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Project Number 19079.000.001

Preliminary Geotechnical Investigation

Metlifecare Whenuapai Village,
99 Totara Road, Whenuapai, Auckland

Submitted to:
Metlifecare Limited
20 Kent Street
Newmarket
Auckland 1023

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1 Introduction

ENGEO Limited was requested by Metlifecare Limited to undertake a preliminary geotechnical investigation of the property 99 Totara Road, Whenuapai, Auckland (herein referred to as 'the site'). The purpose of this work was to support your eligibility assessment for a fast-track application to develop a new retirement village on-site.

Based on our discussions, we understand that you intend to obtain a fast-track consent with the application being made in two stages; Stage 1 of the application is to demonstrate the project meets eligibility criteria, and Stage 2 comprises the full resource consent application. This report is intended to support Stage 1 of this application.

This report has been revised (revision 1) to include a review of updated development plans which were provided subsequent to the issue of our original report.

2 Site Description

The site at 99 Totara Road is an irregularly shaped, 9.71 Ha parcel, legally described as LOT 1 DP 170291. Auckland Council designates the land as Future Urban Zone.

Approximately 700 m of the northern, eastern and the northern portion of the western property boundaries are defined by the coastal margin within the Kotukutuku Inlet of the Waitemata Harbour. The southern portion of the western boundary borders 97 and 93 Totara Road (residential plots) and Totara Road itself. The southern boundary is shared entirely with 101 Totara Road.

Neighbouring site, 101 Totara Road is currently unoccupied and grassed, however we understand that a residential development is planned. The properties at 97 & 93 Totara Road appear to be lifestyle plots, largely grassed residential dwellings and associated garages / sheds.

The site (99 Totara Rd) is accessed via a private driveway directly off Totara Road, running along the site's southern boundary. The driveway connects to a group of deteriorating farm sheds and a single level brick clad dwelling. The site is currently unoccupied, however previously appears to have been subject to horticultural and agricultural activities.

Topographically the site is largely flat or undulating, except for the coastal margin which is defined by slopes and sea cliffs up to 10 m high, and several gully / stream and scarp features adjacent to the coastal margin. These are discussed in more detail in later sections.

A site aerial image showing key features is presented in Appendix 1.

3 Proposed Development

We have been provided with the RESET Urban Design Masterplan Layout for this development, dated 2 February 2023 (Appendix 2). These plans show that the proposed development will comprise large amenity and care buildings within the southern / central portion of the site, with the remainder of the buildings largely comprising single level villas / units.

An esplanade reserve is proposed along the coastal margin, including a park on the headland at the western-most edge of the site. An existing stream passing through the site near the eastern boundary is proposed to be revitalised, including a bridge which will connect proposed villas on the eastern side of the stream with the care and amenity buildings west of the stream.

At this stage earthworks plans are not available for the development. However, based on existing landform and development plans, we anticipate that earthworks will comprise moderate cut to fills primarily to moderate undulations across the site. However larger earthworks volumes are expected to backfill a gully in the eastern portion of site and to level scarp features in the western portion of the site. The later may require additional stabilisation measures.

4 Desktop Study

4.1 Geology

Published geological maps indicate the northern portion of the site is largely underlain by East Coast Bays Formation (ECBF) soils of the Waitemata Group. These deposits are evident along much of the eastern coastline of the Auckland area and were formed during the Early Miocene in submarine fan and basin floor depositional environments. These deposits typically consist thick bedded sandstone and interbedded laminated mudstones / siltstones. Residual soils formed by weathering and alteration of the parent sedimentary formation typically form silts and clays.

Alluvial deposits of the Puketoka Formation (Tauranga Group) are mapped in the southern portion of the site. Tauranga Group alluvial deposits comprise Pliocene aged or later deposits including silts, clays and sands with possible peat or organic rich clay soils and carbonaceous deposits. Accordingly, these deposits can be varied in their properties and may include soft and / or organic layers or deposits or soils that are sensitive to disturbance. These soils are anticipated to overlie the ECBF soil / rock.

4.2 Historical Aerial Photography Review

We have reviewed historical aerial photographs from Auckland Council Geomaps and Retrolens. These photographs were viewed under the context of identifying areas of potential instability and changes to landforms. Selected aerial photographs are presented in Appendix 3 and we have summarised our findings in Table 1.

Table 1: Historical Photographs Review

Year	Summary
1940	• Site is largely grassed, appears to be used for agriculture, except the western headland and most of the coastal margin which are vegetated with low bush and larger trees (likely Fir or Pine). • A small shed is present near the western headland, otherwise no other structures are apparent. • The gullies / streams in the eastern portion of the site are apparent, however less vegetated than currently. • Some indication that headscarp features near the western boundary are present, however this is unclear.
1963	• The two headscarp features in the western portion of the site appear in approximately the same form as they do today. • A row of large trees along the northern boundary / coastal margin have been removed. • A shelter belt of Macrocarpa trees has been planted dividing the western third of the site from the east. • Accessway along the southern boundary has been formed, although it passes through the stream/gully (i.e. no culvert installed). • Dwelling constructed within 97 Totara Road.
1972	• Existing dwelling and one of the farm sheds have been constructed on-site. • Culvert under the accessway appears to have been installed. • Some of the vegetation on the western headlands appears to have been cleared.
2000	• Land largely now in horticultural use. • Another farm shed has been constructed near the southern boundary.
2006	• No significant changes since 2000 except for some land in eastern end of site converted back to agricultural use (from horticultural).
2015	• No significant changes since 2006 except for some land in western end of site converted back to agricultural use (from horticultural).

5 Site Investigation

5.1 Investigations Completed

ENGEO attended site on 15 December 2022 to undertake a shallow geotechnical investigation. The investigation comprised:

- Site walkover by an experienced engineering geologist.
- Fourteen hand auger boreholes, named HA01 through HA14, to depths of up to 5.0 m.
- Collection of two bagged samples of shallow soils for Atterberg and Linear Shrinkage testing, primarily to assess soil expansivity.

The borehole logs have been prepared in general accordance with the New Zealand Geotechnical Society field classification guidelines (NZGS, 2005). All investigation locations are presented on the Geotechnical Investigation Location Plan in Appendix 4. Full hand auger borehole logs are presented in Appendix 5.

A summary of findings and development of a preliminary engineering geological model is presented in Section 6.

5.2 Laboratory testing

Soil samples selected for geotechnical laboratory testing were collected from materials recovered from hand augured boreholes. The testing was completed with reference to NZS4402:1986 Tests 2.1 to 2.6 and Test 5.1.2, as well as AS1289:2003 Test 7.1.1.

Summary of laboratory results are presented in Table 2, full test results prepared by ENGEO and received from Babbage Geotechnical Laboratory are included in Appendix 6.

Table 2: Summary of Laboratory Testing

HA ID	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)
HA02	0.5	44.2	84	33	51	21
HA13	0.5	33.1	49	21	28	13

Expansive soils are classified in NZS 3604 as soils with a liquid limit of greater than 50% and a linear shrinkage greater than 15%.

6 Engineering Geological Model

6.1 Surface Features / Geomorphology

ENGEO undertook a site walkover on 15 December 2022 to broadly assess geomorphological site features, exposed ground in coastal margin cliffs / slopes and general surface conditions. Key site features observed during our site walkover and from our desktop study are presented on the site features plan in Appendix 1 and are summarised as follows:

- The central / southern portion of the site is largely flat or slightly undulating where bisected by shallow drainage channels. This area contains the main dwelling and farm sheds and appears to have been largely used for horticulture.
- The eastern portion of the site is moderately undulating and is split by two streams entering from the southern and south-eastern boundary. These converge within the site before connecting to the tidal inlet that defines the north-eastern site boundary.
    - The banks of the stream are generally gently to moderately sloping (approximately 15 to 25 degrees), excluding some localised, low height head-scarp features.
    - The stream currently passes through a culvert beneath the existing accessway near the southern boundary.
- A west-east trending gully up to 4 m deep transmits much of the overland flow from the central portion of the site into the tidal inlet defining the north-eastern site boundary (Figure 1, Photo 1). The northern bank of the gully generally holds a slope of 1V:3H (18 degrees). The southern bank slopes at similar angles in the lower half, however the slope angle typically reduce to 1V:4H (14 degrees) for the top half of the gully wall. Several low height head scarp features were observed near the change in slope angle indicating active erosion.
- A headland protrudes from the western end of the site into the Kotukutuku Inlet; it is approximately 50 m wide at the narrowest point.
- An historical headscarp / slumping feature was observed south of the western headland, oriented west-southwest. The southern end of the headscarp is best defined, becoming less so to the north. Erosion appears to have smoothed the headscarp and vegetation had re-grown suggesting no recent rupture events. However, curved tree trunks at the vegetated toe of the slope may indicate soil creep is actively occurring.
- Another historical headscarp / slumping feature was observed north of the western headland, oriented north-west. The head scarp is not well defined due to erosion and re-vegetation. Mild curve to some tree trunks was observed at the toe of the slope which may indicate active soil creep.
- Slopes at the coastal margin were typically 45 degrees or steeper, varying significantly in height from approximately 1.5 to 10 m high. These traversed the entire coast, except where gullies / streams entered the inlet, and at the toe of the north-western slump feature which typically had gentler grades closer to 18 degrees.

- Exposures in the sea cliffs / slopes comprised orange-brown silts and clays overlying very weak to extremely weak siltstone and sandstone. The latter is associated with the East Coast Bays Formation (ECBF), however the former comprised both Alluvial deposits associated with the Puketoka formation and / or residually weathered ECBF.
  - Presence of the rock outcrops in exposures was variable, with some stretches having it consistently present in the bottom 1 to 3 m of the sea cliffs, while other stretches had no rock exposure or only occasional rock outcropping.
  - Rock outcrops in the central / northern portion of the coastline typically ranged between 7 to 10 degrees with 38 to 45 degree dip direction (Photo 3, Figure 1).
  - Rock outcrops in the western half of the coastline tended to have bedding dips of 44 to 77 degrees, with dip direction ranging from 208 to 287 degrees (Photo 4, Figure 1). We note that dip angles of this magnitude are generally uncommon in ECBF.
  - We were unable to access the eastern portion of the coastline due to rising tide.

