brownish orange. Medium dense; moist; well graded; angular to subangular; schist; sand is fine to coarse.
	2.45m: Gravelly fine to coarse SAND with minor silt; light brown and black. Medium dense; moist; well graded; gravel is fine to medium, angular to subangular, schist; black material has slight hydrocarbon odour, possibly reworked bituminous material. 2.6m: No silt.
2.95m: Alluvial Deposits.	2.95m: Sandy fine to medium GRAVEL with minor silt; brown. Medium dense; wet; well graded; angular to subangular; schist; sand is fine to coarse. 3.6m: Gravel is fine to coarse.
	4.1m: Silty, gravelly CLAY with some sand and trace organic material; grey. Soft; wet; low plasticity; gravel is fine to medium, angular to subangular, schist; sand is fine to coarse.
	4.2m: Gravelly fine to coarse SAND; brown. Medium dense; wet; well graded; gravel is fine to medium, angular to subangular, schist.

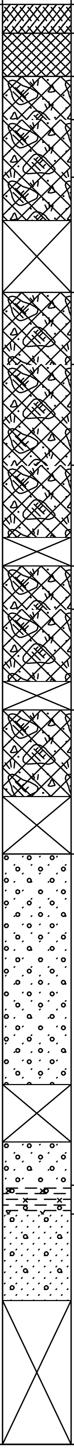
Non Engineered Fill

ALLUVIAL DEPOSITS

Test Records	Drilling Method
	PQWL
	PQWL
ss 2,1,0, 1,1,2 N=4	SPT
	PQWL
ss 1,0,1, 1,1,2 N=5	SPT
	PQWL
ss 2,5,6, 6,7,6 N=25	SPT
	PQWL
ss 3,5,5, 5,5,4 N=19	SPT
	PQWL
ss 2,3,1, 2,4,4 N=11	SPT
	PQWL

Depth: 1, 2, 3, 4

Instrumentation: 1 ▼



GROUNDWATER OBSERVATIONS

Date Time	Piezometer	Reading ID	Water Depth (m)
31/07/2018 09:00		1	1.8

Hand Held Shear Vane

vane shear strength per NZGS guideline

Remarks
Coordinates are in NZTM and are approximate.
SPT hammer efficiency 62.7%.
Hole backfilled with cement grout.
Driller reported soft zones from ~10.5m to 13m bgl.

Casing Details

Depth	Diameter
15	122.6

Date logged	31/07/2018
Logged	JKH
Checked	KDL

Driller	Speight Drilling
Drill Rig	Rotary Core
Core Boxes	5
Started	30/07/2018
Finished	30/07/2018

Page 1 of 3

Date Printed: 7/08/2018

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION	BH01

Client	Dunedin City Council
Project	Mosgiel Pump Station Upgrade
Project number	60555639

Co-ordinates	1394188mE	4917296mN	
Orientation	-90°	Elevation	22m (Approx)
Location	5 Reid Avenue, Mosgiel		
Feature	Carpark near stopbank.		

GEOLOGICAL DESCRIPTION

ALLUVIAL DEPOSITS

Test Records

Shear Vane residual - peak 0 - 200 kPa

N Values 0 - 50

Drilling Method Casing remarks

Core Loss/Lift 0 - 100%

Depth

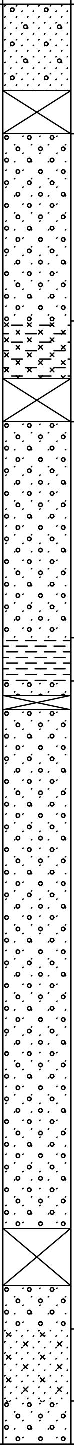
Graphic Log

SOIL PROPERTIES

Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

Instrumentation

Test Records	Drilling Method	Depth
ss 4,4,3, 3,4,5 N=15	SPT	
	PQWL	
ss 4,4,2, 3,3,5 N=13	SPT	6
	PQWL	
ss 3,5,2, 1,3,4 N=10	SPT	7
	PQWL	
ss 3,6,6, 9,8,10 N=33	SPT	8
	PQWL	
ss 2,6,4, 4,2,3 N=13	SPT	9
	PQWL	



5.45m: Sandy fine to medium GRAVEL with trace silt; brown. Medium dense; wet; well graded; angular to subangular; schist; sand is fine to coarse.

6.1m: Clayey SILT with minor gravel; grey. Soft; wet; non-plastic; gravel is fine to medium, subangular, schist.

6.45m: Sandy fine to medium GRAVEL with minor silt; brown. Medium dense; wet; well graded; angular to subangular; schist; sand is fine to coarse.

7.2m: CLAY; grey. Firm; moist; moderate plasticity.

7.35m: Sandy fine to medium GRAVEL with minor silt; brown. Medium dense; wet; well graded; angular to subangular; schist; sand is fine to coarse.

9.6m: Silty fine SAND; brownish grey. Medium dense; wet; uniformly graded.

description next page

GROUNDWATER OBSERVATIONS

Date Time	Piezometer	Reading ID	Water Depth (m)

Hand Held Shear Vane

vane shear strength per NZGS guideline

Remarks

Coordinates are in NZTM and are approximate.
SPT hammer efficiency 62.7%.
Hole backfilled with cement grout.
Driller reported soft zones from ~10.5m to 13m bgl.

Casing Details

Depth	Diameter

Date logged	31/07/2018
Logged	JKH
Checked	KDL

Driller	Speight Drilling
Started	30/07/2018
Drill Rig	Rotary Core
Finished	30/07/2018
Core Boxes	5

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION **BH01**

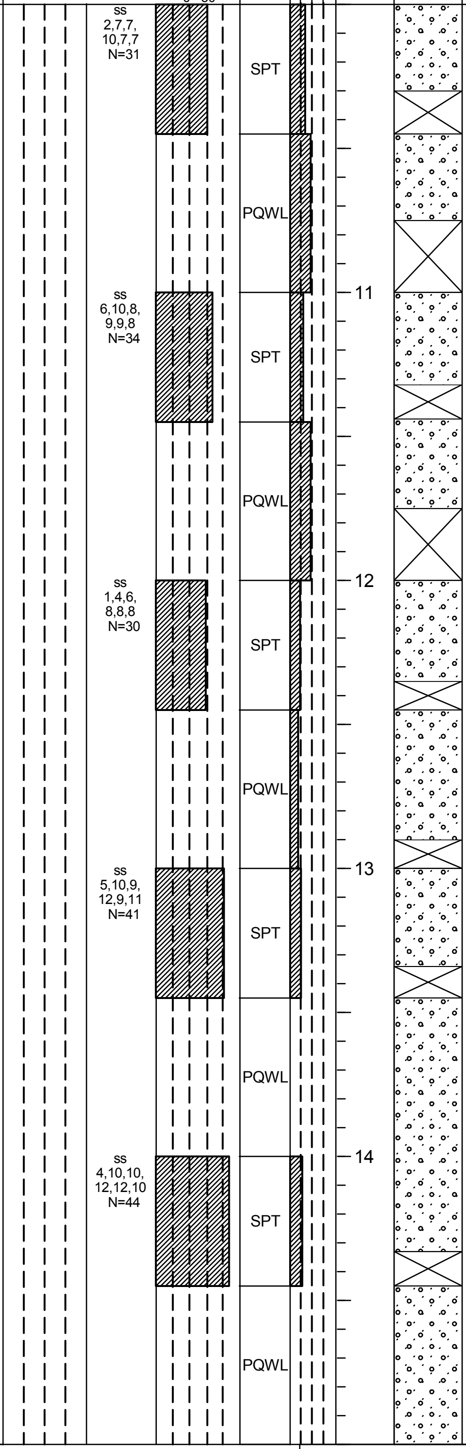
Client	Dunedin City Council
Project	Mosgiel Pump Station Upgrade
Project number	60555639

Co-ordinates 1394188mE 4917296mN
Orientation -90° Elevation 22m (Approx)
Location 5 Reid Avenue, Mosgiel
Feature Carpark near stopbank.

GEOLOGICAL DESCRIPTION | Test Records (Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50) | Drilling Method (Casing remarks) | Core Loss/Lift (0 - 100%) | Depth | Graphic Log | SOIL PROPERTIES | Instrumentation

SOIL PROPERTIES: Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

ALLUVIAL DEPOSITS



Test Records	Drilling Method
ss 2,7,7, 10,7,7 N=31	SPT
	PQWL
ss 6,10,8, 9,9,8 N=34	SPT
	PQWL
ss 1,4,6, 8,8,8 N=30	SPT
	PQWL
ss 5,10,9, 12,9,11 N=41	SPT
	PQWL
ss 4,10,10, 12,12,10 N=44	SPT
	PQWL

9.85m: Sandy fine to medium GRAVEL with minor silt; brown. Dense; wet; well graded; angular to subangular; schist; sand is fine to coarse. *(continued)*

11.45m: Gravel is fine to coarse.

12.6 to 12.75m: Silty.

13.5 to 13.8m: Silty.

BH01 terminated at 15m
Target Depth

GROUNDWATER OBSERVATIONS

Date Time	Piezometer	Reading ID	Water Depth (m)

Hand Held Shear Vane

vane shear strength per NZGS guideline

Remarks
Coordinates are in NZTM and are approximate.
SPT hammer efficiency 62.7%.
Hole backfilled with cement grout.
Driller reported soft zones from ~10.5m to 13m bgl.

Casing Details
Depth Diameter

Date logged 31/07/2018
Logged JKH
Checked KDL

Driller Speight Drilling
Started 30/07/2018
Drill Rig Rotary Core
Finished 30/07/2018
Core Boxes 5

Page 3 of 3

Date Printed:
7/08/2018

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - top of floodbank
Job Number: 12519887
Commenced: 22/02/2020
Completed: 22/02/2020

Hole No. : BH01 (CE16/0110)
Sheet : 1 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404927.5298
Northing: 799298.7413
System: TAIETM2000
RL: 18.6
Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
0	Fine to medium sandy SILT; dark brown. Soft to firm, moist, non-plastic (FLOODBANK FILL)	FLOODBANK FILL	M	S-F
0.55	0.55 - 0.75 m: CORE LOSS (inferred)			
0.75	Gravelly SILT; brown. Gravel is medium grainsize, basaltic			
1	1.00 - 1.20 m: CORE LOSS (inferred)			
1.2	Silty SAND; brown. Very loose, moist, poorly graded; sand is fine to medium; possible buried topsoil (DUKES FORMATION)	DUKES FORMATION	M	VL
2.00	2.00 - 2.45 m: loose			L
2.45	2.45 - 2.78 m: CORE LOSS (inferred)			
2.78	Silty SAND; brown. Loose, moist, poorly graded; sand is fine to medium		M	L
3.13	SILT, minor clay, trace fine to medium sand; light grey and orange-brown. Medium dense, moist, low plasticity		M	VSt
3.4	Sandy GRAVEL, minor silt; brown. Medium dense, moist to wet, well graded; sand is coarse; gravel is fine, angular to subrounded, quartz and schist		M-W	MD
4.90	4.90 - 4.95 m: fine to medium SAND, no gravel			

Sample Number / Type	Result
C 0.30 - 0.50	
1.00	SPT 2/2 1/0 0/0 N = 1
C 1.70 - 2.00	
2.00	SPT 1/1 1/1 2/1 N = 5
3.00	SPT 2/1 2/3 7/9 N = 21
4.00	SPT 5/5 4/4 4/4 N = 16
C 4.50 - 4.70	
5.00	SPT 5/9 8/5 6/5 N = 24

Method	TCR SCR RQD (%)
SNC	80
SPT	56
SNC	100
SPT	87
SNC	40
SPT	89
SNC	100
SPT	78
SNC	91
SPT	76
SNC	100

Water level: 5-03-2020

RL (m) | Depth (m) | Graphic | Material Description | Geological Unit | Moisture condition | Consistancy / Relative density | Sample: Number / Type, Result | Casing | Method | Flush Return (%) 25 50 75 | Weathering | Estimated Strength (MPa) EW VW W MS VS ES | TCR SCR RQD (%) | Defect Spacing (mm) 20 60 200 600 2000 | Instrumentation Installation | Water level

RL (m): 18, 17, 16, 15, 14, 13
Depth (m): 0, 1, 2, 3, 4, 5, 6

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Nested piezos - screens at 0.6 - 1.2 m and 3.5 - 5.5 m

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical
Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - top of floodbank
Job Number: 12519887
Commenced: 22/02/2020 Completed: 22/02/2020

Hole No. : BH01 (CE16/0110)
Sheet : 2 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404927.5298 Northing: 799298.7413 System: TAIETM2000
RL: 18.6 Datum: NZVD2016

RL (m)	Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
12	6	Sandy GRAVEL, minor silt; brown. Medium dense, moist to wet, well graded; sand is coarse; gravel is fine, angular to subrounded, quartz and schist *(continued from layer starting at 3.4m)*	DUKES FORMATION		
11	7.05	SILT, minor to some clay, minor fine to medium sand; grey and orange-brown. Stiff, moist, high plasticity 7.40 - 7.45 m: fine to medium sandy SILT 7.45 - 7.85 m: minor fine, angular to subrounded, quartz and schist gravel 7.85 - 8.10 m: trace fine gravel		M	St
	8.1	Organic SILT, minor clay, minor fine to medium sand; dark brown. Soft, moist, low plasticity; looks like buried topsoil		M	S
10	8.2	Silty SAND; brown and orange-brown. Loose, moist, poorly graded; sand is fine to medium		M	L
	8.6	SILT, minor to some clay, minor fine to medium sand; grey and orange-brown. Stiff, moist, high plasticity		M	St
9	8.75	Silty, sandy GRAVEL; grey and brown. Dense, moist to wet, well graded; sand is coarse; gravel is fine, angular to subrounded, quartz and schist 9.50 - 10.45 m: trace coarse, angular to subrounded, quartz and schist gravel		M-W	D
8	10.45	End of Hole @ 10.45m,Target Depth.			
7	11				

Sample Result	Method	TCR SCR RQD (%)
SPT 8/7 7/6 5/6 N = 24	SPT	67
	SNC	100
SPT 2/1 2/2 3/4 N = 11	SPT	98
	SNC	82
SPT 0/0 0/1 1/2 N = 4	SPT	89
	SNC	100
SPT 10/13 10/12 9/10 N = 41	SPT	76
	SNC	100
SPT 8/10 10/8 9/8 N = 35	SPT	82

Sample: Number / Type; Result. Casing. Flush Return (%) 25 50 75. Weathering. Estimated Strength (MPa) EW VW W MS VS ES. Defect Spacing (mm) 20 60 200 600 2000. Instrumentation Installation. Water level.