Figure 1: Site Photos 15 December 2022

Photo 1: Existing gully in the north-eastern portion of the site. View Looking east from southern gully wall.

Photo 2: View looking north at headscarp in the south-western portion of the site. Approx. headscarp location shown by dotted line.

Photo 3: Sub-horizontal ECBF exposure on northern coastal margin.

Photo 4: Sub-vertical ECBF exposure in north-western coastal margin.

6.2 Subsurface Conditions

The site is veneered by topsoil with thicknesses ranging from 100 mm to 300 mm. Pre-existing fill was not encountered within the boreholes drilled on-site. However, discrete sections of filling are likely to be present proximal to pavements, residential dwellings and areas where recent farming related earthworks may have occurred.

Alluvial soils associated with the Puketoka formation were encountered beneath topsoil, except in boreholes HA05, HA12 and HA14. This material largely comprised stiff to very stiff silty clay or clayey silt, with localised softer layers. Beds of saturated silty sands and sandy silts (0.4 to 2.0 m thick) were also encountered, typically at 2 to 3 m beneath ground surface. Strengths in these silty sand /sandy silt beds were highly variable, ranging from soft to very stiff for sandy silts or very loose to dense for silty sands. Silt and sand dominant portions typically had low to no plasticity and some showed dilatant characteristics. Pumiceous silts made up a variable component of the overall silt content. At 1.2 to 1.3 in borehole HA04, the silt appeared to be almost entirely pumiceous, and exhibited high dilatancy.

Hand auger boreholes HA05 and HA12 were undertaken in the tidal zone beyond the site boundary; these encountered estuarine deposits to between 0.6 and 0.8 m bgl which were generally described as soft to stiff organic clayey SILT.

East Coast Bays Formation residual soils were encountered beneath the Puketoka Formation Soils in boreholes HA04, HA08, HA10, HA11, HA13 and HA14; and beneath the estuarine deposits in boreholes HA05 and HA12. These comprised stiff to hard silts and clays, with some to trace sand. Scala penetrometer testing in the base of boreholes HA04 and HA14 met refusal at 3.3 to 5.0 m bgl appearing to indicate the weathering transition into the underlying parent siltstone and sandstone. A Scala penetrometer undertaken in borehole HA12 met practical refusal on inferred transition zone at 1.3 m bgl. This borehole was located in the low lying tidal zone beyond the site boundary.

The variable presence and depth of ECBF in the hand auger boreholes suggests significant undulation in the ECBF / Puketoka Formation non-conformity. This also appears to align with variable presence of outcrop in the coastal cliffs as mentioned in Section 6.1.

Table 3: Generalised Geological Profile

Unit	Description	Occurrence	Depth Range (m bgl)	Thickness (m)	Average Peak Shear Strength [1] (kPa)	Average Scala Blows / 100 mm [2]
Topsoil	Organic SILT.	All test locations except HA05 & HA12.	0 to 0.3	0.1 – 0.3	NA	NA
Puketoka Formation	CLAY, SILT and SAND.	All test locations except HA05 & HA12.	0.1 to >5.0	3.0 to >5.0	116	6
East Coast Bays Formation	CLAY and SILT with variable sand content	HA04, HA05, HA08, HA10, HA11, HA12, HA13 and HA14	0.7 to >5.0	>5.0	180	NA

[1] only measured in cohesive soils

[2] only measured in silty sands and sandy silts.

NA = not assessed.

6.3 Groundwater

Depths to groundwater were recorded during the drilling of each borehole. Where possible, boreholes were re-dipped at the end of works to allow for groundwater to equalise. A summary of indicative groundwater levels measured in this manner are listed in Table 4.

Notably, heavy rainfall occurred during the morning of the investigation, and one to two months of unusually high levels of rain preceded the investigation.

Table 4: Groundwater Dip Measurements

HA ID	Groundwater Dip Measurement (m bgl)	
	During Drilling	End of Works
HA01	0.1	N/A
HA02	0.4	0.45
HA03	0.4	0.17
HA04	1	0.2
HA05	0*	N/A*
HA06	1.8	N/A
HA07	0.6	0.5
HA08	2.5	2.3
HA09	1.4	1.2
HA10	0.35	N/A
HA11	0.55	N/A
HA12	0*	N/A*
HA13	1.4	1.4
HA14	2.1	1.6

N/A = not assessed

*borehole drilled in tidal zone.

7 Geohazards and Geotechnical Assessments

7.1 Introduction

The following geotechnical hazards have been evaluated to determine the suitability of this site for the proposed development:

- Seismic hazards including liquefaction and lateral spreading.
- Consolidation settlement.
- Coastal regression and slope stability.

Each of these aspects is discussed in the following sections.

7.2 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, and landslides.

The following sections present a discussion of seismic hazards as they apply to the site.

7.2.1 Site Soil Class

A site soil classification of ‘Class C – Shallow Soils’ as per NZS 1170.5:2004 is considered to be appropriate for the site based on soil strength materials identified to the base of the investigations and our understanding of the geological setting.

7.2.2 Ground Rupture

There are no known active faults located within the site. Based on regional mapping, and the results of our field exploration, it is our opinion that fault-related ground rupture is unlikely at the subject site.

7.2.3 Ground Shaking

Ground shaking and subsequent effects on structures, infrastructure and engineering systems can be extensive and affect large areas. The intensity, frequency and duration of ground shaking drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

In geotechnical assessments, amplitude, frequency and duration of shaking are the main factors considered.

Based on our experience with previous Metlifecare developments, we understand the potential site development largely includes Importance Level 2 (IL2) structures, however the Care Building would typically be considered Importance Level 3 (IL3). Peak horizontal ground accelerations (a_{max}) for use in geotechnical assessments for each importance level are provided in Table 5. A_{max} values have been taken from the recommended values provided in Table A1 - Appendix A of MBIE/NZGS Module 1 for Auckland.

Table 5: Peak Horizontal Ground Acceleration

Limit State	Return Period	a_{max}	Magnitude
SLS	25 years	0.05 g	5.9
ULS (IL2)	500 years	0.19 g	6.5
ULS (IL3)	1000 years	0.20 g	6.5

7.2.4 Liquefaction

Soil liquefaction and lateral spread results from loss of strength during cyclic loading, such as those imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded cohesionless materials. Empirical evidence indicates that loose to medium dense gravels, sands, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable, with the more clayey soils more likely to experience strain-softening. Empirical data suggest geologically young (i.e. Holocene) sediments with the characteristics described above are most susceptible to the effects of liquefaction. Thin, constrained layers are unlikely to liquefy when bounded by non-liquefiable layers, particularly if they are not laterally continuous.

We consider the soft to firm sandy silts and loose to medium dense silty sands within the Puketoka Formation could have up to a moderate risk of liquefaction in an ultimate limit state event. However, given the high silt content, relative thinness of the beds and low anticipated accelerations, we expect liquefaction risk in the SLS case to be low.

Further testing (Cone Penetration Tests) and detailed analysis is recommended prior to detailed design phase to refine the liquefaction risk to the development. However, if a liquefaction risk from those deposits is assessed to be prevalent, we expect that this could be addressed through use of piled foundations penetrating through to the ECBF unit (which typically has a very low risk of liquefaction) or designing structures to limit differential settlement risk through use of stiffened foundations. The latter option should be relatively straightforward given proposed structures are predominantly single or two storey.

7.2.5 Lateral Spreading

Lateral spread is the lateral movement of ground as a result of liquefaction during an earthquake. Lateral spread occurs when a soil mass slides laterally on a liquefied layer and gravitational and seismic forces cause the layer, and overlying non-liquefied material, to move in a downslope direction or towards a free face. A free face can include any near-vertical cut but is commonly associated with riverbanks or creek terraces. The magnitude of lateral spreading depends on a variety of factors including earthquake magnitude and peak ground acceleration, thickness of the liquefied layer, and ground slope or ratio of free-face height to distance between the free face.

Based on our assessment in Section 7.2.4 and geometry of the coastal cliffs, we consider that there may be some risk of lateral spread, or liquefaction induced slope instability within the sea cliffs on the coastal margin. As the less susceptible ECBF is generally exposed at or above the high tide line, we would typically expect groundwater to be beneath the Puketoka Formation soils, and therefore this to be a low risk. However, our investigation has shown that a relatively steep groundwater gradient at the coastal margin could be present, at least after heavy rainfall, which increases this risk.

Lateral spread risk adjacent to the streams in the southwest of the site is considered to be low as soils in this area appear to be largely cohesive in nature. Lateral spread risk to the development adjacent to the gully in the northeast of the site is also considered low as we expect this will be backfilled.

Further testing and analysis should be undertaken during detailed design to refine the lateral spread risk. Having said that, we do not expect this to be an issue in SLS case. In the ULS case we anticipate the risk should be able to be addressed through reasonable setbacks from free faces and / or use of options such as land drainage, gravel rafts and / or piled foundations.

7.3 Consolidation Settlement

Soils undergo consolidation settlement when subject to loading such as bulk earth filling and / or loading from foundations. This occurs through particles rolling, sliding, and slipping into the void spaces together with some particle crushing (or fracturing) at the contact points. Soils that have been deposited relatively recently (e.g. alluvial or colluvial deposits) are generally more susceptible to consolidation settlement.

The Puketoka Formation soils encountered on-site largely comprised stiff to very stiff silts and clays which typically indicate relatively low potential for consolidation, however the loose sandy silts and soft to firm sandy silts encountered in some areas could have a moderate risk of consolidation settlement.

Based on existing plans, the proposed structures appear to be limited to a maximum of two storeys, and therefore foundation loads should be relatively low. However, as no earthworks plan is available at this time, we cannot assess the consolidation settlement risk from proposed fill loads.

Once preliminary earthworks plans become available, we recommend that supplementary testing (e.g. CPT, DMT, field dissipation and laboratory testing) is undertaken to refine the consolidation characteristics of soils on-site, and that detailed settlement analysis is undertaken to quantify expected surface settlement and timeframes for this due to fill and foundation loads. Based on our experience in similar, nearby sites we recommend that fill depths are limited as much as practical, and that the project timeline allows for preloading and / or monitoring of fill to confirm attenuation of settlements prior to commencing the vertical build phase.

7.4 Slope Stability

7.4.1 Coastal Regression

The site is bounded to the west, north and east by unprotected, coastal slopes and sea cliffs up to 10 m high, and a low energy tidal mudflats and channels at the toe. The assessed average rate of coastline regression for soft cliffs[1] in the Auckland region of 10 m over 100 years is a realistic assumption at this site. Erosion of land at the toe of the slope is a relatively slow process, as the marine environment in the inner Waitemata Harbour, in conjunction with the established mangrove protection, is relatively low energy. Conversely, soils exposed in the cliffs, and deposited as slump debris are weak and easily disturbed and regression of the coastal cliffs will continue to occur.

Evidence of undercutting at the coastal margin was observed during our site walkover and the geomorphology of the landform indicates that surface water flows associated with the overland flow paths at the site have accelerated erosion and scour within the gullies and existing landslide scarps.

Dense vegetation established across the steep slopes is serving to enhance their stability and should be retained where possible as part of a future development. Decommissioning the overland flow paths and limiting the surface water and stormwater discharge onto the slopes will also serve to reduce the rate of slope erosion in the long term. Notably the existing development plan has also designated the coastal margin as an esplanade reserve with structures situated a minimum of approximately 15 m from the crest of the coastal slopes / cliffs. Provided these factors are appropriately implemented / retained, we consider that the risk to the development from coastal regression is relatively low.

This report does not constitute a coastal hazard assessment.

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[1] Auckland Regional Council, Regional Assessment of Areas Susceptible to Coastal Erosion, Volume 1, May 2006

7.4.2 Global Stability

Our site walkover found that the eastern portion of the coastal margin expressed instability features generally aligning with typical coastal regression; that being steep collapses due to undermining of the sea cliffs. Therefore, addressing global stability risk in this area should be relatively straightforward as discussed in Section 7.4.1.

Indication of shallow instability was also observed on the sides of the north-eastern gully; however, we understand this is to be backfilled which should address the associated risk. Similar shallow instability was observed on the banks of the stream(s) in the south-eastern portion of the site. Based on observed ground conditions we do not expect any low angle or deep-seated instability in this area; however, we note that some structures are proposed near the stream bank which could present some risk of undermining due to stream bank regression. Therefore, the buildings should be appropriately set back from the stream, or the stream banks contoured to more stable batter angles and revegetated to aid stability.

Two existing landslide features were observed, in the western portion of the site during our walkover which appear to have a relatively low angle, and therefore likely indicate a mechanism beyond typical coastal regression / undercutting (although coastal regression may also be contributing). These features are described in Section 6.1 and annotated on the plan in Appendix 1. Review of historical aerial photos suggest that these features may have been present prior to 1940, but certainly before 1963, and that there hasn’t been any rapid movement of these since then. However, curving of tree trunks suggests that there may be ongoing and / or periodic creep occurring, particularly in the south-western landslide.

Notably, outcrops on the western coastal margin (below the landslide features) were observed to have a bedding dip much steeper, and at a different orientation than outcrops in the eastern portion of the site (discussed in Section 6.1). This may be a contributing factor to the instability, or potentially a consequence of the instability. In the latter case this could suggest a deep-seated slip plane.

The existing development plans show that several villas are proposed within or near the mapped landslide scarps, and therefore further investigation, including deep borehole investigations and more detailed mapping of the coastal margin should be undertaken to determine the mechanism of these landslides. Notably the south-western slip (which we consider most critical) is currently largely proposed as green space, with only one Villa appearing to marginally encroach on the head-scarp.

Detailed global stability analysis should be undertaken during subsequent project phases to refine the risk presented to the development and facilitate determination of appropriate mitigation measures where required. In addition to assessment of the existing landslide features in the west of the site, the assessment should include representative critical sections of the eastern coastal margin and proposed development above the stream in the southeastern corner of the site.

7.1 Preliminary AUP Groundwater Assessment

Where long-term groundwater take is expected, Auckland Council may require application for a consent for groundwater take and divert under Section E7.6.1.6 and / or E7.6.1.10 of the Auckland Unitary Plan. Although earthworks and retaining wall plans are not currently available, given the shallow groundwater encountered on-site, we expect that a consent is likely to be required. Once earthworks plans become available, groundwater monitoring should be undertaken in critical areas to assess whether cuts will extend below the groundwater table, and by how much. At that stage an assessment against Section E7.6.1.6 and E7.6.1.10 should be undertaken to determine requirement for a consent.

Where groundwater drawdown is proposed near site boundaries, a detailed assessment of affects may also be required to assess effects of drawdown and any boundary retaining walls on neighbouring structures. However, given the limited development and current site topography, we consider that this is not likely to be necessary.

8 Recommendations

8.1 Slope Protection

Evidence of historical and ongoing instability was noted in some areas of the site from various mechanisms, as discussed in Section 7.4.

Low to moderate instability risk presents from typical costal regression and collapse of stream banks in the eastern portion of the site, however a relatively higher risk is present to some of the villas proposed within the west of the site which are proposed above existing, larger landslide features.

Pending further investigation and analysis, we expect that existing setbacks from the coastal margin should be suitable to address instability risk from typical coastal regression and collapse of stream banks.

Where further analysis finds proposed setbacks are inappropriate, and to address risk to villas above the landslide features, one or more of the following solutions may be appropriate: installation of slope drainage (e.g. counterfort drains), rerouting of overland flow paths, remedial earthworks (e.g. unloading of slopes, slope benching, and construction of shear keys and / or batter slopes) and / or construction of in-ground palisade walls or other specifically designed retaining walls.

Based on aerial photos we are not expecting significant regression of the existing landslide features and therefore the risk from the western landslides could alternatively be addressed through moving the proposed villas outside of the landslide scarp area.

The preferred solutions should be determined in subsequent phases of the project in coordination with the client and pending results of proposed computational slope stability assessment. However, we recommend that an emphasis should be placed in the early design stages on unloading of slopes (i.e. prioritizing cuts over filling near slopes) and routing of overland flow away from existing slopes.

Further, a specialist coastal engineer should be engaged to complete a design-level assessment of the coastal margin to determine the requirement for coastal protection (such as construction of a rock-revetment or sea wall). If coastal protection is not implemented, regression over the next 100 years may extend up to 10 m inland of the current coastal margin.

8.2 Preliminary Building Foundations

8.2.1 Shallow Foundations

Based on the results of our investigation, and the assumption that future development will involve only one to two storied structures, we consider shallow foundations should be suitable for structures located away from instability areas.

Due to soft / loose soils observed in some areas, a reduced preliminary geotechnical ultimate bearing capacity of 200 kPa should be adopted for design of shallow strip, pad or raft foundations, founded within the native soils below any topsoil. Footing and foundation depths can be reassessed following confirmation of the site earthworks plans.

Based on shrink-swell index testing completed for this site, we consider that a preliminary site soil classification of H (High) with 500 year design characteristic surface movement return (ys) of 78 mm should be applied for the native clays and silts on-site. We do however note that the testing from one of the two samples fell within the M category and therefore if desired, further testing can be carried out across the site as part of the detailed design stage to refine the expansive soils for specific areas.

Shallow foundations may also need to consider liquefaction induced differential settlements. Likely this will only be relevant for the Ultimate Limit State case, however the magnitude of settlement is yet to be determined. Calculated settlements will be provided at later design stages. Where liquefaction settlement is assessed to be a concern foundations will need to be designed to accommodate these movements or the underlying ground improved to reduce the effect of this settlement on the proposed buildings. Typical responses include the use of a gravel raft under foundations or that buildings are supported on piled foundations extending through the liquefiable materials.

8.2.2 Deep Piles

If required, deep piled foundations should be specifically designed to meet future performance objectives of slope stability, liquefaction and compressible soils. Due to the presence of shallow groundwater and saturated silt and sand layers, bored piles that extend below the groundwater table may require dewatering, and / or casing to prevent necking or collapse of the pile holes.

Driven piles may provide a more practical option and often higher bearing capacities can be provided (compared to bored piles). Vibration effects on neighbouring structures requires consideration, however given the neighbouring properties are largely undeveloped, this is unlikely to be an issue.

If piled foundations are required, ENGEO should be engaged to provide bearing parameters.

As no deep investigations have been undertaken on-site, we are unable to provide an estimate of depth to rock for piles proposed to be founded within bedrock. However, given the relatively low height of most the proposed buildings, we do not expect piles founded within rock to be necessary for most buildings.

8.3 Preliminary Geotechnical Parameters for Retaining Wall Design

Based on the site topography, we anticipate retaining walls will be required to create level building platforms.

The following soil parameters may be used for preliminary retaining wall design. Future walls will likely retain native Puketoka Formation soils or engineered fill however this needs to be confirmed once finished levels are decided upon. ENGEO should be engaged to confirm appropriate soil parameters for specific retaining walls prior to commencing detailed design.

A summary of preliminary soil parameters for retaining wall design is provided in Table 6.