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Nested piezos - screens at 0.6 - 1.2 m and 3.5 - 5.5 m

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

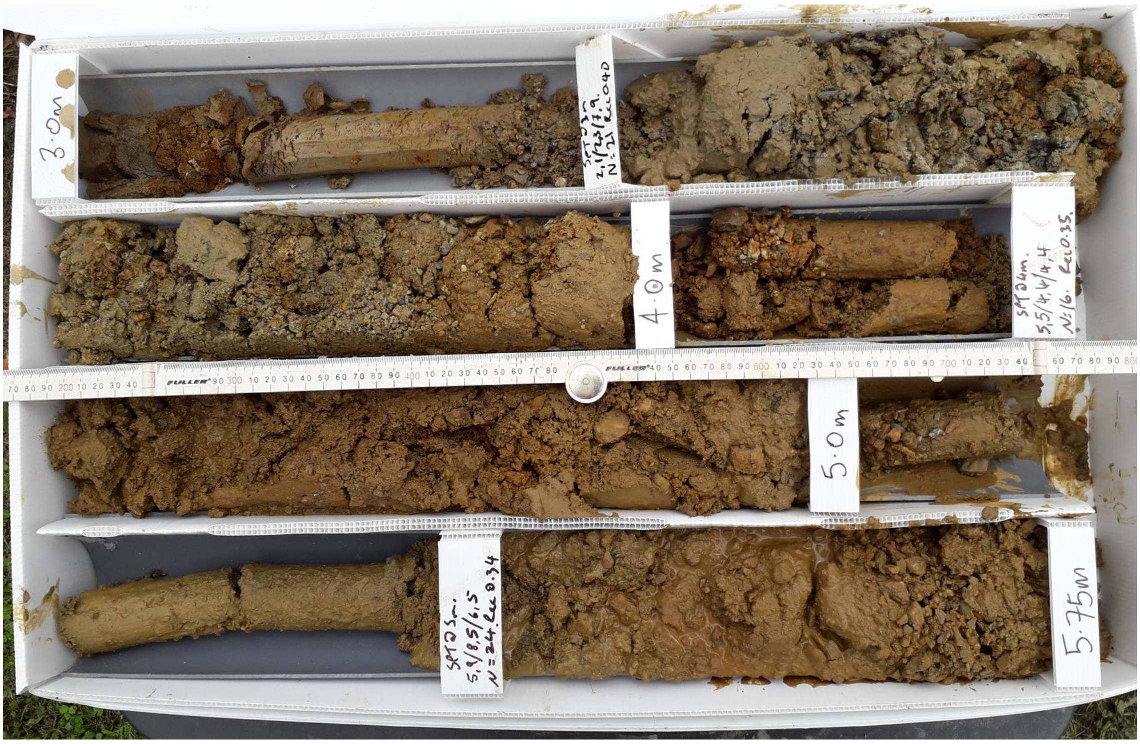
Date	Time	Reading (mbgl)	Hole depth (mbgl)
05/03/20	10:00	5.015	10.45

Report of Photographs

Site Identification: BH01

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	22/02/20	**Completed**	22/02/20
Site	On top of floodbank	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

0.0 m to 3.0 m



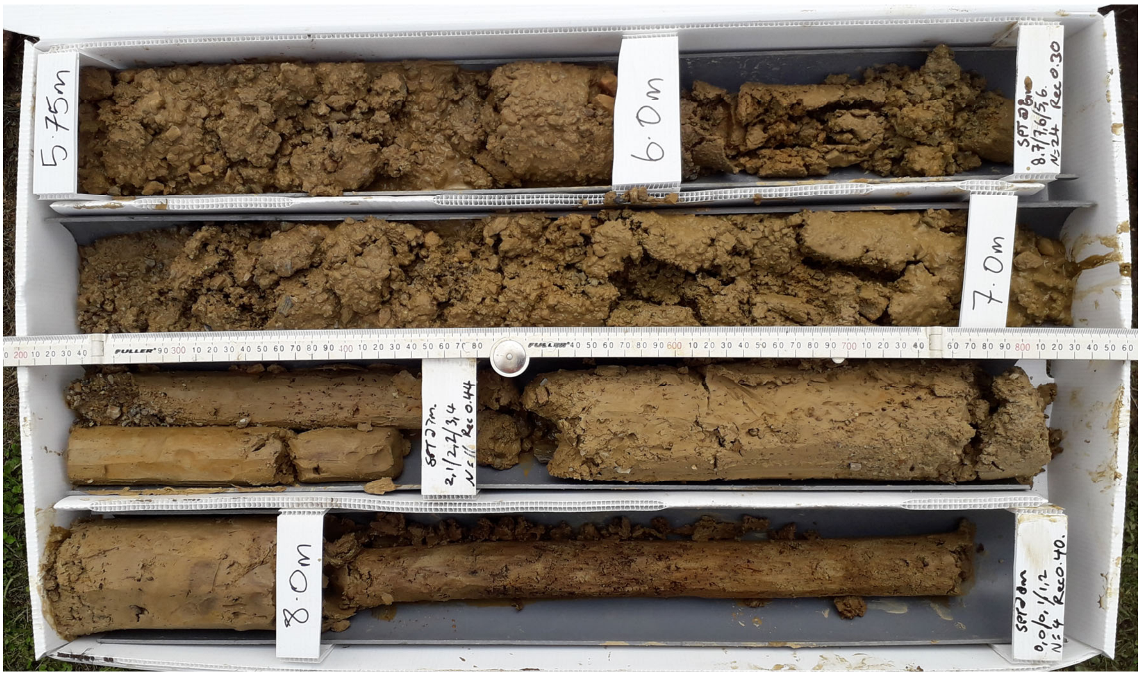
3.0 m to 5.75 m

CLIENTS | PEOPLE | PERFORMANCE

Report of Photographs

Site Identification: BH01

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	22/02/20	**Completed**	22/02/20
Site	On top of floodbank	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		



5.75 m to 8.45 m

8.45 m to 10.45 m (EOH)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - adjacent to ablutions block
Job Number: 12519887
Commenced: 22/02/2020
Completed: 22/02/2020

Hole No. : BH02 (CE16/0109)
Sheet : 1 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404932.59
Northing: 799291.3075
System: TAIETM2000
RL: 17.3
Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
0 – 0.1	Fine sandy SILT; dark brown. Soft, moist to wet, non-plastic (TOPSOIL / FLOODBANK FILL)	FILL	M-W	S
0.1 – 0.3			M	St
0.3 – 0.5	Fine SAND / SILT intermixed; brown. Stiff, moist, low plasticity (FLOODBANK FILL)		M	L
0.5 – 1	Silty SAND; brown. Loose, moist, poorly graded; sand is fine to medium (DUKES FORMATION) 0.50 - 1.00 m: CORE LOSS (inferred)	DUKES FORMATION		
1 – 1.2	Silty SAND; brown. Loose, moist, poorly graded; sand is fine to medium		M	L
1.2 – 1.8	20 mm Organic SILT; dark grey SILT, some fine to medium sand, minor clay; light grey with orange-brown mottles. Soft to firm, moist, low plasticity		M	S-F
1.8 – 5.25	Sandy GRAVEL, minor to some silt; grey and orange-brown. Medium dense, moist, poorly graded; sand is coarse; gravel is fine, angular to subrounded, quartz and schist 2.45 - 2.50 m: 50 mm silty fine to medium SAND 2.80 - 2.90 m: 100 mm fine to coarse SAND 3.90 - 3.95 m: 50 mm silty fine to medium SAND 5.00 - 5.10 m: 100 mm CORE LOSS 5.10 - 5.20 m: silty GRAVEL		M	MD
5.25 – 5.45	SILT, some clay to clayey, minor fine sand; brown with orange-brown mottles. Firm to stiff, moist, high plasticity		M	F-St
5.45 – 5.65	5.45 - 5.65 m: CORE LOSS (inferred)			
5.65 – 6	Silty sandy GRAVEL; brown and light brown. Loose to medium dense, saturated, well graded; sand is fine; gravel is fine, angular to subrounded quartz and schist		S	L-MD

Depth (m)	Method	TCR SCR RQD (%)	Sample Number / Type	Result
0 – 1	SNC	50		
1 – 1.45	SPT	91		SPT 1/1 0/1 1/1 N = 3
1.45 – 2	SNC	100		
2 – 2.45	SPT	58		SPT 6/8 7/7 7/5 N = 26
2.45 – 3	SNC	100	C 2.50 – 2.70	
3 – 3.45	SPT	67		SPT 4/3 3/2 3/3 N = 11
3.45 – 4	SNC	100		
4 – 4.45	SPT	38		SPT 3/4 3/2 2/4 N = 11
4.45 – 5	SNC	100		
5 – 5.45	SPT	78		SPT 4/5 4/3 4/4 N = 15
5.45 – 6	SNC	64		

Water level: ∇ 5-03-2020

RL (m): 17, 16, 15, 14, 13, 12
Depth (m): 0, 1, 2, 3, 4, 5, 6

Flush Return (%): 25 50 75
Estimated Strength (MPa): EW VW W MS S VS ES
Defect Spacing (mm): 20 60 200 600 2000

Notes and Comments:

End of Hole @ 10.45m, Target Depth.

Nested piezos - screens at 2.5 - 4.5 m and 6.1 - 7.1 m

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical
Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - adjacent to ablutions block
Job Number: 12519887
Commenced: 22/02/2020 Completed: 22/02/2020

Hole No. : BH02 (CE16/0109)
Sheet : 2 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404932.59 Northing: 799291.3075 System: TAIETM2000
RL: 17.3 Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
6.05	SILT, minor clay, trace to minor fine to medium sand; brown and grey. Stiff, moist, low plasticity	DUKES FORMATION	M	St
6.5	6.50 - 7.00 m: CORE LOSS inferred from minimal recovery: gravelly SILT, soft			
7	SILT, minor clay, trace to minor fine sand; brown and grey. Very stiff, moist, low plasticity		M	VSt
7.3	Gravelly SAND; brown. Dense, wet to saturated, poorly graded; gravel is fine; sand is coarse 8.55 - 8.60 m: light grey SAND (no gravel), high mica content - muscovite?		S	D
9.05	SILT, minor clay, trace to minor fine to medium sand; brown and grey. Stiff, moist, low plasticity 9.20 - 9.45 m: minor fine to medium sand, trace clay		M	St
9.45	9.45 - 9.63 m: CORE LOSS (inferred)			
9.63	SILT, minor clay, trace to minor fine to medium sand; brown and grey. Stiff, moist, low plasticity		M	St
10	10.00 - 10.16 m: CORE LOSS (inferred)			
10.16	Fine to coarse SAND, minor silt, trace fine gravel; brown. Dense, moist, poorly graded		M	D
	End of Hole @ 10.45m,Target Depth.			

Sample: C 7.50 - 7.70

Method	Result	TCR SCR RQD (%)
SPT	SPT 1/1 1/3 2/4 N = 10	98
SNC		9
SPT	SPT 1/2 4/11 12/14 N = 41	100
SNC		100
SPT	SPT 10/8 10/9 8/5 N = 32	58
SNC		100
SPT	SPT 1/1 2/3 5/5 N = 15	91
SNC		67
SPT	SPT 6/7 9/9 9/10 N = 37	64

Columns: RL (m), Depth (m), Graphic, Casing, Flush Return (%) 25 50 75, Weathering, Estimated Strength (MPa) EW VW W MS VS ES, Defect Spacing (mm) 20 60 200 600 2000, Instrumentation Installation, Water level

RL (m): 11, 10, 9, 8, 7, 6
Depth (m): 6, 7, 8, 9, 10, 11, 12

FINAL

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Nested piezos - screens at 2.5 - 4.5 m and 6.1 - 7.1 m

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)
05/03/20	09:50	3.6	10.45

CLIENTS | PEOPLE | PERFORMANCE

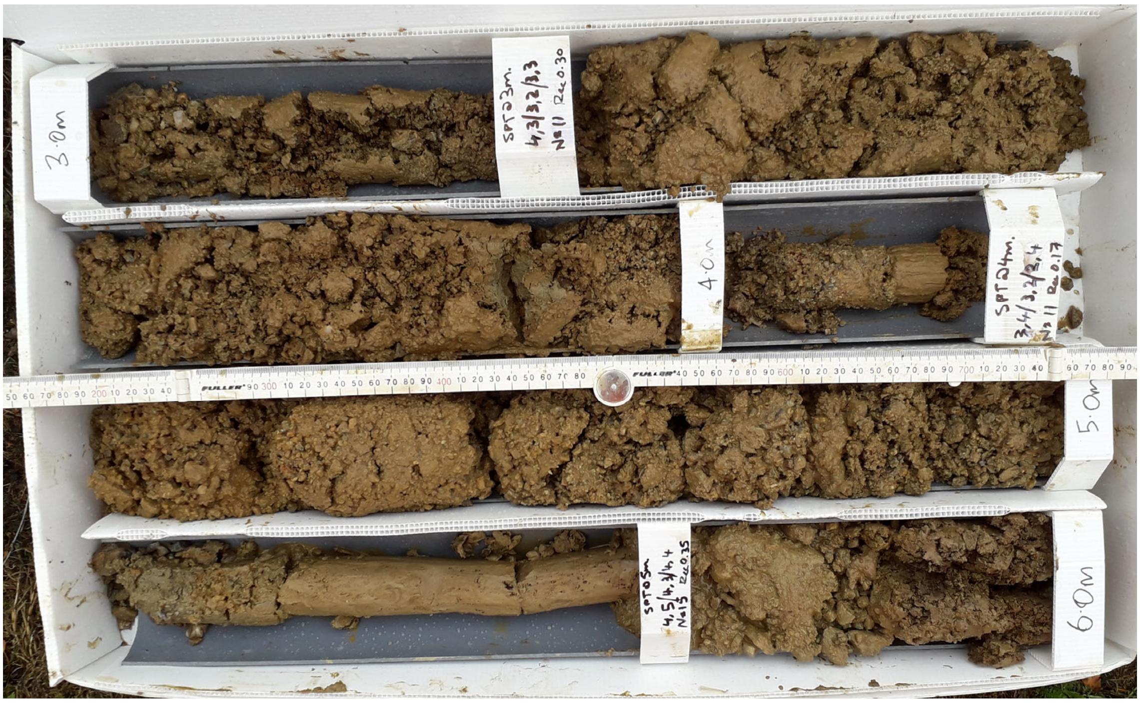
Report of Photographs

Site Identification: BH02

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	22/02/20	**Completed**	22/02/20
Site	Adjacent to ablutions block	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

(Not photographed)

0.0 m to 3.0 m



3.0 m to 6.0 m

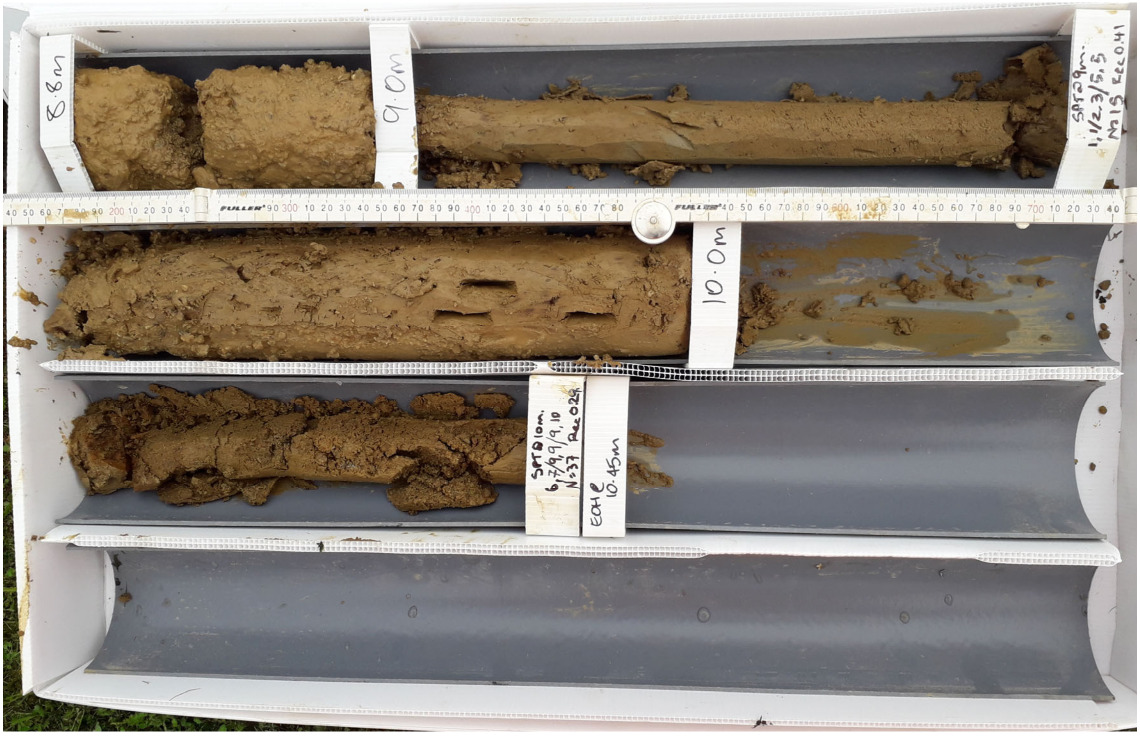
CLIENTS | PEOPLE | PERFORMANCE

Report of Photographs

Site Identification: BH02

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	22/02/20	**Completed**	22/02/20
Site	Adjacent to ablutions block	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