Table 6: Geotechnical Soil Parameters for Retaining Wall Design

Material Type	Unit Weight □ (kN/m^3)	Friction Angle ϕ' (degrees)	Effective Cohesion (c' kPa)	Undrained Shear Strength (Su kPa)
Retained Native Puketoka Formation Soil	17	27	0	40
Retained Engineered Clay Fill	18	32	5	100

These retaining wall parameters may not be suitable for use in the design of slope stabilisation structures, or for design of walls within the existing landslide features. Those walls should be designed by, or in close coordination with ENGEO. Further, design of any retaining walls near neighbouring boundaries should be assessed for their effects on neighbouring structures.

The design of rigid retaining walls (i.e. walls that are restrained from movement at the top), should be based on an 'at rest' lateral earth pressure coefficient (Ko). Flexible walls that are free to deform or rotate at least 1% of the exposed wall height (H) may be designed using an active soil coefficient (Ka).

Design calculations should take into account slope angles above the wall, toe slopes and other surcharges above the wall. Where retaining walls, support roads / accessways are on site boundaries, these should be designed to accommodate a surcharge of at least 12 kPa.

8.4 Settlement Monitoring

Based on our high-level consolidation settlement assessment in Section 7.3, we consider there is some risk of consolidation settlement from filling and other loads above the Puketoka Formation soils. The magnitude of settlement will depend on proposed fill depths and locations. Further testing and specific settlement analysis should be undertaken once earthworks plans become available.

Settlement monitoring would typically comprise installation of a series of survey points installed above the fills, which are monitored on a regular basis to confirm fill induced settlements have attenuated prior to building construction commencing. The majority of settlement often occurs during fill placement, however in our experience it can take in the order of three to twelve months for settlements to attenuate. Often this can be worked into the project schedule by staging monitored areas to later phases of the development, however if these time frames are not acceptable then alternative options can be considered. Those may include options such: as surcharge / preloading, utilising light weight fills or specification of ground improvement options such as gravel rafts, rigid inclusions or load compensation (i.e. dig-out and replacement with light weight fills).

8.5 General Site Works

8.5.1 Demolition

It is essential that all foundations and building debris from demolition of the existing buildings and retaining walls are completely removed from within the extent of works prior to earthworks commencing. Any septic tanks and related infrastructure associated with the dwellings and farm buildings should be decommissioned and removed completely.

Where foundations are removed below final ground level, they will need to be backfilled with approved hardfill (e.g. GAP65 or similar approved product) compacted in maximum 200 mm thick layers to ensure a consistent subgrade.

If any existing services are to be decommissioned, the abandoned lines should be fully removed or backfilled with grout to avoid creating preferential groundwater flow paths. All trench backfill will also need to be removed and replaced with engineer certified fill in the vicinity of the proposed buildings in order to avoid the need for pipe bridging piles, which may also trigger a requirement for additional ground investigation.

8.5.2 General Site Preparation

All topsoil / vegetation and pre-existing fill should be stripped from beneath the building platforms prior to any fill being placed. An allowance should also be included for additional undercuts to remove unsuitably soft native soils, and backfilling of undercuts with engineered fill.

8.5.3 Tree Removal

If vegetation removal is proposed, it is essential the geotechnical engineer is contacted for guidance. Mature trees established on the slopes throughout the site involve extensive root systems and it may be detrimental to the stability of the surficial soils if removed.

Where trees are to be removed within development areas, it is important that all tree stumps and large roots (greater than thumb-size) are completely removed from the building platform and the immediate surroundings. Where large areas are to be cleared of vegetation the most effective approach would be to undercut the affected area to remove the large root systems and replace with engineered fill to design levels as required.

8.5.1 Existing Gullies and Overland Flow Paths

The existing gullies and overland flow paths at the site serve to direct surface water and irrigation runoff to the coastal margins of the site, where scour has exacerbated regression of the slopes at those locations.

Where development is proposed above overland flow paths, these will need to be mucked out to expose inorganic native soil, drained via a 160 mm diameter perforated highway grade novacoil pipe in geotextile-wrapped TNZ F2 drainage bedding, and capped with site-won clay fill or other approved engineered fill up to finished ground level. The drains should either connect to the proposed stormwater infrastructure at the site if levels permit, or discharge to the toe of the boundary slopes via a geotechnical engineer approved outlet structure (e.g. PVC flume).

8.5.2 Bulk Earthworks

Although earthworks plans are not yet available, we expect that significant cut and fill earthworks will be required to create desired finished levels, largely within the Puketoka Formation soils These soils should generally be suitable for handling and compaction using conventional earthworks plant but may be wet of optimum and accordingly some conditioning may be required prior to placement as structural fill.

Fill should comprise clean clay, crushed concrete or hardfill and should be approved by the Engineer prior to use. Compaction should be carried out to certified standards (NZS 4431) with conventional plant and under engineering control. The Geotechnical Engineer should be given every opportunity to observe materials prior to placement and during compaction to carry out QA testing as required.

Where fill is to be placed on sloping ground, the ground must be benched to receive fill to minimise the risk of a preferential shear surface developing at the fill / native interface. Where groundwater springs and seepage are encountered in fill areas, geotechnical underfill drainage will be required to collect the water and direct it to the stormwater system or an approved outlet structure.

Where fill is to be placed adjacent to the coastal margins or streams in the southwest corner, specific slope stability analyses will be required. All retaining walls adjacent to these slopes will be designed to support the fill and new building loads, and to intercept slip surfaces having unacceptable Factors of Safety that may encroach into the development area. These analyses are best completed at the detailed design stage of the project when earthworks design levels are better understood, the retaining wall locations are determined, and supplementary deep investigations can be advanced as required.

8.5.3 Cuts / Temporary Batters

Short-term batter slopes of 1V:1H (45 degrees) may be used for batter heights of up to 1.5 m. Unsupported cut batters greater than 1.5 m should be specifically designed.

All temporary batters should be covered with polyethene sheeting or a similar material to control moisture loss and prevent scouring from rainfall. A bund should be formed above the crest of the batter to divert surface runoff away from the batters.

We do not expect that rock breaking will be required during bulk earthworks or excavation of service trenches.

8.6 Stormwater and Effluent Disposal

Overland flows should be directed away from existing slopes to reduce the risk of ponding and erosion leading to slope instability concerns. Further, we recommend avoiding soakage and / or dispersal of stormwater above identified landslides and steep slopes around the coastal margin. Notably, the soils beneath the site are inferred to have a very low permeability, and therefore it’s unlikely that ground soakage will be a practical method for stormwater disposal.

8.7 Pavement Subgrade CBR

An inferred preliminary CBR design value of approximately 3% may be adopted for preliminary pavement and slab design in native soils.

The above CBR values are preliminary only. Specific *in situ* testing of the exposed subgrade is recommended following earthworks and prior to finalising pavement designs, including the use of *in situ* and soaked CBR testing and falling weight deflectometer. Where localised uncontrolled fill is encountered, it will be necessary to remove this fill and replace it with engineered fill. Additional subgrade improvement requirements may be necessary to achieve council requirements. This may include undercut and replacements, and / or the use of geogrid.

9 Future Work

Once earthworks and architectural plans become available, we should be given an opportunity to review them and confirm that the assumptions underlying this report are still valid and modify any recommendations as necessary.

As noted throughout the report we would also recommend the following supplementary investigations and analysis to quantify identified risks and facilitate design of their mitigation:

- A suite of cone penetration tests to facilitate liquefaction and consolidation settlement assessments.
- A suite of deep machine boreholes: these are intended to be used to generally refine the ground model for deeper soils and to assess the depth and characteristics of the bedrock. However, these are also necessary to facilitate development of our slope stability models including to assess the mechanism and characteristics of landslides in the western portion of the site.
- Supplementary shallow investigations (hand augers and / or test pits) may be required to infill critical areas between existing investigations, for example to target locations of proposed retaining walls.
- Installation of piezometers and monitoring of groundwater levels to facilitate liquefaction, consolidation settlement, and AUP groundwater assessments.
- Detailed computation liquefaction and lateral spreading assessment.
- Computational slope stability assessment.
- A specialist coastal engineer should be engaged to complete a design-level assessment of the coastal margin to determine the requirement for coastal protection (such as construction of a rock-revetment or sea wall).

We recommend that we are involved throughout the detailed design phase to provide geotechnical inputs as required. However, as a minimum we must be given an opportunity to review the final drawings prior to building consent application.

It is also essential that we are given every opportunity to attend a pre-start meeting on-site prior to works commencing and then to observe site works, including site stripping, earthworks operations and ground conditions in foundation and retaining wall excavations (prior to pouring concrete) to confirm works are carried out in accordance with the recommendations of this report and that ground conditions are as assumed.

Upon successful completion of the works we would then be in a position to provide a Producer Statement – Construction Review (PS4).

10 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Metlifecare Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by

Nick Mellsop
Geotechnical Engineer

Report reviewed by

Paul Fletcher, CMEngNZ (CPEng)
Associate Geotechnical Engineer

APPENDIX 1:

Site Features Plan

Legend

- Watercoarse
- Inferred Headscarp
- Headscarp
- Gully
- Slope/Sea Cliff Crest
- Site Boundary

0 25 m 50 m

© Nearmap,

ENGEO

Produced by **Datanest.earth**

Title: Site Features Plan		
Client: Metlifecare Ltd		Appendix No.: 1 Size: A3
Project: Metlifecare Whenuapai Village	Drawn: NM	
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

APPENDIX 2:

Development Plan

MASTERPLAN
Whenuapai Metlifecare

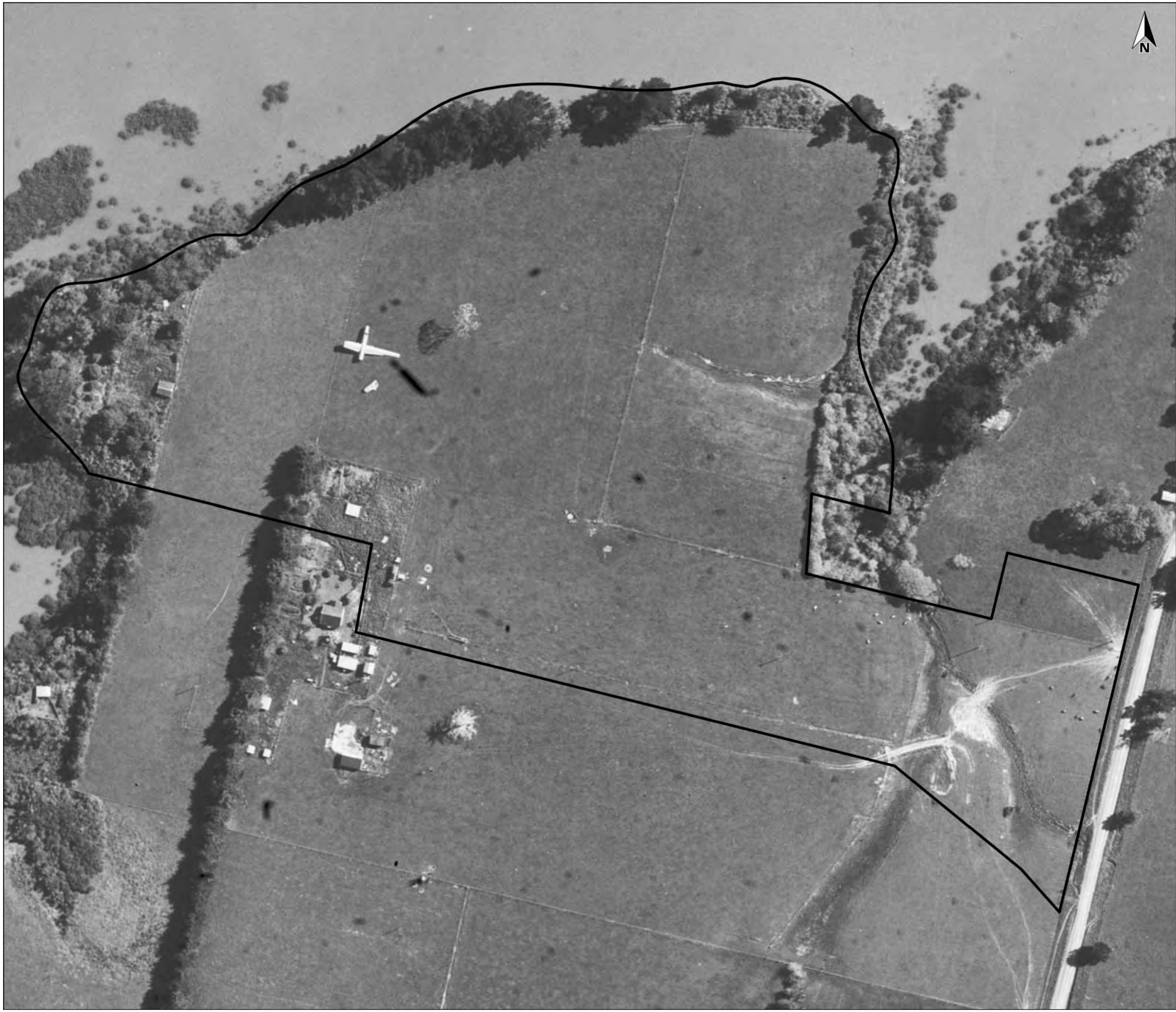
02/02/2023

1:2000 @A3 North

RESET URBAN DESIGN

APPENDIX 3:

Historical Aerials



Title: 2015 Historical Aerial		
Client: Metlifecare Ltd		
Project: Metlifecare Whenuapai Village	Drawn: NM	Appendix: A3-1 Size: A3
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

Legend

Site Boundary

0 25 m 50 m

© Nearmap

ENGEO

Produced by Datanest.earth

Title: 1963 Historical Aerial		
Client: Metlifecare Ltd		
Project: Metlifecare Whenuapai Village	Drawn: NM	Appendix: A3-2 Size: A3
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

Legend

Site Boundary

0 25 m 50 m
© Nearmap

ENGEO

Produced by **Datanest.earth**

Title: 1972 Historical Aerial		
Client: Metlifecare Ltd		
Project: Metlifecare Whenuapai Village	Drawn: NM	Appendix: A3-3 Size: A3
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

Legend

Site Boundary

0 25 m 50 m

© Nearmap, public

ENGEO

Produced by **Datanest.earth**

Title: 2000 Historical Aerial		
Client: Metlifecare Ltd		Appendix: A3-4 Size: A3
Project: Metlifecare Whenuapai Village	Drawn: NM	
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

Legend

Site Boundary

0 25 m 50 m

© Nearmap, public

ENGEO

Produced by **Datanest.earth**

Title: 2006 Historical Aerial		
Client: Metlifecare Ltd		
Project: Metlifecare Whenuapai Village	Drawn: NM	Appendix: A3-5 Size: A3
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

Legend

Site Boundary

2015 & 2016 Historical Aerial

Boundary

Footprint

Image

Red: Band_1

Green: Band_2

Blue: Band_3

0 25 m 50 m

© Nearmap, public

ENGEO

Produced by **Datanest.earth**

Title: 2015 Historical Aerial		
Client: Metlifecare Ltd		
Project: Metlifecare Whenuapai Village	Drawn: NM	Appendix: A3-6 Size: A3
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

APPENDIX 4:

Investigation Location Plan

Legend

- Hang Auger Boreholes
- Proposed Villa
- Proposed Amenity Building
- Proposed Care Building
- Site Boundary

0 25 m 50 m

© Nearmap,

ENGEO

Produced by **Datanest.earth**

Title: Geotechnical Investigation Location Plan		
Client: Metlifecare Ltd		Appendix No.: 4 Size: A3
Project: Metlifecare Whenuapai Village	Drawn: NM	
Date: 01-02-2023	Checked: PF	
Proj No.: 19709.000.001	Scale: 1:1600	Version: 2.0

APPENDIX 5:

Hand Auger Logs

LOG OF AUGER HA01

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 3 m
Hole Diameter : 50 mm

Shear Vane No : 2093
Logged By : JC
Reviewed By : NM
Latitude : -36.7779016
Longitude : 174.6241967

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm (2 4 6 8 10 12)
	TS	OL	[TOPSOIL]			▽		N/A		
0.5	PUKETOKA FORMATION	ML	Clayey SILT; light grey with orange streaks. Low plasticity.				M	VSt	112/38	
		CH	Silty CLAY; light grey with orange streaks. High plasticity.		8			VSt	110/48	
1.0		ML	Clayey SILT; light grey with orange streaks. Low plasticity.					VSt	135/56; 127/53	
1.5, 2.0, 2.5		ML	Clayey SILT with trace fine sand; light grey with orange streaks. Low plasticity.		7			VSt-H	160/63; 167/39; >197; >197	
3.0		ML	Sandy SILT with minor clay; light grey with orange streaks. Low plasticity.		6			VSt	148/24	
			End of Hole Depth: 3 m Termination Condition: Target depth							

Hand Auger met target depth at 3 m.
Dip test during drilling showed standing water at 0.1 m depth.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

TS = Topsoil; N/A = Not Assessed.

LOG OF AUGER HA02

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 3 m
Hole Diameter : 50 mm

Shear Vane No : 1546
Logged By : NM
Reviewed By : NM
Latitude : -36.777522
Longitude : 174.6224994

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0–0.15	TS	OL	[TOPSOIL]			M	N/A	
0.15–0.8	PUKETOKA FORMATION	ML	Clayey SILT; greyish brown. Low plasticity.		▽ ▼ (~0.4–0.45 m)	M	VSt	104/30 (0.3); 128/52 (0.6)
0.8–3.0	PUKETOKA FORMATION	CH	Silty CLAY; orange brown. High plasticity	11 (1.0); 10 (2.0); 9 (3.0)		M	St-VSt	125/56 (0.9); 151/68 (1.2); 178/107 (1.5); 128/74 (1.8); 125/64 (2.1); 95/64 (2.4)
2.7			2.7 m - becomes stiff and saturated.			S	St	89/70 (2.7)
3.0			3.0 m - becomes greyish white with orange streaks.					67/53 (3.0)

End of Hole Depth: 3 m
Termination Condition: Target depth

Scala Penetrometer — Blows per 100mm: 2 4 6 8 10 12

Hand Auger met target depth at 3 m.
Dip test during drilling showed standing water at 0.4 m depth.
Dip test at end of works showed standing water at 0.45 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed.

LOG OF AUGER HA03

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 3 m
Hole Diameter : 50 mm

Shear Vane No : 3230
Logged By : CH
Reviewed By : NM
Latitude : -36.7772057
Longitude : 174.6210131

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0–0.2	TS	OL	[TOPSOIL]		▼	M	N/A	
0.2–0.4	PUKETOKA FORMATION	ML	Clayey SILT; orange brown. Low plasticity.		▽	M	St	56/7
0.4–1.1	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange streaks. High plasticity. 0.8 m - becomes grey with orange streaks	12		W	Vst	106/45 192/111
1.1–1.9	PUKETOKA FORMATION	ML	Clayey SILT with trace pumiceous silt; grey with orange streaks. Low plasticity.	11		W	VSt-H	>205 >205 173/110
1.9–3.0	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange streaks. High plasticity. 2.6 m - becomes stiff. 2.7m - poor recovery due to groundwater suction.	10		W	VSt St	106/56 138/91 75/47 69/45
3.0			End of Hole Depth: 3 m Termination Condition: Target depth					

Scala Penetrometer — Blows per 100mm: 2 4 6 8 10 12

Hand auger met target depth at 3 m.
Dip test during drilling showed standing water at 0.4 m depth.
Dip test at end of works showed standing water at 0.17 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed.