6.0 m to 8.8 m



8.8 m to 10.45 m (EOH)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - Memorial Park field
Job Number: 12519887
Commenced: 21/02/2020
Completed: 21/02/2020

Hole No. : BH03 (CE16/0118)

Sheet : 1 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404899.7401
Northing: 799227.8324
System: TAIETM2000
RL: 16.72
Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
0 – 0.2	SILT, minor fine to medium sand; grey-brown. Firm, moist, low plasticity; rootlets (TOPSOIL)	TS	M	F
0.2 – 1.1813	Fine to medium SAND, minor to some silt; brown with orange staining. Loose, moist, poorly graded (DUKES FORMATION) 0.70 - 1.13 m: grey and orange-brown	DUKES FORMATION	M	L
1.1813	Organic SILT; dark grey. Soft, moist to wet, high plasticity		M-W	S
– 1.9	SILT, minor clay, trace to minor fine to medium sand; grey with orange-brown streaks. Soft, moist to wet, high plasticity 1.75 - 1.90 m: gravelly (fine)		M-W	S
1.9 – 5.2	Sandy GRAVEL, some silt; grey and brown. Medium dense, wet to saturated, poorly graded; sand is coarse; gravel is fine, angular to subrounded quartz and schist 4.65 - 4.70 m: 50 mm SILT, minor clay, minor fine to medium sand; orange-brown. Soft, moist, high plasticity 4.70 - 4.90 m: 200 mm silty fine to coarse SAND; grey and brown. 'Loose', wet to saturated, poorly graded 4.90 - 5.20 m: medium dense		S M S	MD S 'L' MD
5.2 – 5.7	SILT, trace fine sand; orange-brown and grey. Firm, moist, low plasticity 5.50 - 5.60 m: 100 mm CORE LOSS (inferred) - likely pebbly gravel		M	F
5.7 – 5.95	Silty SAND; grey and brown. Medium dense, saturated, poorly graded; sand is fine to medium		S	MD

Sample

Number / Type	Result
J 0.02 – 0.20	
J 0.40 – 0.50	
J 0.80 – 1.00	
	SPT 0/0 0/0 0/0 N = 0
J 1.30 – 1.50	
	SPT 0/4 6/6 5/4 N = 21
	SPT 5/6 4/3 2/2 N = 11
	SPT 2/4 4/5 4/3 N = 16
	SPT 3/2 2/2 3/4 N = 11

Method	TCR SCR RQD (%)
SNC	100
SPT	100
SNC	100
SPT	60
SNC	100
SPT	67
SNC	100
SPT	76
SNC	100
SPT	76
SNC	84

Water level: 21-02-2020

RL (m): 16, 15, 14, 13, 12, 11
Depth (m): 0, 1, 2, 3, 4, 5, 6

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Groundwater measured 1 hour post-drilling

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical
Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - Memorial Park field
Job Number: 12519887
Commenced: 21/02/2020
Completed: 21/02/2020

Hole No. : BH03 (CE16/0118)
Sheet : 2 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404899.7401
Northing: 799227.8324
System: TAIETM2000
RL: 16.72
Datum: NZVD2016

RL (m)	Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density	Result
	6.0	SILT, trace to minor clay, trace fine sand; grey with orange-brown and black mottles. Very stiff, moist, low plasticity; minor organic inclusions, minor iron-stained inclusions *(continued from layer starting at 6.0m)* 6.40 - 7.00 m: inclusions gone; grey and orange-brown; hard	DUKES FORMATION	M	VSt H	SPT 2/2 4/4 6/7 N = 21
10	7	Gravelly SILT; brown, orange-brown and grey. Hard, moist, low plasticity; gravel is fine 7.30 - 7.50 m: sandy gravelly SILT; sand is fine to medium; gravel is fine; hard pan?		M	H	SPT 4/7 12/8 11/9 N = 40
	7.5	Fine to medium SAND, minor to some silt; brown and orange-brown. 'Loose to medium dense', moist, poorly graded		M	'L-MD'	
9	7.8	Gravelly SAND, minor to some silt; brown and orange-brown. Medium dense, moist, poorly graded; gravel is fine; sand is fine to medium 8.15 m: 30 mm diameter vesicular basalt clast, dark red-brown staining in soil around		M	MD	SPT 7/8 5/7 6/6 N = 24
8	8.5	SILT; brown. Firm to stiff, moist, low plasticity; occasional basaltic gravel clasts		M	F-St	
	9.05	Fine to medium SAND, minor to some silt; brown. Medium dense, moist, poorly graded 9.60 - 9.70 m: gravelly		M	MD	SPT 2/2 3/3 5/7 N = 18
7	9.85	Gravelly SAND, minor silt; brown. Medium dense, moist, poorly graded; gravel is fine; sand is fine 10.00 - 10.45 m: dense		M	MD D	SPT 6/10 11/10 10/10 N = 41
6	10.45	End of Hole @ 10.45m,Target Depth.				
5						

Method	TCR SCR RQD (%)
SPT	100
SNC	100
SPT	76
SNC	100
SPT	84
SNC	100
SPT	82
SNC	100
SPT	76

Sample: Number / Type; Casing; Flush Return (%) 25 50 75; Weathering EW HW MW SW FR; Estimated Strength (MPa) VW W MS S VS ES; Defect Spacing (mm) 20 60 200 600 2000; Instrumentation Installation; Water level

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Groundwater measured 1 hour post-drilling

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical
Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)
21/02/20	14:00	2.7	10.45

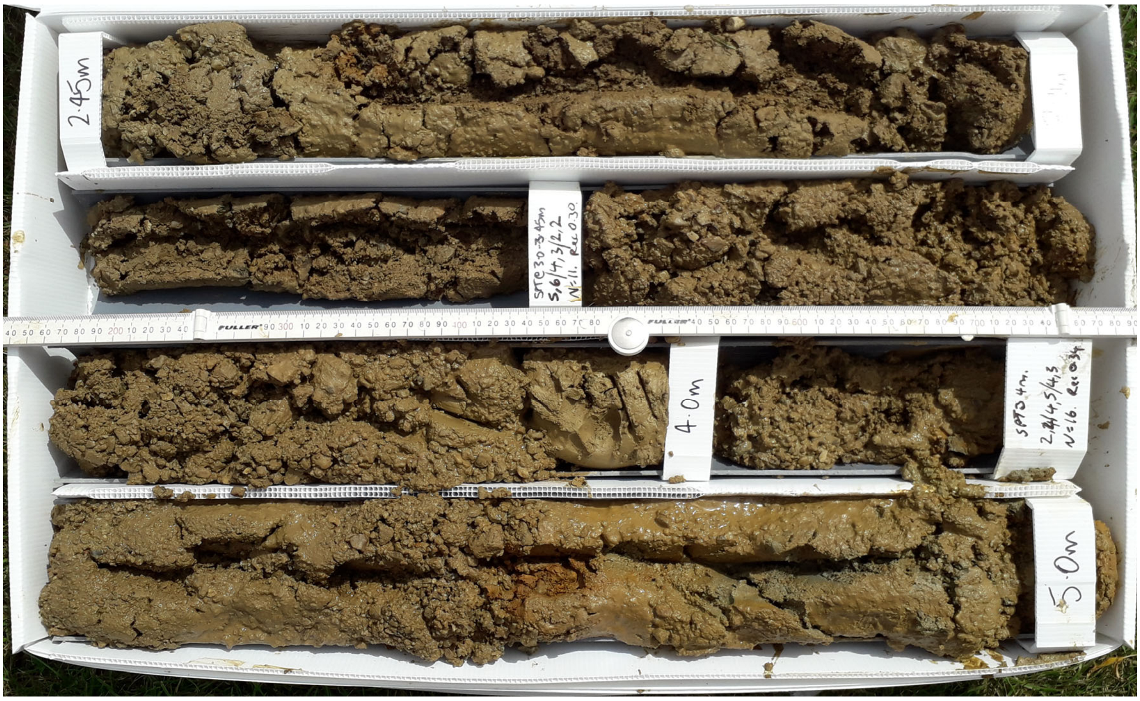
CLIENTS | PEOPLE | PERFORMANCE

Report of Photographs

Site Identification: BH03

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	21/02/20	**Completed**	21/02/20
Site	Memorial Park field	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

0.0 m to 2.45 m



2.45 m to 5.0 m

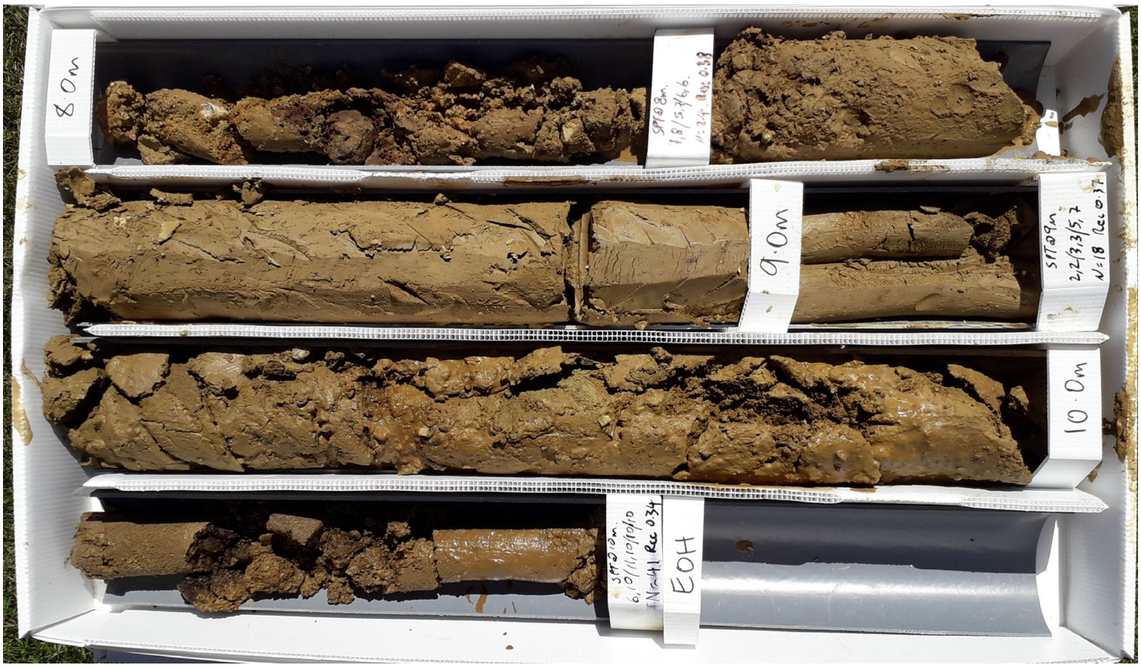
CLIENTS | PEOPLE | PERFORMANCE

Report of Photographs

Site Identification: BH03

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	21/02/20	**Completed**	21/02/20
Site	Memorial Park field	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

5.0 m to 8.0 m



8.0 m to 10.45 m (EOH)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - Memorial Park field
Job Number: 12519887
Commenced: 21/02/2020 Completed: 21/02/2020

Hole No. : BH04 (CE16/0119)

Sheet : 1 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404947.2618 Northing: 799248.0089 System: TAIETM2000
RL: 16.82 Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density
0 – 0.15	Fine sandy SILT; brown-grey. Firm, moist, low plasticity (TOPSOIL)	TS	M	F
0.15 – 1	Fine to medium SAND, minor to some silt; brown and orange-brown. 'Loose', moist, poorly graded (DUKES FORMATION)	DUKES FORMATION	M	'L'
1 – 1.5	20 mm red-brown SILT 50 mm dark grey SILT Sandy SILT; grey with orange-brown streaks. Soft, moist, low plasticity; sand is fine	DUKES FORMATION	M	S-F
1.5 – 3	Gravelly SAND; grey and brown. Medium dense, moist, poorly graded; gravel is fine; sand is medium to coarse 2.60 - 3.00 m: grades into sandy GRAVEL; sand is coarse; gravel is fine	DUKES FORMATION	M	MD
3 – 3.3	Silty SAND; grey and orange-brown. Medium dense, wet to saturated, poorly graded; sand is fine to medium	DUKES FORMATION	S	MD
3.3 – 3.5	Sandy GRAVEL; brown. Medium dense, wet to saturated, poorly graded; sand is coarse; gravel is fine	DUKES FORMATION	S	MD
3.5 – 3.7	Silty SAND; grey and orange-brown. Loose, wet, poorly graded; sand is fine to medium	DUKES FORMATION	W	L
3.7 –	Sandy GRAVEL; brown. Medium dense, wet to saturated, poorly graded; sand is coarse; gravel is fine, angular to subrounded, quartz and schist 4.45 - 4.60 m: 150 mm gravelly SAND; gravel is fine; sand is coarse 5.20 - 5.40 m: silty fine SAND; loose 5.40 - 6.05 m: fine to medium gravel	DUKES FORMATION	S	MD L MD

Sample

Number / Type	Result
J 0.02 – 0.20	
J 0.40 – 0.50	
J 0.80 – 1.00	
J 1.30 – 1.50	
	SPT 0/0 0/1 0/1 N = 2
	SPT 5/6 7/6 7/7 N = 27
	SPT 1/2 4/5 4/3 N = 16
	SPT 5/6 6/4 2/2 N = 14
	SPT 4/4 3/2 0/0 N = 5

Method	TCR SCR RQD (%)
SNC	70
SPT	82
SNC	100
SPT	80
SNC	100
SPT	71
SNC	100
SPT	53
SNC	100
SPT	71
SNC	76

Casing, Flush Return (%) (25 50 75), Weathering, Estimated Strength (MPa) (EW VW W MS S VS ES), Defect Spacing (mm) (20 60 200 600 2000), Instrumentation Installation

Water level: 21-02-2020

RL (m): 16, 15, 14, 13, 12, 11
Depth (m): 1, 2, 3, 4, 5, 6

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Groundwater measured 1 hour post-drilling

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

Date	Time	Reading (mbgl)	Hole depth (mbgl)

Project : Mosgiel Aquatic Facility - Geotechnical Input
Client : Dunedin City Council
Site : Mosgiel Caravan Park - Memorial Park field
Job Number: 12519887
Commenced: 21/02/2020
Completed: 21/02/2020

Hole No. : BH04 (CE16/0119)
Sheet : 2 of 2
Hole Length : 10.45m
Scale @ A4 : 1:30
Logged : MF
Processed : MF
Checked : SW

Easting: 404947.2618
Northing: 799248.0089
System: TAIETM2000
RL: 16.82
Datum: NZVD2016

Depth (m)	Material Description	Geological Unit	Moisture condition	Consistancy / Relative density	Sample Result
6.05	Clayey SILT, trace to minor fine sand; yellow-brown. Very stiff, moist, high plasticity	DUKES FORMATION	M	VSt	SPT 1/3 3/6 7/8 N = 24
6.9	SIlty GRAVEL; light brown. Dense, moist, poorly graded; gravel is fine		M	D	SPT 9/12 12/9 9/8 N = 38
7.45	SILT, minor fine gravel; grey and brown. Very stiff, moist, low plasticity 7.55 - 7.70 m: no gravel		M	VSt	
7.7	Fine to medium SAND, minor fine gravel; grey and orange-brown. Dense, moist, poorly graded		M	D	
8	Silty sandy GRAVEL; brown. Dense, moist, poorly graded; sand is fine; gravel is fine, angular to subrounded quartz and schist		M	D	SPT 7/10 9/8 7/8 N = 32
8.75	SILT, minor clay, trace to minor fine sand; brown. Very stiff, moist, low plasticity		M	VSt	
9	Fine to medium SAND; brown. Dense, moist to wet, poorly graded		M-W	D	SPT 1/2 6/12 9/10 N = 37
9.25	Silty GRAVEL; brown. Dense, wet, poorly graded; gravel is fine to medium, angular to subrounded, quartz and schist 9.55 - 10.45 m: grades into sandy, silty, fine to medium GRAVEL		W	D	SPT 8/11 12/12 11/12 N = 47

End of Hole @ 10.45m,Target Depth.

Method	TCR SCR RQD (%)
SPT	96
SNC	100
SPT	71
SNC	100
SPT	64
SNC	100
SPT	89
SNC	100
SPT	80

Notes and Comments:
End of Hole @ 10.45m, Target Depth.
Groundwater measured 1 hour post-drilling

Refer to explanation sheets for abbreviation and symbols

Inclination: Vertical
Orientation:
Contractor: Speight Drilling Limited
Equipment: Sonic Rig
Shear Vane Id:

Ground Water Level

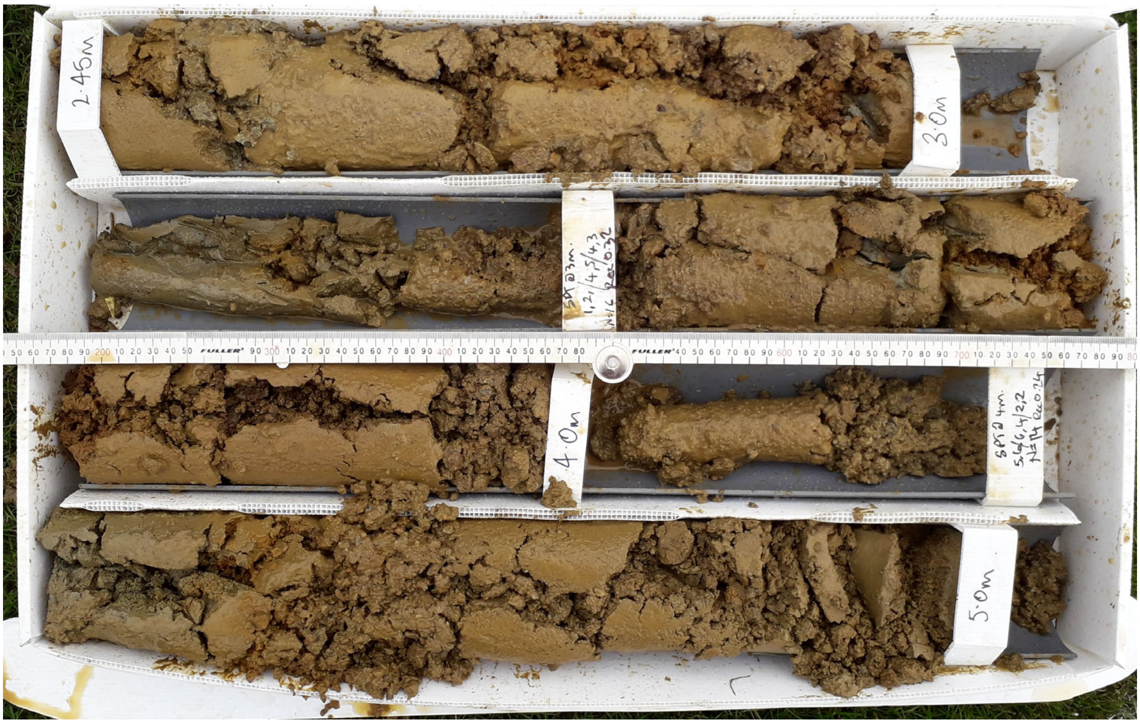
Date	Time	Reading (mbgl)	Hole depth (mbgl)
21/02/20	17:30	2.68	10.45

Report of Photographs

Site Identification: BH04

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	21/02/20	**Completed**	21/02/20
Site	Memorial Park field	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		

0.0 m to 2.45 m

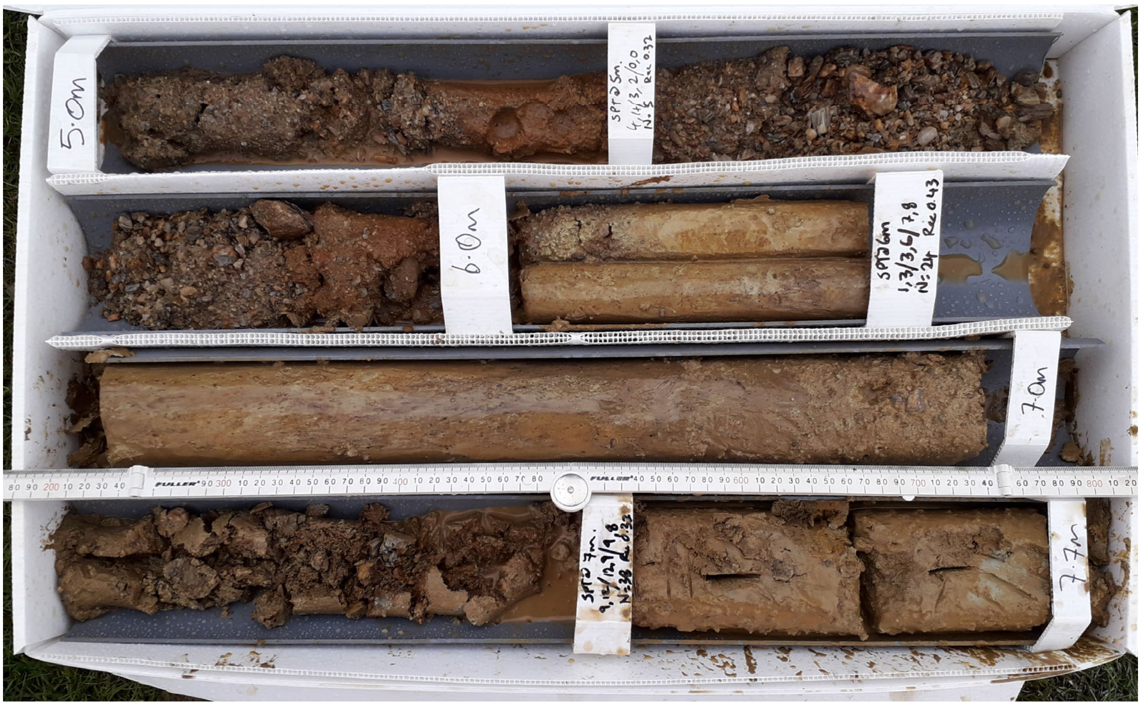


2.45 m to 5.0 m

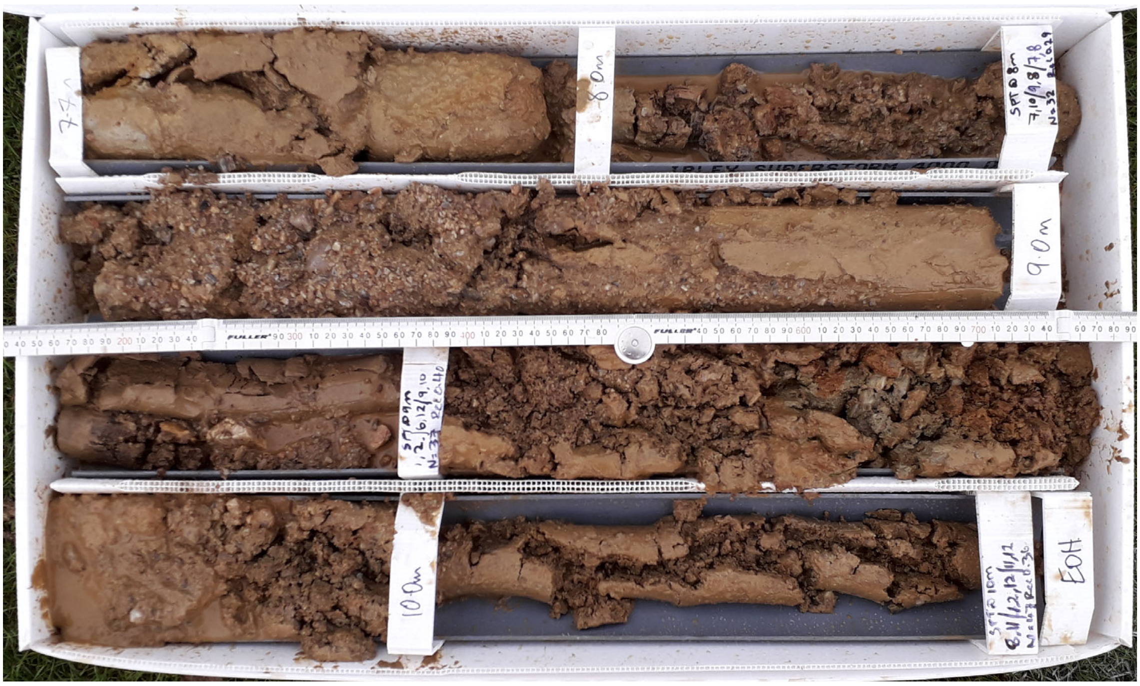
Report of Photographs

Site Identification: BH04

Project	Mosgiel Aquatic Facility – Geotechnical Input	**Commenced**	21/02/20	**Completed**	21/02/20
Site	Memorial Park field	**Logged By**	MF		
Job #	12519887	**Checked By**	SW		
Client	Dunedin City Council	**Hole Depth**	10.45 m		



5.0 m to 7.7 m

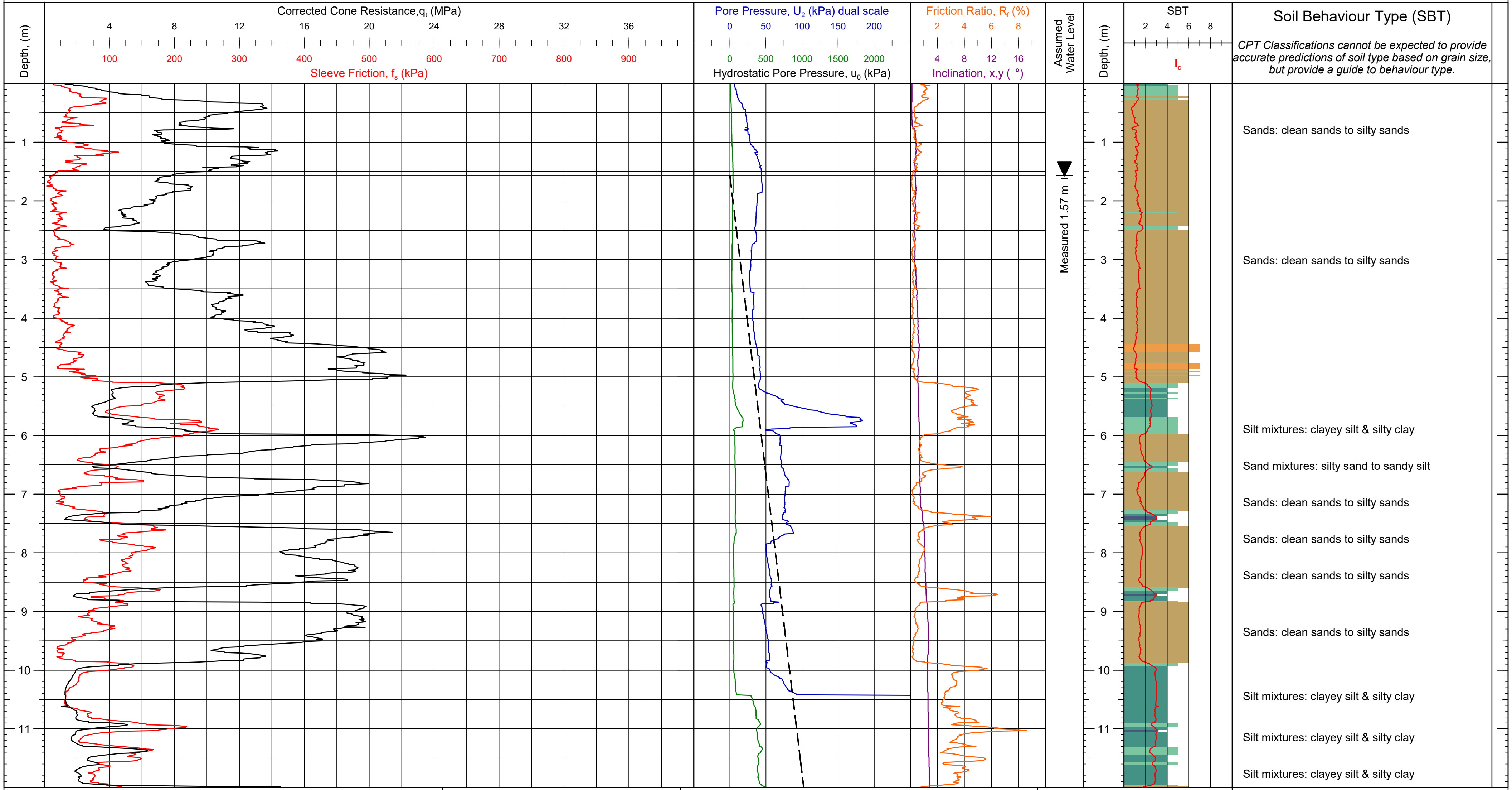


7.7 m to 10.45 m (EOH)

Tonkin+Taylor

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: Housing New Zealand Investigation
Location: 2 Murray & 29-31 Oban Streets, Fairfield, Mosgiel
Engineer: Danny Beasant
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: 10cm2 Compression
Area Ratio: 0.8
Filter Type: u2

NZTM 2000 N, E (m): 4917066.56, 1394310.28
WGS84 (deg): -45.869227,170.349828
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target depth