LOG OF AUGER HA04

ENGEO

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

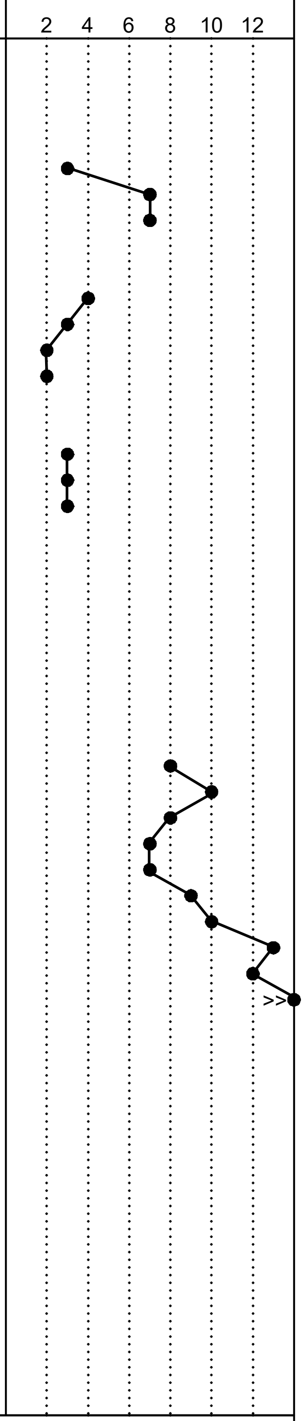
Client	: Metlifecare Ltd	**Shear Vane No**	: 3230
Client Ref.	: 19709.000.001	**Logged By**	: RD
Date	: 15/12/2022	**Reviewed By**	: NM
Hole Depth	: 5 m	**Latitude**	: -36.7770329
Hole Diameter	: 50 mm	**Longitude**	: 174.61991

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0–0.3	TS	OL	[TOPSOIL]		▼	W	N/A	
0.3–0.6	PUKETOKA FORMATION	ML	SILT with minor clay and fine sand; greyish brown. Low plasticity; dilatant.	8		S	St	
0.6–1.2	PUKETOKA FORMATION	ML	Fine Sandy SILT with trace clay; light brown. Low plasticity; dilatant.		▽ (1.0)	S	St-VSt	
1.2–1.35	PUKETOKA FORMATION	ML	Fine Sandy SILT; light grey. Non-plastic; dilatant. SILT is pumiceous.	7		S	F	
1.35–3.4	PUKETOKA FORMATION	ML	Fine Sandy SILT with trace medium sand and clay; light brown. Low plasticity; dilatant. 1.5 m -poor recovery below 1.5 m due to groundwater suction. 1.8 m: 50 mm layer of clayey SILT; some fine sand. Low plasticity. 2.2 m - becomes dark grey and very stiff to hard. 3.2 m - becomes wet.	6, 5		S (to 3.2), W	St (to 2.2), VSt - H	57/26 (2.1); >205 (2.4); >205 (2.7)
3.4–5.0	ECBF	ML	Clayey SILT with trace fine sand; dark grey. Low plasticity.	4		W	VSt - H	>205 (4.2); >205 (4.5); >205 (4.8)

End of Hole Depth: 5 m
Termination Condition: Target depth

Scala Penetrometer

Blows per 100mm



Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 1.0 m depth.
Dip test at end of works showed standing water at 0.2 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed; ECBF = East Coast Bays Formation.

ENGEO

LOG OF AUGER HA05

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 1.1 m
Hole Diameter : 50 mm

Shear Vane No : 3230
Logged By : RD
Reviewed By : NM
Latitude : -36.776799
Longitude : 174.6193548

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm (2 4 6 8 10 12)
0.0–0.8	ESTUARINE DEPOSITS	ML	Clayey SILT; brown. Low plasticity.		1		S	S	15/7; 22/7	
0.8–1.1	ECBF	ML	Clayey SILT; with minor fine sand; grey. Low plasticity.						>205	

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

End of Hole Depth: 1.1 m
Termination Condition: Practical refusal

Hand auger met practical refusal at 1.1 m depth on hard material.
Dip test during drilling showed standing water at the ground surface.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

ECBF = East Coast Bays Formation.

LOG OF AUGER HA06

ENGEO

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 2853
Logged By : ZS
Reviewed By : NM
Latitude : -36.7767276
Longitude : 174.6224499

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TS	OL	[TOPSOIL]	M	N/A
PUKETOKA FORMATION	ML	SILT with trace clay; orangish brown. Low plasticity.	M	VSt
PUKETOKA FORMATION	ML	Clayey SILT; brownish orange and dark grey staining. Low plasticity	M	VSt
PUKETOKA FORMATION	ML	Clayey SILT; light blueish grey with orange streaks. Low plasticity.	M / W	VSt
PUKETOKA FORMATION	ML	Clayey SILT with trace sand; light bluish grey with orange streaks. Low plasticity.	W	VSt
PUKETOKA FORMATION	SM	Silty SAND; intermixed orange brown and light blueish grey. Low plasticity.	S	VL
PUKETOKA FORMATION	ML	Clayey SILT with minor sand; orange brown. Low plasticity. 4.5m - becomes blueish grey.	S	St-VSt

End of Hole Depth: 5 m
Termination Condition: Target depth

Depth (m BGL): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

Elevation (mRL): 11, 10, 9, 8, 7

Water Level: ▽ (approx. 1.8 m)

Shear Vane Undrained Shear Strength (kPa) Peak/Remolded: 141/30, 188/58, 145/46, 145/48, 149/64, 149/72, 138/62, 122/68, 127/46, 73/19, 127/61, 101/64, 110/55, 91/51

Scala Penetrometer — Blows per 100mm: 2, 4, 6, 8, 10, 12

Hand auger met target depth at 5 m.
Dip test during drilling showed standing water at 1.8 m depth.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

TS = Topsoil; N/A = Not Assessed.

ENGEO

LOG OF AUGER HA07

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 4 m
Hole Diameter : 50 mm

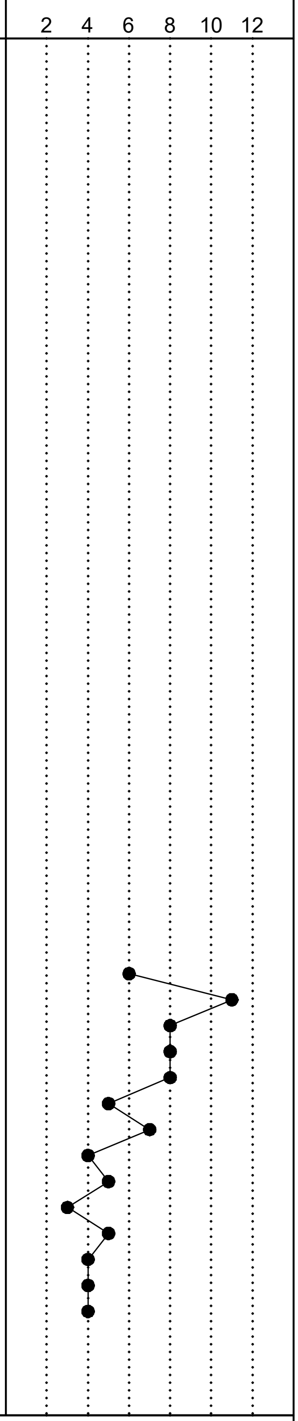
Shear Vane No : 1413
Logged By : LM
Reviewed By : NM
Latitude : -36.77664
Longitude : 174.6213884

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
TS	OL	[TOPSOIL]	M	N/A	
PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand and rootlets; dark brown. Low plasticity.	M	St	65/9 72/14
PUKETOKA FORMATION	ML	Fine to medium Sandy SILT with some clay; light brown. Low plasticity.	M	St-VSt	50/11 56/11 196/45
PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand; light to dark grey. Low plasticity. 2.7 m - becomes light grey with orange streaks. 2.8 m - becoms wet.	M / W	F-St	64/39 44/40 62/48
PUKETOKA FORMATION	SM	Silty fine to medium SAND; dark grey. Low plasticity.	W	MD-D	

End of Hole Depth: 4 m
Termination Condition: Practical refusal

Depth (m BGL): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0
Elevation (mRL): 10, 9, 8, 7

Scala Penetrometer



Hand auger met practical refusal at 4 m depth on hard material.
Dip test at end of works showed standing water at 0.6 m depth.
Dip test at end of works showed standing water at 0.5 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A= Not assessed

ENGEO

LOG OF AUGER HA08

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 1413
Logged By : LM
Reviewed By : NM
Latitude : -36.7762555
Longitude : 174.6198428

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
0.0–0.2	TS	OL	[TOPSOIL]	M	N/A
0.2–3.3	PUKETOKA FORMATION	ML	Clayey SILT with some fine to medium sand; light grey with orange streaks. Low plasticity. 0.4 m - becomes orange brown. 0.7 m - becomes light grey with orange streaks. 1.1 m - becomes firm. 1.4 m - becomes stiff to very stiff. 2.3 m - encountered standing groundwater. 2.5 m - becomes wet.	M (to 2.5 m), W	St (0.2–1.1 m), F (1.1–1.4 m), St-VSt (1.4–3.3 m)
3.3–3.7	PUKETOKA FORMATION	ML	Clayey SILT some fine to coarse sand; light grey with orange and black mottling. Low plasticity. Sand is angular to sub rounded.	W	VSt
3.7–3.9	PUKETOKA FORMATION	ML	Clayey SILT with some fine sand and trace medium sand; orange. Low plasticity. 3.8 m - becomes light with orange mottling.	W	VSt-H
3.9–4.3	ECBF	ML	Fine Sandy SILT with some clay; intermixed light grey and orange. Low plasticity.	W	H
4.3–4.6	ECBF	ML	Clayey SILT with some fine sand; light grey with orange mottling. Low plasticity. 4.5 m - becomes dark grey.	W	H
4.6–5.0	ECBF	SM	Silty fine SAND with some clay; dark grey. Low plasticity. 4.7 m - becomes light brownish grey. 4.8 m - becomes light grey with orange mottling.	W	N/A

End of Hole Depth: 5 m
Termination Condition: Target depth

Elevation (mRL): 10 (at 0.2 m), 9 (at 1.2 m), 8 (at 2.2 m), 7 (at 3.2 m), 6 (at 4.2 m)

Water Level: standing water at 2.3 m (end of works) and 2.5 m (during drilling)

Depth (m)	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.3	64/23
0.6	87/30
0.9	87/37
1.2	47/11
1.5	146/78
1.8	73/23
2.1	93/33
2.4	140/58
2.7	140/68
3.0	135/54
3.3	140/47
3.6	143/56
3.9	>218
4.2	UTP
4.5	UTP

Scala Penetrometer — Blows per 100mm: 2 4 6 8 10 12

Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 2.5 m depth.
Dip test at end of works showed standing water at 2.3 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed; ECBF = East Coast Bays Formation.