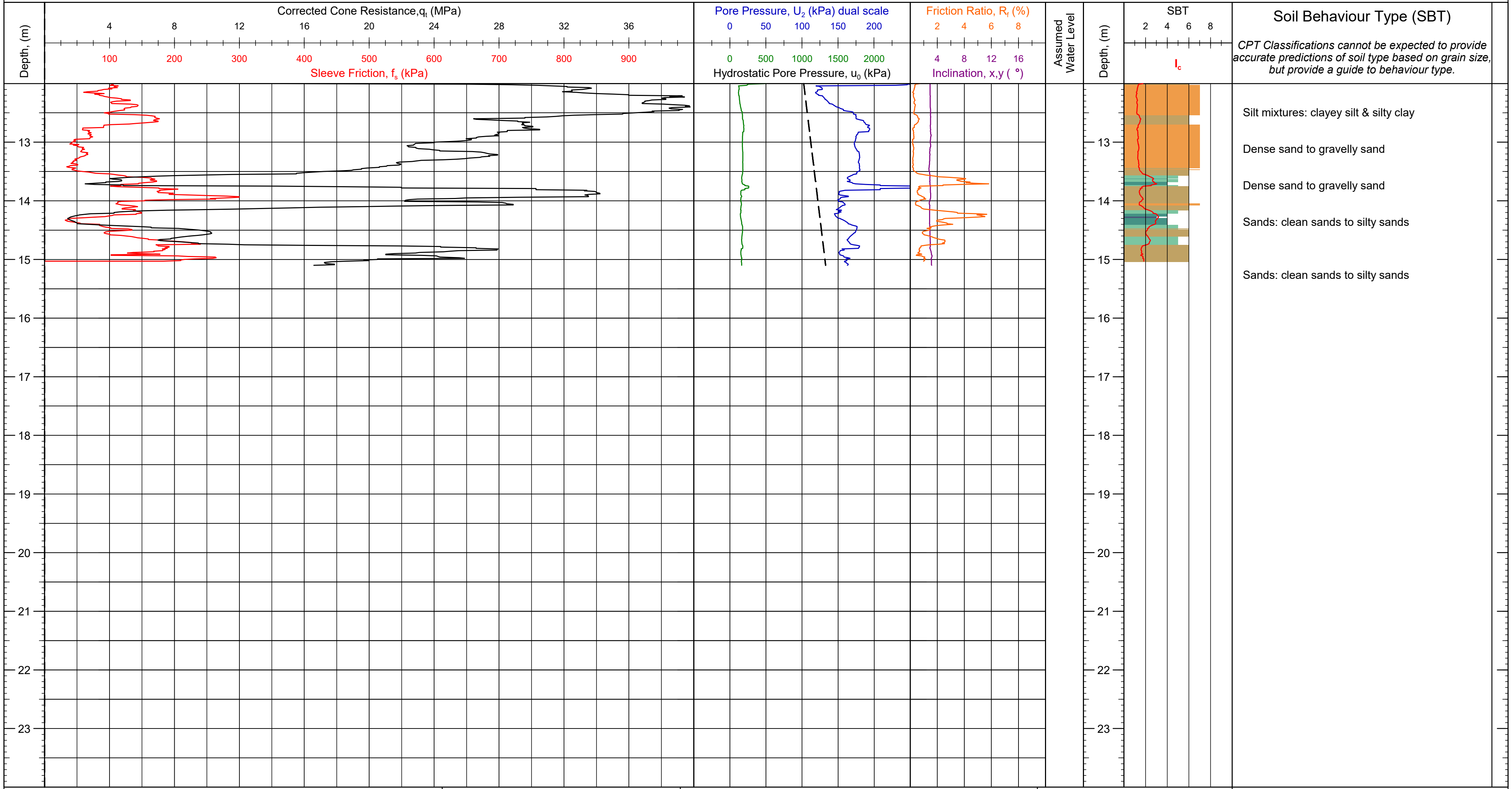
Elevation (m): Unknown
Date of Test: 17/12/2019
Depth (m): 15.10
Pre Drill (m): N/A

Client Job Ref:
Test Number: CPT-01
G.I. Job Ref: 190972

Comments:

NZGD ID: 136020

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: Housing New Zealand Investigation
Location: 2 Murray & 29-31 Oban Streets, Fairfield, Mosgiel
Engineer: Danny Beasant
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: 10cm2 Compression
Area Ratio: 0.8
Filter Type: u2

NZTM 2000 N, E (m): 4917066.56, 1394310.28
WGS84 (deg): -45.869227,170.349828
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target depth

Elevation (m): Unknown
Date of Test: 17/12/2019
Depth (m): 15.10
Pre Drill (m): N/A

Client Job Ref:
Test Number: CPT-01
G.I. Job Ref: 190972

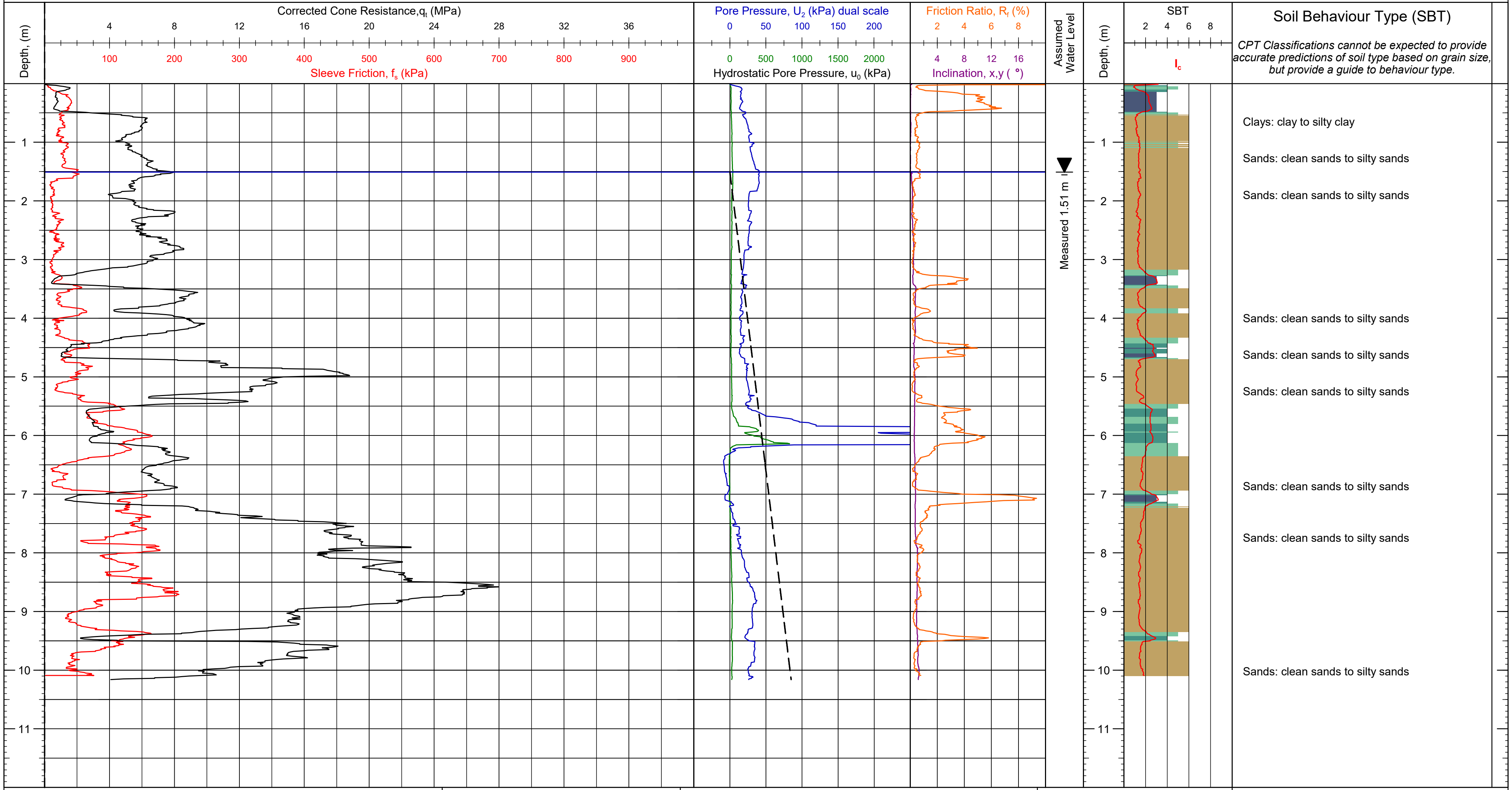
Comments:

NZGD ID: 136020

Tonkin+Taylor

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: Housing New Zealand Investigation
Location: 2 Murray & 29-31 Oban Streets, Fairfield, Mosgiel
Engineer: Danny Beasant
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: 10cm2 Compression
Area Ratio: 0.8
Filter Type: u2

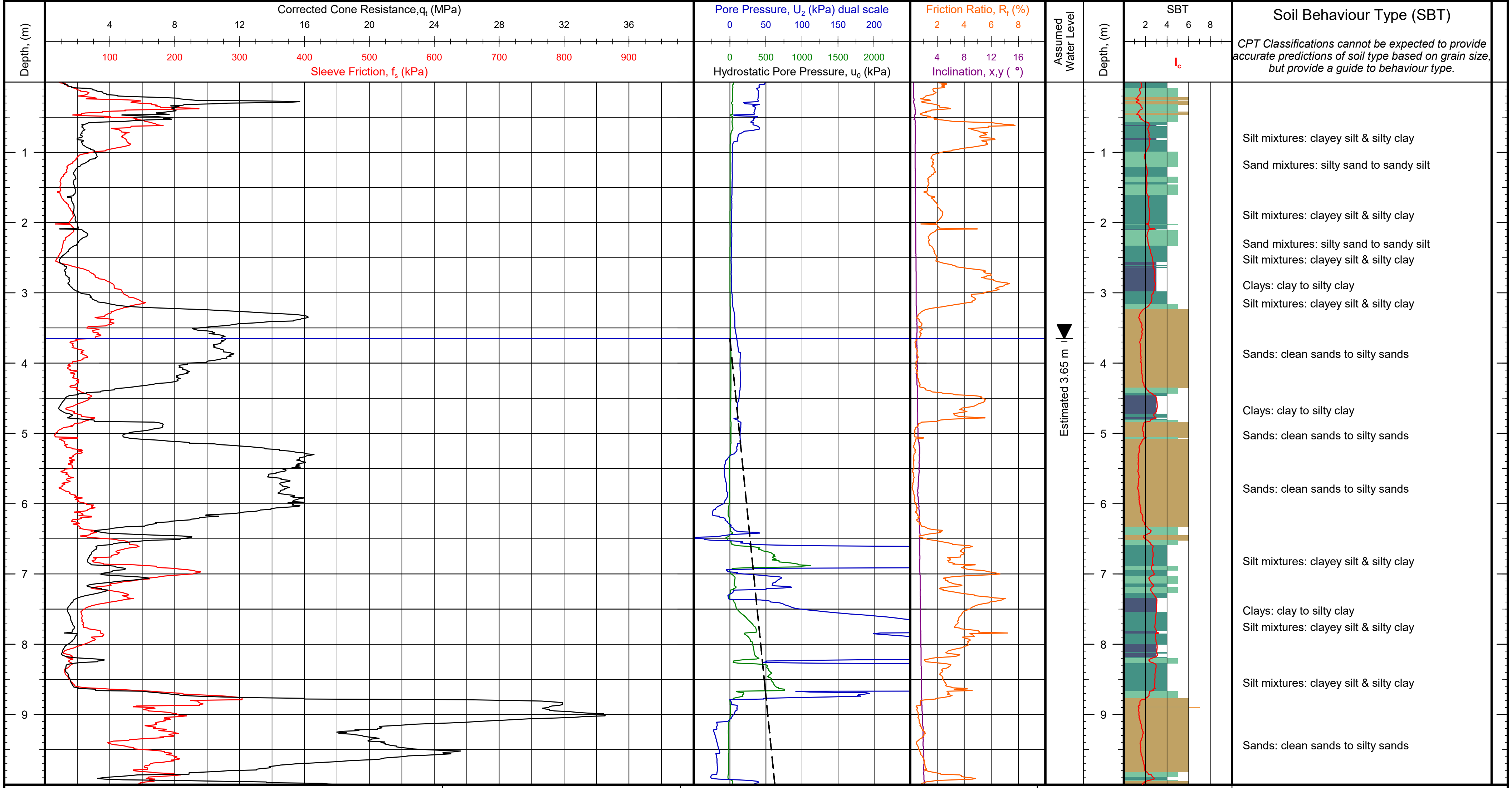
NZTM 2000 N, E (m): 4917015.93, 1394286.32
WGS84 (deg): -45.869675,170.349498
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target depth

Elevation (m): Unknown
Date of Test: 17/12/2019
Depth (m): 10.16
Pre Drill (m): N/A

Client Job Ref:
Test Number: CPT-02
G.I. Job Ref: 190972

Comments:

CONE PENETRATION TEST (CPT) LOG



Client: GHD Ltd.
Project: 215 Gordon Road
Location: Fairfield Mosgiel North Island
Engineer: Matthew Fitzmaurice
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: $10cm^2$ Compression
Area Ratio: 0.8
Filter Type: u_2

NZTM 2000 N, E (m): 4917247.17, 1394018.65
WGS84 (deg): -45.867516,170.346152
Location Method: Handheld GPS
Surveyor:

Elevation (m): Unknown
Date of Test: 19/02/2020
Depth (m): 15.11
Pre Drill (m): N/A

Termination Reason: Target Depth

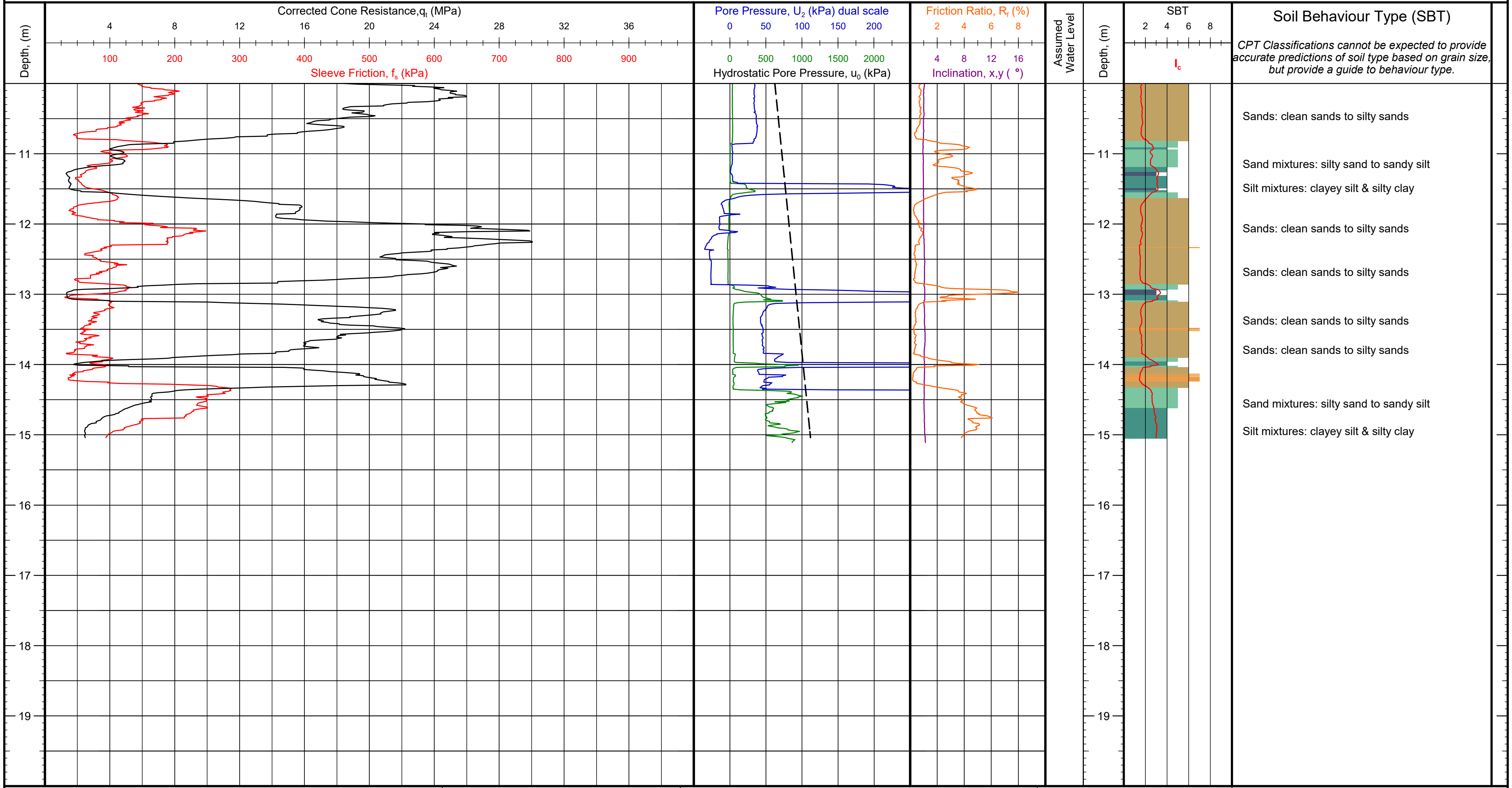
Client Job Ref:

Test Number: **CPT-02**

G.I. Job Ref: 190986

Comments:

CONE PENETRATION TEST (CPT) LOG



Client:	GHD Ltd.	**Operator:**	Brendon Lemm	**NZTM 2000 N, E (m):**	4917247.17, 1394018.65	**Elevation (m):**	Unknown
Project:	215 Gordon Road	**Cone Ref:**	MKJ208	**WGS84 (deg):**	-45.867516,170.346152	**Date of Test:**	19/02/2020
Location:	Fairfield Mosgiel North Island	**Cone Type:**	10cm^2 Compression	**Location Method:**	Handheld GPS	**Depth (m):**	15.11
Engineer:	Matthew Fitzmaurice	**Area Ratio:**	0.8	**Surveyor:**		**Pre Drill (m):**	N/A
Contractor:	Ground Investigation Ltd.	**Filter Type:**	u_2	**Termination Reason:**	Target Depth		

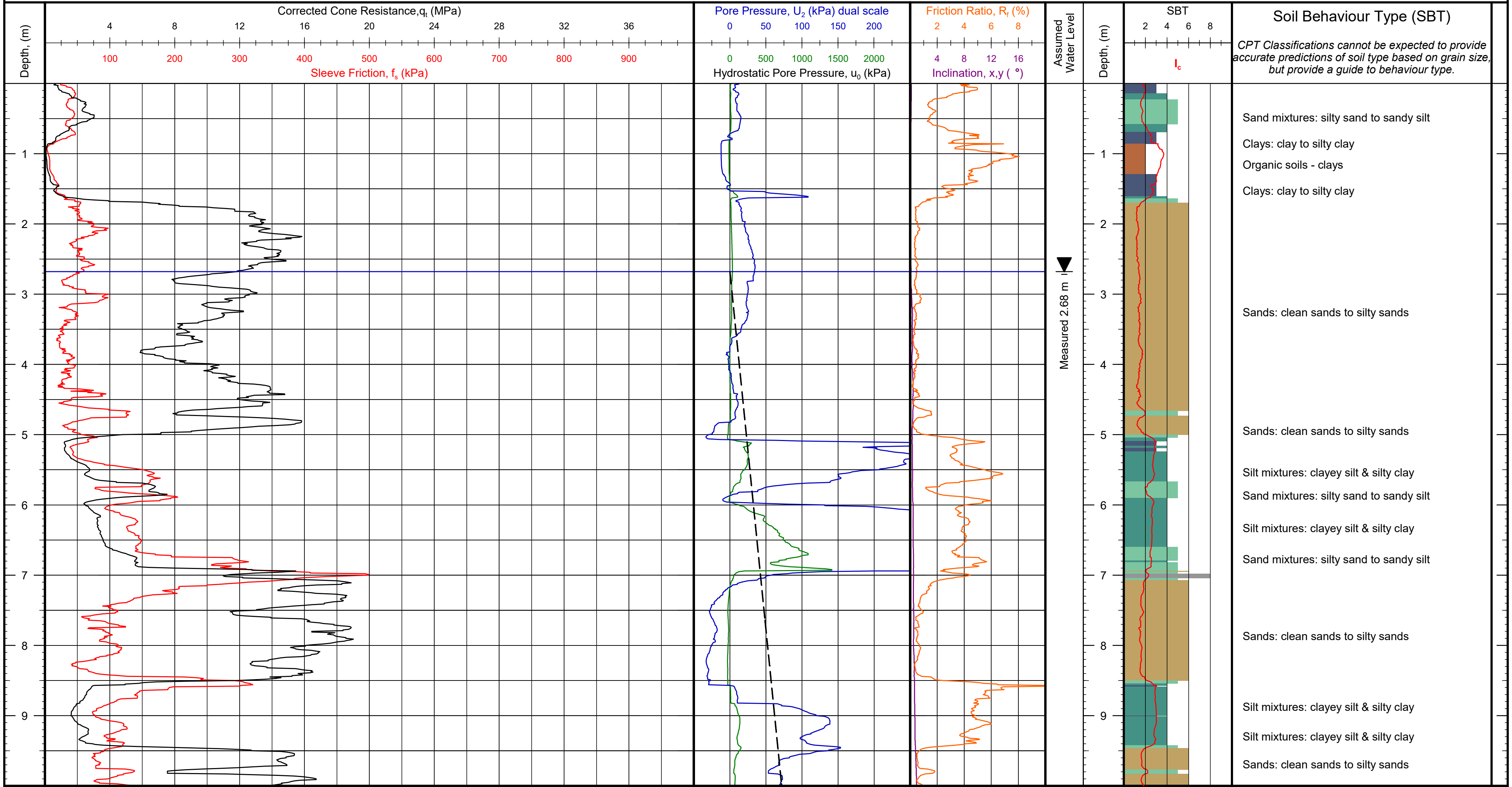
Client Job Ref:

Test Number: **CPT-02**

G.I. Job Ref: 190986

Comments:

CONE PENETRATION TEST (CPT) LOG



CPT Classifications cannot be expected to provide accurate predictions of soil type based on grain size, but provide a guide to behaviour type.

Client: GHD Ltd.
Project: 215 Gordon Road
Location: Fairfield Mosgiel North Island
Engineer: Matthew Fitzmaurice
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: $10cm^2$ Compression
Area Ratio: 0.8
Filter Type: u_2

NZTM 2000 N, E (m): 4917148.93, 1393977.71
WGS84 (deg): -45.868387,170.345583
Location Method: Handheld GPS
Surveyor:

Elevation (m): Unknown
Date of Test: 18/02/2020
Depth (m): 15.16
Pre Drill (m): N/A

Termination Reason: Target Depth

Client Job Ref:

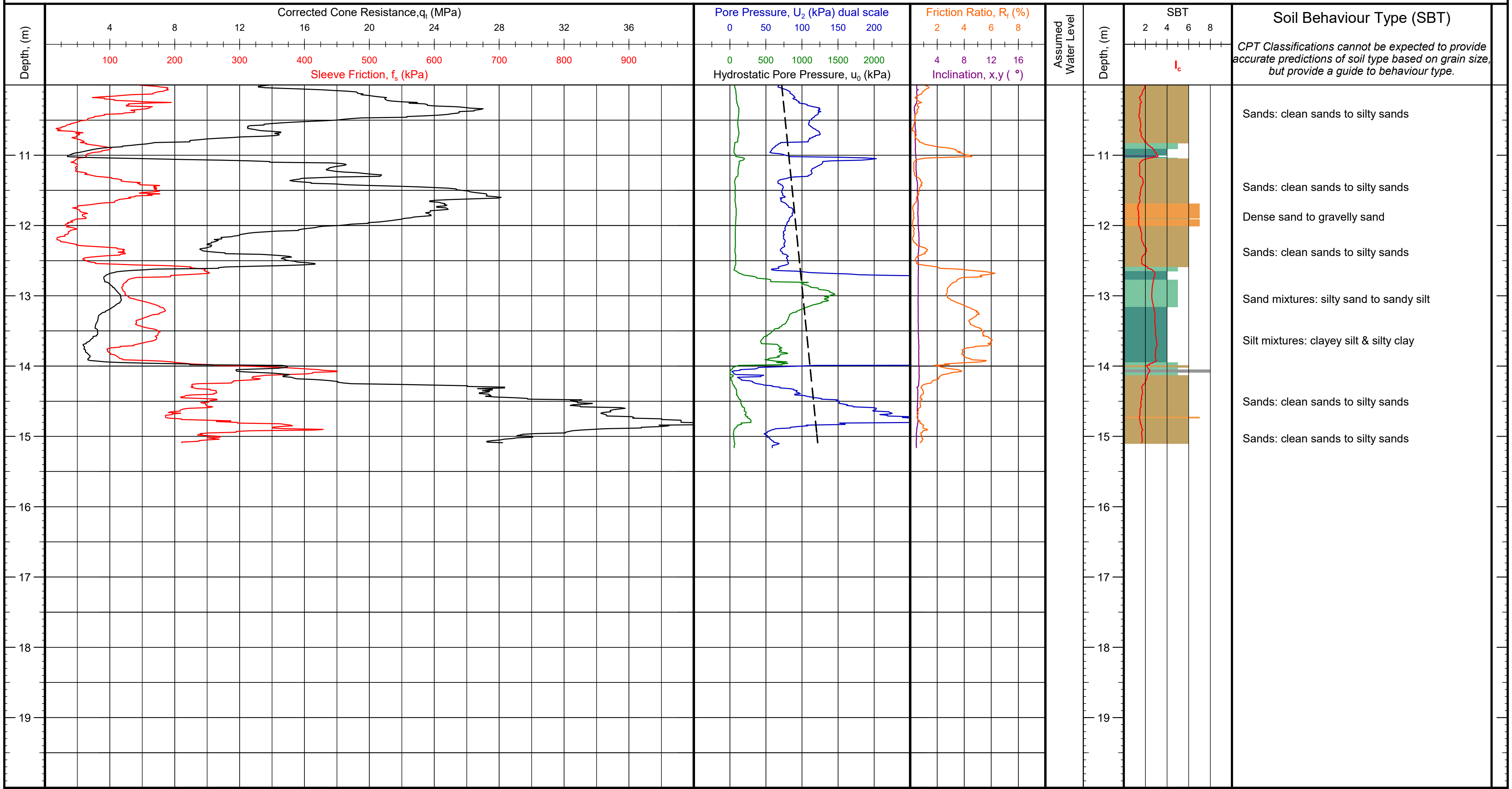
Test Number: **CPT-07**

G.I. Job Ref: 190986

Comments:

NZGD ID: 139876

CONE PENETRATION TEST (CPT) LOG



Client: GHD Ltd.
Project: 215 Gordon Road
Location: Fairfield Mosgiel North Island
Engineer: Matthew Fitzmaurice
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ208
Cone Type: $10cm^2$ Compression
Area Ratio: 0.8
Filter Type: u_2

NZTM 2000 N, E (m): 4917148.93, 1393977.71
WGS84 (deg): -45.868387,170.345583
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target Depth

Elevation (m): Unknown
Date of Test: 18/02/2020
Depth (m): 15.16
Pre Drill (m): N/A

Client Job Ref:

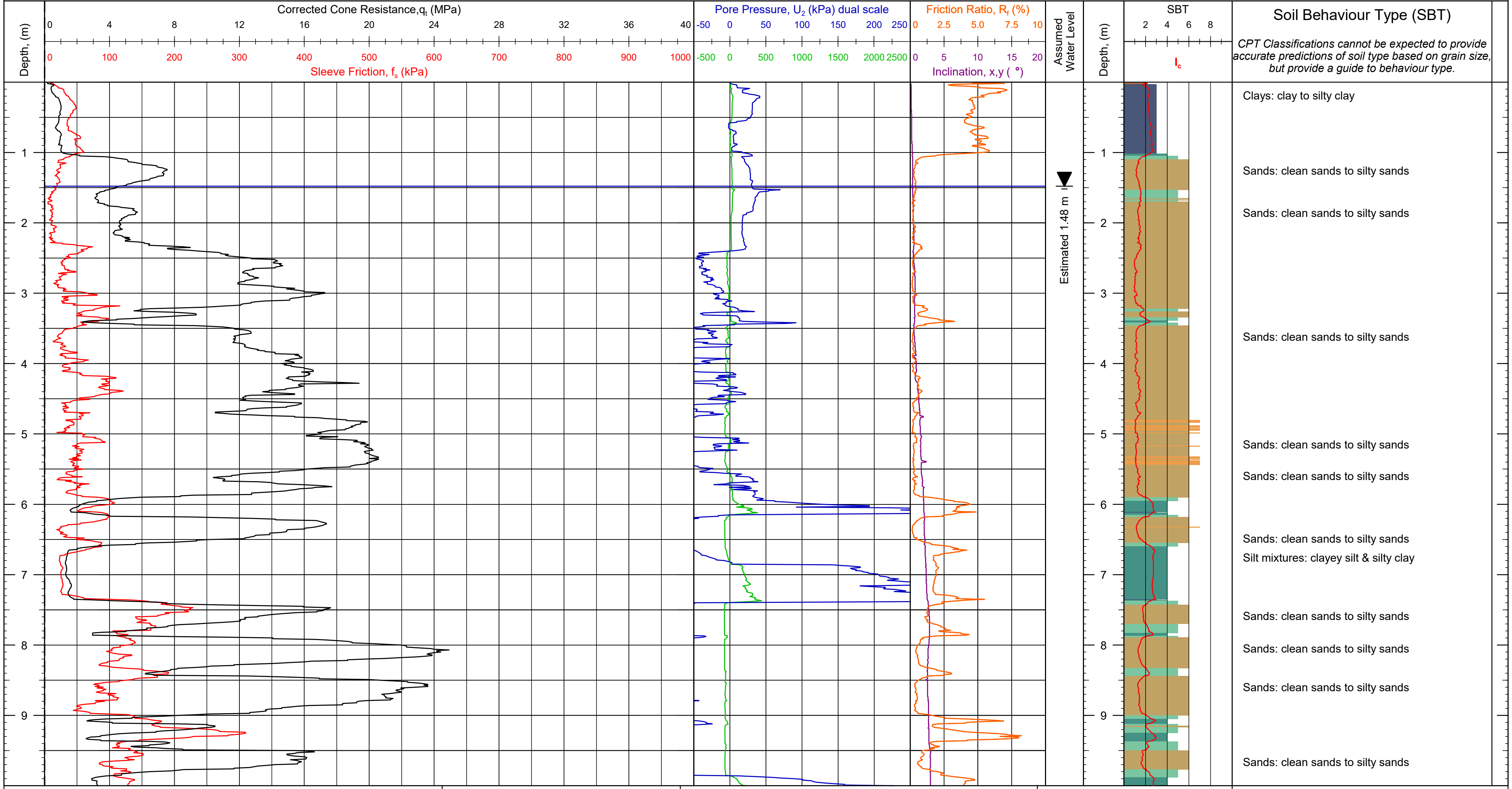
Test Number: **CPT-07**

G.I. Job Ref: 190986

Comments:

NZGD ID: 139876

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: 6 Stirling Cresent
Location: Fairfield, Mosgiel
Engineer: Danny Beasant
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ309
Cone Type: 10cm2 Compression
Area Ratio: 0.8
Filter Type: u2

NZTM 2000 N, E (m): 4916974.31, 1394640.76
WGS84 (deg): -45.870155,170.354042
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target Depth

Elevation (m): Unknown
Date of Test: 11/11/2019
Depth (m): 15.13
Pre Drill (m): N/A

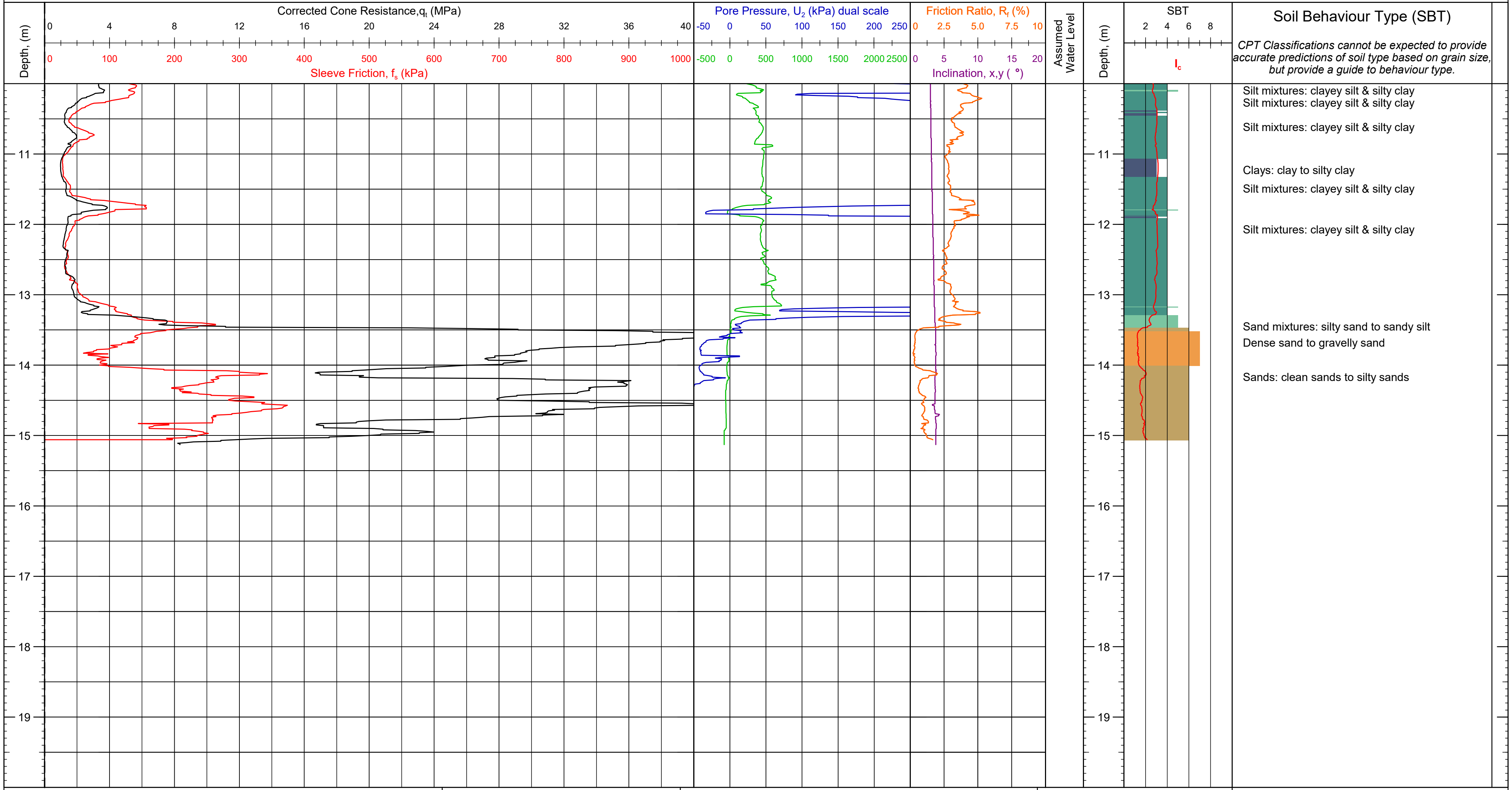
Client Job Ref:
Test Number: **CPT-02**
G.I. Job Ref: 190774

Comments:

Tonkin+Taylor

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: 6 Stirling Cresent
Location: Fairfield, Mosgiel
Engineer: Danny Beasant
Contractor: Ground Investigation Ltd.

Operator: Brendon Lemm
Cone Ref: MKJ309
Cone Type: 10cm2 Compression
Area Ratio: 0.8
Filter Type: u2

NZTM 2000 N, E (m): 4916974.31, 1394640.76
WGS84 (deg): -45.870155,170.354042
Location Method: Handheld GPS
Surveyor:

Elevation (m): Unknown
Date of Test: 11/11/2019
Depth (m): 15.13
Pre Drill (m): N/A

Termination Reason: Target Depth

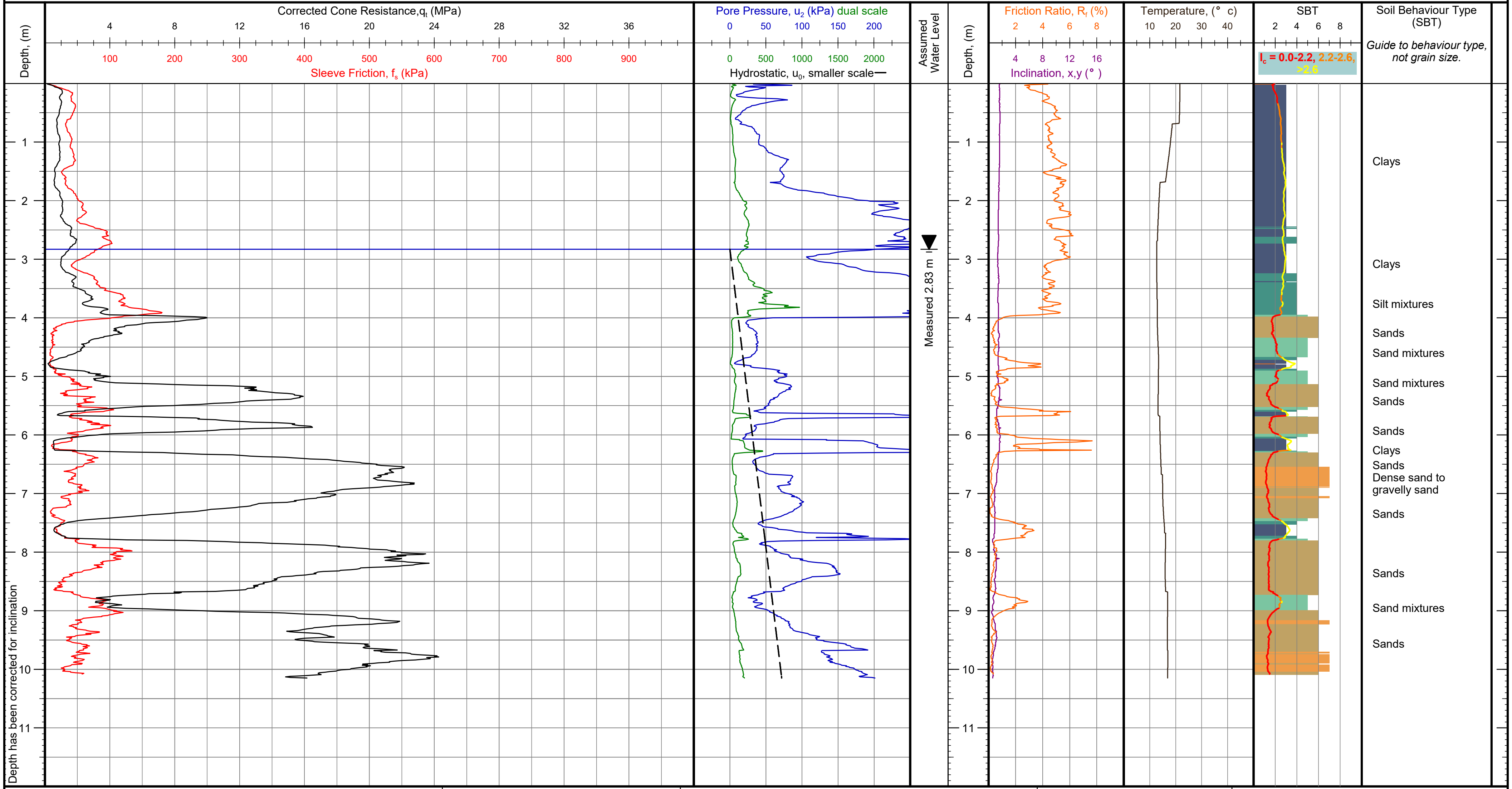
Client Job Ref:
Test Number: **CPT-02**
G.I. Job Ref: 190774

Comments:

GEOSOLVE
ENGINEERING CONSULTANTS

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: GeoSolve Ltd.
Project: Silverstream School, 52 Green Street
Location: Mosgiel
Engineer: Tim Plunket
Contractor: Ground Investigation Ltd

Operator: Brendon Lemm
Cone Ref: MKJ300
Cone Type: 10cm^2 Compression
Area Ratio: 0.79
Filter Type: u_2

NZTM 2000 N, E (m): 4916584.80, 1395325.70
WGS84 (deg): -45.873859,170.362743
Location Method: Handheld GPS
Surveyor:
Termination Reason: Target depth

Elevation (m): Unknown
Date of Test: 4/10/2022
Depth (m): 10.15
Pre Drill (m): N/A

Client Reference:
Test Number: CPT-06
G.I. Job Ref: 220062

Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

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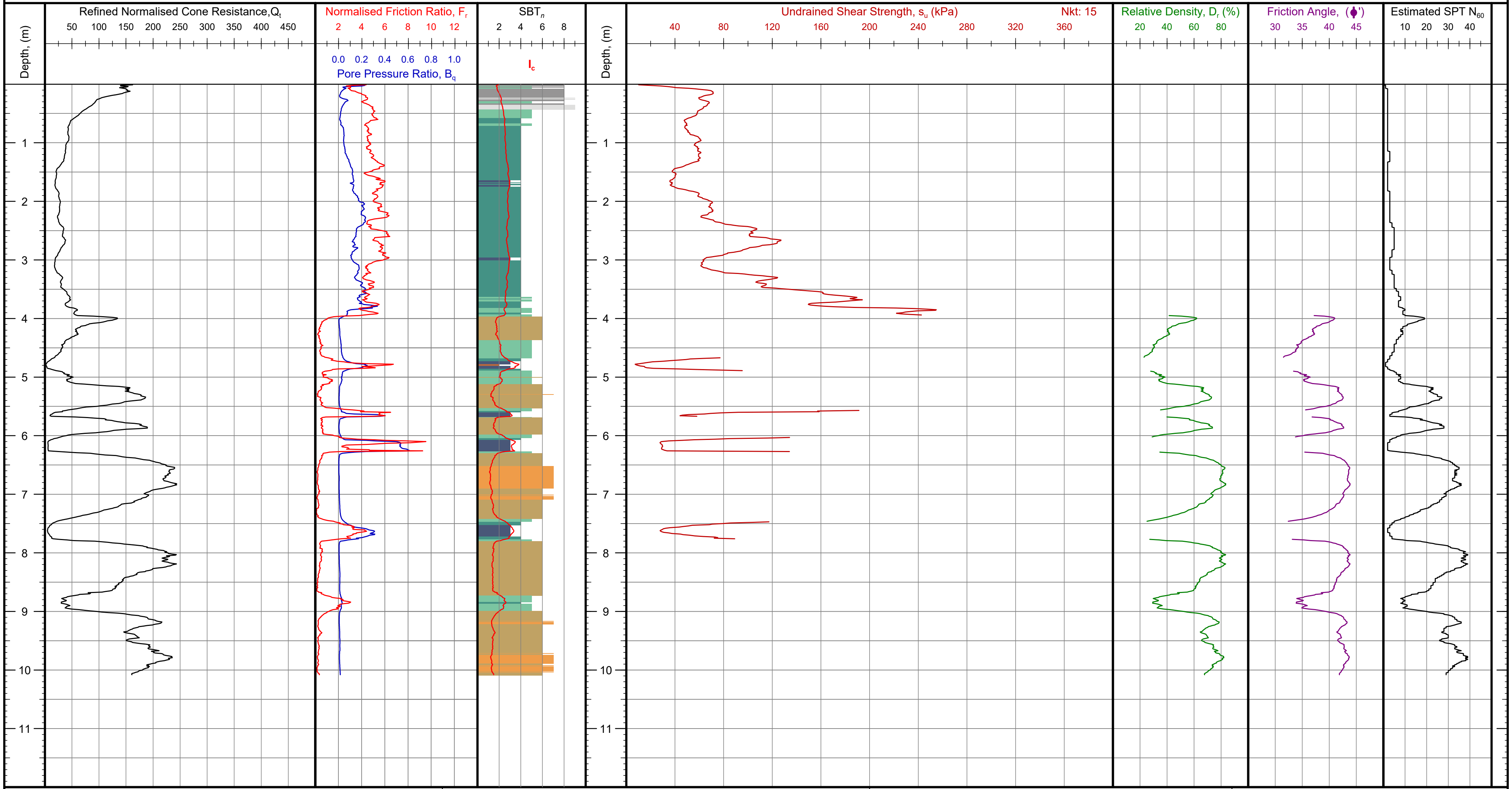
ENGINEERING CONSULTANTS

CPT PARAMETER LOG

GROUND INVESTIGATION



Client: GeoSolve Ltd.
Project: Silverstream School, 52 Green Street
Location: Mosgiel
Engineer: Tim Plunket
Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine grained	6	Sands: clean sands to silty sands
2	Organic: Organic clay/silt, peat	7	Dense sand to gravelly sand
3	Clay: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff silt/clay

Notes and Limitations:

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P.K. Robertson and K.L. Cabel (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed by the user. Ground Investigation Ltd. does not warrant the correctness or applicability of any of the geotechnical soil and design parameter shown and does not assume any liability for any use of the results in any design or review. The used should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Client Reference:

Test Number: CPT-06

G.I. Job Ref: 220062

HAND AUGER LOG

HOLE Id: **HA1 [6_Stirling]**

Hole Location: 6 Sterling Crescent

SHEET: 1 OF 1

PROJECT: Housing New Zealand
LOCATION: Mosgiel, Dunedin
JOB No.: 1011895.1000

CO-ORDINATES: (NZTM2000) 4916956.05 mN 1394622.61 mE
R.L.: 20.00m
DATUM: DUNEHT1958

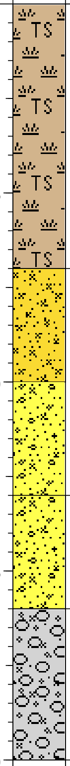
DRILL TYPE: Hand Auger
DRILL METHOD: HA

HOLE STARTED: 11/11/2019
HOLE FINISHED: 11/11/2019
DRILLED BY: T+T
LOGGED BY: NIMO
CHECKED: SWSU

GEOLOGICAL

ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MATERIAL COMPOSITION.	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/50mm) 0 1 2 3 4 5 6 7 8 9	TESTS	SAMPLES	RL (m)	DEPTH (m)	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	Description and Additional Observations
Topsoil		100	HA+DCP					0.0	D-M	F		0.0m: SILT, minor roots, trace ash; dark brown. Firm, dry to moist, non-plastic
					HA1 @ 0.50m			0.5				
Alluvial Deposits								0.7	M			0.7m: SILT, some sand; brown mottled orange brown. Firm, moist, low plasticity; sand; fine
							19	1.0		L		1.0m: Gravelly SAND, minor silt; grey. Loosely packed, moist; sand, fine to coarse; gravel, fine to medium
					HA1 @ 1.10m							
	11/11/2019 ▼							1.3	W			1.3m: Grades gravelly SAND, some silt; light brown, wet
					HA1 @ 1.60m			1.5				
								1.6	S			1.6m: Silty sandy GRAVEL; light brown. Loosely packed, saturated; sand, fine to coarse; gravel; fine to medium
		0	DCP				18	2.0				
								2.5				
								2.9				2.9m: Target depth
							17	3.0				
								3.5				



COMMENTS: Levels are in terms of Dunedin 1958. Estimated from https://www.topomap.co.nz/NZTopoMap/nz22123/DUNEDIN/

Hole Depth 2.9m

Scale 1:20

Tonkin+Taylor

HAND AUGER LOG

HOLE Id: **HA2 [6_Stirling]**

Hole Location: 6 Sterling Crescent

SHEET: 1 OF 1

PROJECT: Housing New Zealand
LOCATION: Mosgiel, Dunedin
JOB No.: 1011895.1000

CO-ORDINATES: (NZTM2000) 4916975.42 mN
1394652.35 mE
R.L.: 20.00m
DATUM: DUNEHT1958

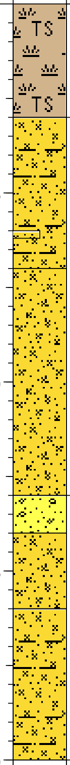
DRILL TYPE: Hand Auger
DRILL METHOD: HA

HOLE STARTED: 11/11/2019
HOLE FINISHED: 11/11/2019
DRILLED BY: T+T
LOGGED BY: DSA
CHECKED: SWSU

GEOLOGICAL

ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MATERIAL COMPOSITION.	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/50mm) 0 2 4 6 8 10 12 14 16 18	TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	Description and Additional Observations
Topsoil		100	HA+DCP					0.0		S	F		0.0m: SILT, minor sand and roots, trace ash; dark brown. Firm, saturated, low plasticity. 0.1m: Grades brown mottled orange brown and grey
Alluvial Deposits								0.3		W	F-St		0.3m: SILT, minor clay, trace sand; brown mottled orange brown and grey. Firm to stiff, wet, low plasticity 0.6 - 0.62m: Lense sandy SILT; grey, micaceous
					HA2 @ 0.70m	▭		0.7			St		0.7m: Sandy SILT; grey. Stiff, wet, non plastic; sand, fine to medium, micaceous 0.9 - 0.95m: Grades orange brown 1.1 - 1.15m: Lense sandy SILT, trace gravel; gravel, medium to coarse, sub-rounded, schist
							19	1.0					
	▼ 11/11/2019							1.3		S	L		1.3m: Gravelly SAND, minor silt; brown. Loosely packed, saturated; gravel, fine to medium; sand, fine to coarse
								1.4			F		1.4m: Sandy SILT interbedded with very thin to thin beds of silty fine SAND; brown. Firm, saturated, low plasticity; sand; fine
								1.6					1.6m: SILT, minor clay, minor sand; brown. Firm, saturated, low plasticity; sand, fine 1.9 - 1.92m: Lense sandy SILT; orange brown
		0	DCP				18	2.0					
							17	2.9					2.9m: Target depth



Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5

COMMENTS: Levels are in terms of Dunedin 1958. Estimated from https://www.topomap.co.nz/NZTopoMap/nz22123/DUNEDIN/

Hole Depth
2.9m

Scale 1:20

Rev.: A

NZGD ID: 153581

Tonkin+Taylor

HAND AUGER LOG

HOLE Id: **HA2 [Murray_Oban]**

Hole Location: 2 Murray St / 29-31 Oban St

SHEET: 1 OF 1

PROJECT: Housing New Zealand

LOCATION: Mosgiel, Dunedin

JOB No.: 1011895.1000

CO-ORDINATES: (NZTM2000) 4917058.05 mN
1394304.10 mE

R.L.: 17.00m

DATUM: DUNEHT1958

DRILL TYPE: Hand Auger

DRILL METHOD: HA

HOLE STARTED: 17/12/2019

HOLE FINISHED: 17/12/2019

DRILLED BY: T+T

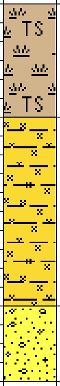
LOGGED BY: NIMO

CHECKED: DSA

GEOLOGICAL

ENGINEERING DESCRIPTION

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MATERIAL COMPOSITION.	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm) 0 1 2 3 4 5 6 7 8 9	TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	Description and Additional Observations
Topsoil									TS	D	S		0.0m: SILT, with trace rootlets; dark brown. Soft; non-plastic; dry.
Alluvial Deposits								0.5		D-M	St		0.3m: Clayey SILT, with trace rootlets; dark grey. Stiff; low plasticity; dry to moist. 0.4m: Grades brown with mottled orange and grey. Moderate plasticity.
							16	1.0		M	D		0.80m: SAND, with some gravel; dark grey. Tightly packed, moist; sand, fine; gravel, fine, angular.
													1m: Refusal
								1.5					
							15	2.0					
								2.5					
							14	3.0					
								3.5					



COMMENTS: Levels are in terms of Dunedin 1958. Estimated from Beca Topographical Survey (refer Beca Group Ltd 19/10/2018. 2 Murray St & 29-31 Oban St, Mosgiel. Drawing Ref: 7614900-402-WS-05. Rev A). Original Levels in terms of Otago Drainage Datum (Dunedin 1958 +100m)

Hole Depth 1m

Scale 1:20

Rev.: A

HandAugerLog - 19/01/2021 10:04:28 AM - Produced with Core-GS by GeRoc

NZGD ID: 153581

Tonkin+Taylor

HAND AUGER LOG

HOLE Id: **HA5 [Murray_Oban]**

Hole Location: 2 Murray St / 29-31 Oban St

SHEET: 1 OF 1

PROJECT: Housing New Zealand
LOCATION: Mosgiel, Dunedin
JOB No.: 1011895.1000

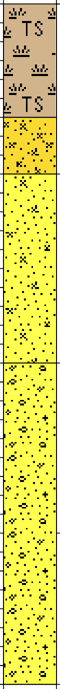
CO-ORDINATES: (NZTM2000) 4917008.64 mN 1394292.71 mE
R.L.: 17.00m
DATUM: DUNEHT1958

DRILL TYPE: Hand Auger
DRILL METHOD: HA

HOLE STARTED: 17/12/2019
HOLE FINISHED: 17/12/2019
DRILLED BY: T+T
LOGGED BY: NIMO
CHECKED: DSA

GEOLOGICAL

ENGINEERING DESCRIPTION



GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MATERIAL COMPOSITION.	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm) 0 1 2 3 4 5 6 7 8 9	TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	Description and Additional Observations
Topsoil									TS	M	S		0.0m: SILT, with some rootlets; dark brown. Soft; moderate plasticity; moist.
Alluvial Deposits											St		0.3m: SILT, with some sand, with trace gravel; dark grey. Stiff; high plasticity; moist; sand, fine; gravel, fine.
								0.5		M-W	D		0.45m: SAND, with some silt, with trace gravel; dark grey mottled orange. Tightly packed; moist to wet; sand, fine; gravel, fine.
											L		
							16	1.0		W	D		0.95m: Gravelly SAND; brown. Tightly packed; wet; sand, fine to coarse; gravel, fine to medium.
	17/12/2019 ▼									S			
								1.5					
													1.8m: Refusal
							15	2.0					
								2.5					
							14	3.0					
								3.5					

COMMENTS: Levels are in terms of Dunedin 1958. Estimated from Beca Topographical Survey (refer Beca Group Ltd 19/10/2018. 2 Murray St & 29-31 Oban St, Mosgiel. Drawing Ref: 7614900-402-WS-05. Rev A). Original Levels in terms of Otago Drainage Datum (Dunedin 1958 +100m)

Hole Depth
1.8m

Scale 1:20

Rev.: A

Appendix C Independent Reviewer Curriculum Vitae

Ioannis Antonopoulos

Geo-Professionals Discipline Lead, Technical Specialist Earthquake Engineering

Ioannis is a Chartered Geotechnical Engineer specialising in large infrastructure and development projects. His expertise include working within interdisciplinary teams in both design and construction, with a particular focus on geotechnical earthquake engineering design, water reservoirs, roading, ports, seawalls, surface and deep foundations, cut-and-cover structures, tunnelling, slope stability, hydrogeology, and water resource management. Ioannis key areas of expertise revolve around earthquake geotechnical engineering, soil-foundation-structure interaction (SFSI), numerical analysis and modelling, retaining structures, geotechnical design of soft soils, and the characterisation of geotechnical material properties. He places strong emphasis on technical proficiency and maintaining a client-focused approach. By adopting a collaborative mindset, Ioannis strives to deliver focused, pragmatic, and value-driven engineering solutions. He is committed to implementing innovative design strategies that align with clients' expectations and project objectives. To achieve this, he incorporates state-of-the-art research-based methods and effectively manage risks throughout the project lifecycle.

EDUCATION AND TRAINING

MSc DIC Engineering Geology, Imperial College, University of London, 1992

BSc, National & Kapodistrian University, Athens, 1990

Project Management Training - Level 1, Tetra Tech, 2018

Advanced Seismic Training, Itasca, Christchurch 2017

Knowledge Assessment, Engineering New Zealand, Equivalence to a Washington Accord qualification, 2015

Height Safety, Safety'n Action, Dunedin, 2014

BLS/AED Provider Courseq, European Resuscitation Council, Athens, 2008

Computational Geotechnics for Experienced Users, Plaxis, Delft, 2002

AutoCAD Map R5 update, Infostudio, Athens, 2003

AutoCAD 2002 update Level I, Infostudio, Athens, 2003

Plaxis Course, Plaxis, Istanbul, 2001

PROJECT EXPERIENCE

Lyttelton Port of Christchurch Te Awaparahi Bay Stage 1 Wharf | Tetra Tech Coffey | Technical Advisor | 2021-2022
Ioannis provided technical advice for geotechnical and earthquake geotechnical engineering design, including, reviewing Proof of Concept and Preliminary Design geotechnical inputs, setting up programming, bespoke coding, and reviewing numerical models and outputs, participating in meetings with the design team and engaging in general discussions with the design team to assist with design optimisation.

Water Urban Flooding Risk Reduction
Gore and Wyndham Flood Protection | Environment Southland | Geotechnical Lead | 2021-2022
Ioannis role involved Geotechnical design for the upgrade of existing levees (stopbanks) and the design and construction of new ones to accommodate climate change and sea level rise. Responsibilities included planning geotechnical site investigations, such as geophysical MASW surveys, test pits, and laboratory testing.

Port Nelson Seismic Resilience Study | Port Nelson Limited | Project Technical Lead/Designer | 2020-2021
Ioannis completed assessment of the seismic performance of various assets of Port Nelson Limited and the soil-structure interaction effects under OLE and CLE shaking conditions. The behaviour of soil materials under shaking is examined through 2D non-linear dynamic analyses (NDA) using the two-dimensional finite difference software FLAC (Itasca Consulting Group, Inc., 2019). These simulations allow for the development of liquefaction and strain-softening using dedicated plasticity constitutive models. The NDA are conducted over pertinent cross sections that include wharves and buildings with their appropriate structural elements. These sections were selected in collaboration with PNL to capture a wide range of assets and performances, providing a benchmark to aid future design, resilience assessments, and

construction planning. The numerical simulations are carried out using 2D NDA according to ASCE/COPRI 61-14 (Seismic Design of Piers and Wharves, 2014), section C4.5.5. To support this study, a series of additional work was carried out, comprising of a seismic hazard assessment to provide interim seismic demands based on the current knowledge of the seismicity and vulnerability of the area, and to provide recommendations for design and further work, a detailed site investigation program comprising CPTu, boreholes, seismic CPTu, laboratory testing, and MASW geophysical surveys, along with factual and interpretive geotechnical reports and seismic site characterisation.

Water Distribution and Transmission Pipelines | Wellington Water | Soil Mechanics and Geotechnical Earthquake Engineering Lead | 2019-2020
The purpose of the Cross-Harbour Pipeline (CHP) project is to improve the disaster and operational resilience of Wellington City's water supply by providing an alternative pipeline route that does not cross the Wellington Fault. The project consists of a new bulk water supply pipeline that connects the Waterloo Water Treatment Plant (WTP) in Petone to the Carmichael Reservoir in Wellington City via a Cross-Harbour Pipeline (CHP). Ioannis led the planning of laboratory tests on undisturbed samples extracted from marine boreholes and assessed the results. He also led the interpretation and cross-correlation of existing and new marine site investigations, which included boreholes, CPTs, down-the-hole shear vanes, and lab results. This work resulted in a consistent ground model across the investigated alignment, populated with material properties to assist the next phase of the design. Additionally, he led the geotechnical earthquake engineering aspects of the project by providing the design methodology, pertinent material properties, and reviewing the PSHA.

Main Collecting Sewer Duplication Pipeline | Wellington Water | Designer | 2019-2020
Ioannis served as the Designer for the assessment of ground conditions along the pipeline and the dewatering-induced settlements. His responsibilities included, planning, managing, and participating in the assessment of the site investigation, which comprised boreholes, CPTs, and laboratory testing, estimating material properties, assessing liquefaction potential, and conducting time history analysis of critical sections of the site using advanced plasticity constitutive models for liquefaction and strain softening and estimating dewatering-induced settlements and dewatering demands.

WP-WC-CP Whitmore Street to Bowen | Wellington Water | Technical Lead | 2019
Ioannis served as the Technical Lead for the assessment of ground conditions along the pipeline and the dewatering-induced settlements. His responsibilities included, planning, managing, and participating in the assessment of the site investigation, which comprised geophysical surveys and a borehole and reviewing the estimated material properties and the numerical analyses conducted.

Jeffreys Pump Station Suction Tank | Christchurch City Council | Technical Lead | 2019
Ioannis conducted a sophisticated liquefaction assessment using plasticity models that account for liquefaction, refining the understanding of earthquake shaking effects on the soil. This approach enabled the re-evaluation of the preferred foundation solution and the performance of existing and older structures by assessing the kinematic effects of the non-improved soil and the associated soil-structure interactions. A new optimized foundation solution was tested and proposed.

Kilbirnie Pump Station | Wellington Water | Designer | 2019
Ioannis was responsible for design of Deep Soil Mixing ground improvement to reduce earthquake induced liquefaction settlements by providing a minimum composite shear strength within the specified improved area. Various options for liquefaction mitigation and reduction of liquefaction-induced settlements were considered and evaluated using quasi-static modelling. The reduction of settlements option was selected as a cost-effective solution.

Stantec

Stantec is a global leader in sustainable engineering, architecture, and environmental consulting. The diverse perspectives of our partners and interested parties drive us to think beyond what's previously been done on critical issues like climate change, digital transformation, and future-proofing our cities and infrastructure. We innovate at the intersection of community, creativity, and client relationships to advance communities everywhere, so that together we can redefine what's possible.

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