ENGEO

LOG OF AUGER HA09

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 16/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 1546
Logged By : JM
Reviewed By : NM
Latitude : -36.7760938
Longitude : 174.6225625

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TS	OL	[TOPSOIL]	M	N/A
PUKETOKA FORMATION	ML	Clayey SILT; light greyish brown with orange streaks and black mottles. Low plasticity.	M	VSt
PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand; Light greyish brown with orange and red streaks. Low plasticity. 1.2 m - becomes very stiff. 1.3 m - becomes light grey with orange streaks.	M / W	St / VSt
PUKETOKA FORMATION	ML	Clayey SILT with minor fine sand; light grey with orange streaks. Low plasticity.	W	VSt
PUKETOKA FORMATION	ML	Fine to medium sandy SILT with trace clay; light grey with orange mottles.	S	St-VSt
		End of Hole Depth: 5 m Termination Condition: Target depth		

Depth (m BGL): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0
Elevation (mRL): 10, 9, 8, 7, 6

Shear Vane Undrained Shear Strength (kPa) Peak/Remolded: 105/56, 101/43, 58/30, 116/47, 133/53, 181/64, 156/67, 169/73, 129/73, 148/73, 86/71, 131/59, 148/58, 107/56, 178/98, 156/86, 141/62

Scala Penetrometer – Blows per 100mm: 2, 4, 6, 8, 10, 12

Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 1.4 m depth.
Dip test at end of works showed standing water at 1.2 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed.

ENGEO

LOG OF AUGER HA10

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 2093
Logged By : NM
Reviewed By : NM
Latitude : -36.7761759
Longitude : 174.6214715

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TS	OL	[TOPSOIL]		N/A
PUKETOKA FORMATION	ML	SILT with some clay; greyish brown. Low plasticity.	W	VS
PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange streaks. High plasticity.	W	St-VSt
PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand; light grey with orange streaks. Low plasticity.	W	VSt
PUKETOKA FORMATION	ML	SILT with some fine to medium sand and some clay; yellowish grey. Low plasticity. Poor recovery.	S	F-VSt
ECBF	CH	Silty CLAY; yellowish grey. High plasticity 3.6 m - becomes dark grey.	W	VSt
ECBF	ML	Sandy SILT with minor clay; dark grey. Low plasticity. Poor recovery due to groundwater suction.	S	F-St
ECBF	CH	Silty CLAY; dark grey. High plasticity.	W	VSt
ECBF	ML	SILT with some fine sand and clay; dark grey. Low plasticity.	S	St
		End of Hole Depth: 5 m Termination Condition: Target depth		

Depth (m BGL): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

Elevation (mRL): 10, 9, 8, 7, 6

Water Level: ∇ (at 0.35 m)

Shear Vane Undrained Shear Strength (kPa) Peak/Remolded: 14/3, 93/14, 127/28, 127/56, 124/58, 110/44, 111/32, 42/17, 70/25, 112/34, 84/28, 162/112, 183/67, 98/42

Scala Penetrometer — Blows per 100mm: 2, 4, 6, 8, 10, 12

Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 0.35 m depth.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

TS = Topsoil; N/A = Not Assessed.
ECBF = East Coast Bays Formation.

LOG OF AUGER HA11

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 2853
Logged By : CH
Reviewed By : NM
Latitude : -36.7758827
Longitude : 174.620406

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm
0.0	TS	OL	[TOPSOIL]			W	N/A		
0.2	PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand and rootlets; brown with light orange streaks. Low plasticity.		▽ (0.55 m)	W / S	St	94/23	
0.7		ML	Clayey SILT with some sand; brown. Low plasticity.			S	F	41/17	
1.0		ML	Clayey SILT with trace sand; light grey with some orange. Low plasticity. 1.0 - 1.4 m - poor recovery. 1.4 m - becomes stiff to very stiff.	5		W	S-F / St-VSt	138/48, 58/17, 86/44, 91/36, 86/28, 72/33	
3.2	EAST COAST BAYS FORMATION	ML	Clayey SILT with trace sand; orange with some grey. Low plasticity. 3.4 m - limonite staining (fine gravel sized nodules.)	3 / 2		W	VSt - H	UTP, >193, UTP, UTP	
4.2		ML	SILT with some some fine to medium sand and clay; greyish orange. Low plasticity.			W	H	UTP, UTP	
5.0			End of Hole Depth: 5 m Termination Condition: Target depth	1					

Hand auger met target depth at 5 m.
Dip test during drilling showed standing water at 0.55 m depth.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

TS = Topsoil; N/A = Not Assessed.
ECBF = East Coast Bays Formation.

LOG OF AUGER HA12

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

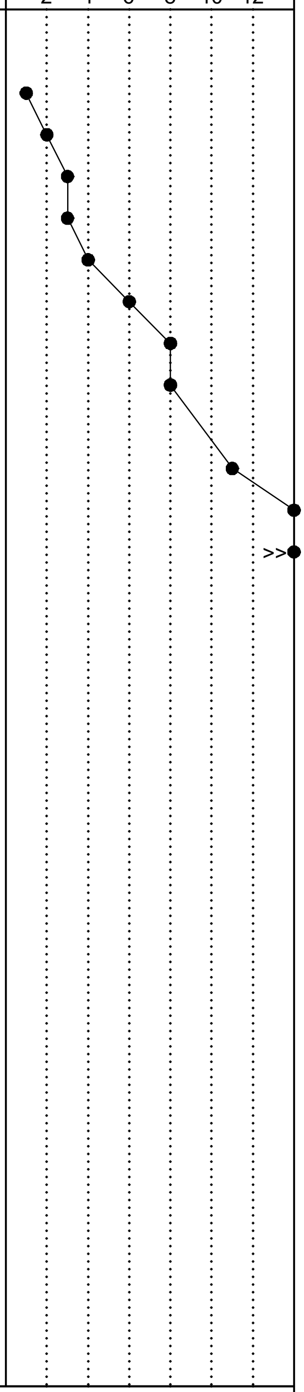
Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 1.3 m
Hole Diameter : 50 mm

Shear Vane No : 2853
Logged By : CH
Reviewed By : NM
Latitude : -36.77568
Longitude : 174.6201661

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0 – 0.7	ESTUARINE DEPOSITS	OL	Organic clayey SILT with minor fine sand; dark brownish grey. Low plasticity. Black fibrous organic inclusions.		1 (at 0.6)	▽ (at 0.0)	M	St - VSt	52/8 (at 0.3); 106/7 (at 0.6)
0.7 – 1.3	ECBF	ML	Clayey SILT with trace fine sand; greyish brown. Low plasticity.				M	VSt - H	>193 (at 0.9)
1.3			End of Hole Depth: 1.3 m Termination Condition: Practical refusal						

Scala Penetrometer

Blows per 100mm



Hand auger met practical refusal at 1.3 m depth .
Dip test during drilling showed standing water at the ground surface.
Coordinates assessed using Datanest Mobile App
Elevations estimated from site topo survey (H&G 2022)

ECBF = East Coast Bays Formation.

LOG OF AUGER HA13

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 3230
Logged By : RD
Reviewed By : NM
Latitude : -36.77553159
Longitude : 174.6216636

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	[TOPSOIL]	8		M	N/A	
	EAST COAST BAYS FORMATION	ML	SILT with some fine sand; grey with orange streaks. Low plasticity.				VSt	169/47
0.5		ML	SILT with some clay and minor fine sand; orange with dark orange mottles. Low plasticity.				VSt	164/98
1.0		CH	CLAY with minor silt and fine sand; light brownish orange with grey streaks. High plasticity. 1 m - becomes light grey with orange streaks.	7	▼ (1.4 m)		VSt	141/75 141/59
1.5		ML	Clayey SILT with some fine sand; light grey with orange streaks. Low plasticity.			W	VSt	192/73 147/75
2.0		CH	CLAY with minor silt and fine sand; light brownish orange with grey streaks. High plasticity. 2.2 m - becomes saturated.	6		S	VSt	164/60 164/63
2.5 / 3.0		ML	Clayey SILT with some fine sand; light grey with orange streaks. Low plasticity.	5			VSt	164/66 147/56 142/56
3.5 / 4.0		ML	Fine sandy SILT with minor clay; orange with light brown streaks. Low plasticity; dilatant.	4			VSt	176/57 173/37 >205
4.5		ML	SILT with some fine sand and minor clay; light grey with orange streaks. Low plasticity.				H	>205 >205
5.0			End of Hole Depth: 5 m Termination Condition: Target depth					

Scala Penetrometer — Blows per 100mm: 2 4 6 8 10 12

Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 1.4 m depth.
Dip test at end of works showed standing water at 1.4 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed; ECBF = East Coast Bays Formation.

ENGEO

LOG OF AUGER HA14

Metlifecare Whenuapai Village
99 Totara Road
Whenuapai, Auckland
19709.000.001

Client : Metlifecare Ltd
Client Ref. : 19709.000.001
Date : 15/12/2022
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 1413
Logged By : LM
Reviewed By : NM
Latitude : -36.7754679
Longitude : 174.6230494

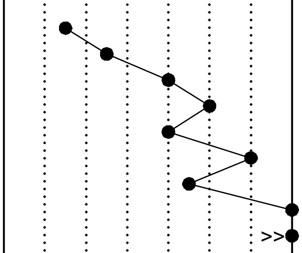
Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0–0.2	TS	OL	[TOPSOIL]				N/A	
0.2–2.0	PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand; orange with grey streaks. Low plasticity. 1.3 m - becomes light grey with orange streaks. 1.4 m - encountered some fine sand.	8, 7	▼ (1.6 m)	M	St-VSt	67/20, 124/54, 118/31, 70/40, 75/28, 123/34
2.0–4.1	PUKETOKA FORMATION	ML	Fine sandy SILT with some clay; light grey with orange streaks and orange mottling. Low plasticity.	6, 5	▽ (2.1 m)	M	St-VSt	96/23, 103/48, 92/51, 98/62, 101/58, 109/54
4.1–4.3	PUKETOKA FORMATION	SM	Silty fine to coarse SAND with trace clay; orange with light grey mottling. Well graded.	4		M	MD	
4.3–5.0	ECBF	ML	Fine sandy SILT; orange with light grey streaks. Low Plasticity.			M	VSt-H	

End of Hole Depth: 5 m
Termination Condition: Target depth

Scala Penetrometer

Blows per 100mm

2 4 6 8 10 12



Hand Auger met target depth at 5 m.
Dip test during drilling showed standing water at 2.1 m depth.
Dip test at end of works showed standing water at 1.6 m depth.
Coordinates assessed using Datanest Mobile App

Elevations estimated from site topo survey (H&G 2022)
TS = Topsoil; N/A = Not Assessed; ECBF = East Coast Bays Formation.

APPENDIX 6:

Laboratory Testing

BGL Babbage Geotechnical Laboratory

Babbage Geotechnical Laboratory
Level 4
68 Beach Road
Auckland 1010
Telephone
E-mail
P O Box 2027
New Zealand
64-9-367 4954
wec@babbage.co.nz

Please reply to: W.E. Campton

ENGEO LTD.
PO Box 33-1527
Takapuna
Auckland 0740

Job Number: 66273#L
BGL Registration Number: 3064
Checked by: JW

22nd December 2022

Attention: **RYAN DINGLE**

ATTERBERG LIMITS & LINEAR SHRINKAGE TESTING

Dear Sir,

Re: 99 TOTARA ROAD, WHENUAPAI

Your Reference: 19709.000.001
Report Number: 66273#L/AL 99 Totara Rd

The following report presents the results of Atterberg Limits & Linear Shrinkage testing at BGL of soil samples delivered to this laboratory on the 16th of December 2022. Test results are summarised below, with page 3 showing where the samples plot on the Unified Soil Classification System (Casagrande) Chart. Test standards used were:

Water Content:	NZS4402:1986:Test 2.1
Liquid Limit:	NZS4402:1986:Test 2.2
Plastic Limit:	NZS4402:1986:Test 2.3
Plasticity Index:	NZS4402:1986:Test 2.4
Linear Shrinkage:	NZS4402:1986:Test 2.6

Borehole Number	Sample Number	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)*
HA02	BAG	0.50	44.2	84	33	51	21
HA13	BAG	0.50	33.1	49	21	28	13

**The amount of shrinkage of the sample as a percentage of the original sample length.*

The whole soils were used for the water content tests (the soils were in a natural state), and for the liquid limit, plastic limit & linear shrinkage tests. The soils were wet up and dried where required for the liquid limit, plastic limit & linear shrinkage tests.

As per the reporting requirements of NZS4402: 1986: Test 2.1: water content is reported to two significant figures for values below 10%, and to three significant figures for values of 10% or greater. Test 2.2: liquid limit, test 2.3: plastic limit, and test 2.6: linear shrinkage are reported to the nearest whole number.

Please note that the test results relate only to the samples as-received, and relate only to the samples under test.

Thank you for the opportunity to carry out this testing. If you have any queries regarding the content of this report please contact the person authorising this report below at your convenience.

Yours faithfully,

Wayne Campton
Key Technical Person
Laboratory Manager
Babbage Geotechnical Laboratory

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation. This report may not be reproduced except in full & with written approval from BGL.

BGL Babbage Geotechnical Laboratory	Job Number:	66273#L	Sheet 1 of 1	Page 3 of 3
	Reg. Number:	3064	Version No:	7
	Report No:	66273#L/AL 99 Totara Rd	Version Date:	July 2022
	Project:	99 TOTARA ROAD, WHENUAPAI		

DETERMINATION OF THE LIQUID LIMIT, PLASTIC LIMIT & THE PLASTICITY INDEX

Test Methods: NZS4402: 1986: Test 2.2, Test 2.3 and Test 2.4

Tested By:	TL / JW	19-Dec-22
Compiled By:	PC	21-Dec-22
Checked By:	WEC	22-Dec-22

SUMMARY OF TESTING

Borehole Number	Sample Number	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification Based on USCS Chart Below
HA02	BAG	0.50	84	33	51	CH
HA13	BAG	0.50	49	21	28	CL

The chart below & soil classification terminology is taken from ASTM D2487-17 [e1] "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)", April 2020, & is based on the classification scheme developed by A. Casagrande in the 1940's (Casagrande, A., 1948: Classification and identification of soil. Transactions of the American Society of Civil Engineers, v. 113, p. 901-930). The chart below & the soil classification given in the table above are included for your information only, and are not included in the IANZ endorsement for this report.

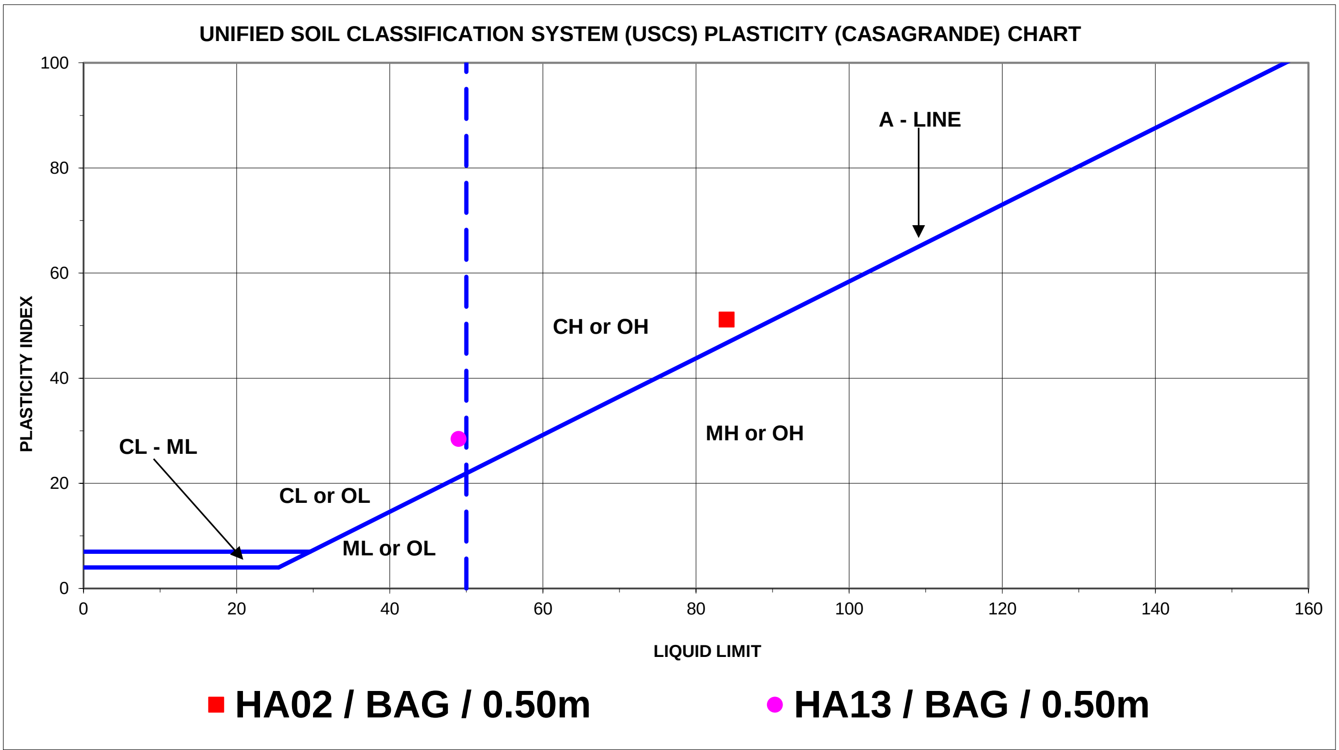


CHART LEGEND

CL = CLAY, low plasticity ('lean' clay)
OL = ORGANIC CLAY or ORGANIC SILT, low liquid limit
ML = SILT, low liquid limit
CL - ML = SILTY CLAY

CH = CLAY, high plasticity ('fat' clay)
OH = ORGANIC CLAY or ORGANIC SILT, high liquid limit
MH = SILT, high liquid limit ('elastic silt')