Geotechnical Assessment Report

JOB NUMBER: 23-0183

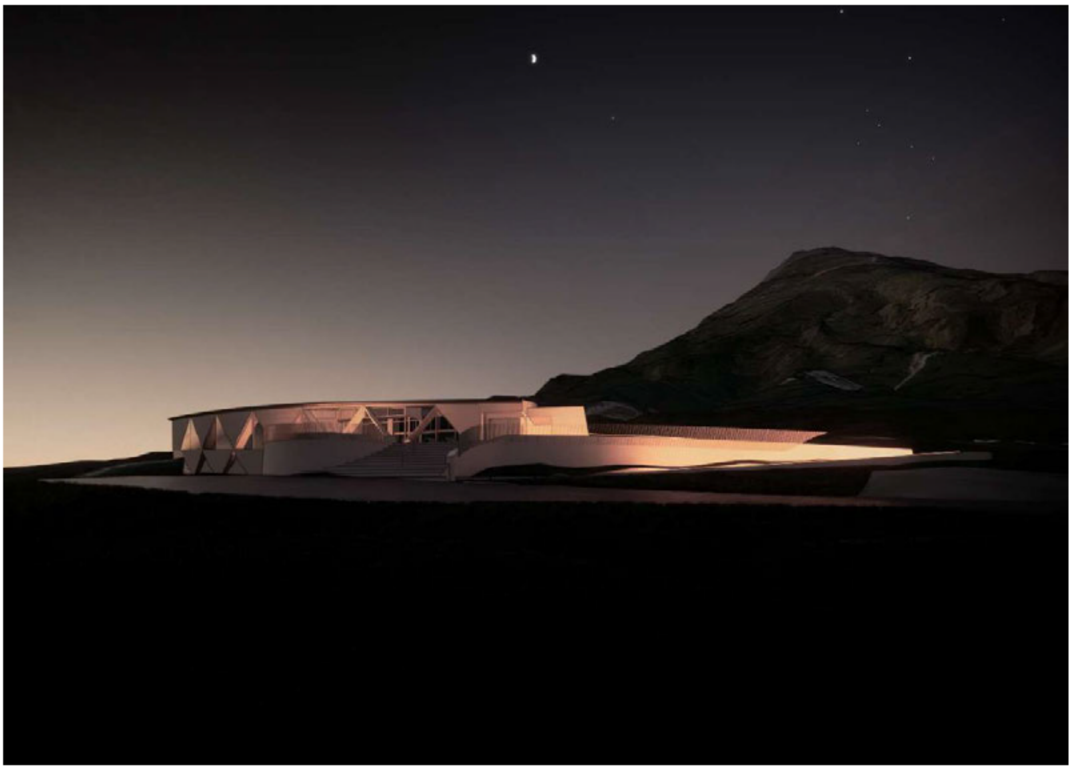
North Taranaki Visitors Centre
PROJECT

Te Kotahitanga O Te Atiawa
CLIENT

Preliminary Design – REV 1
4 July 2023

Geotechnical Assessment Report

23-0183 North Taranaki Visitors Centre



Prepared for: Te Kotahitanga O Te Atiawa **Project no:** 23-0183

Prepared by: [redacted] | Graduate Engineering Geologist,

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Reviewed by: [redacted] | Principal Geotechnical Engineer

Revision	Date	Status	Authorised by:
1	4/07/2023	For Information	[redacted] [redacted] **Principal Geotechnical Engineer** **BSc (Hon) Geology, MEngSc Geotech Engineering, CMEngNZ, CPEng**

EXECUTIVE SUMMARY

RCP Ltd on behalf of Te Kotahitanga o Te Atiawa Trust (Te Atiawa) are planning to construct a new visitors centre at 2879 Egmont Road, Taranaki. BCD Group Limited (BCD) has been requested to provide geotechnical engineering services for the project. This report presents a summary of our assessment and provides foundation design and construction recommendations for Te Atiawa. This report revision is to support detailed design for the development.

	Item	Comments
Our Findings	Liquefaction and lateral spread risk	The risk of liquefaction triggering is considered low based on the soil behaviour type, geological deposition, and depth of groundwater.
	Slope stability risk	Low risk of global instability based on the currently proposed building location, site topography, and soil conditions.
	Expansive soils	The site subsoils are considered non expansive.
	Static settlement	No potentially compressible soils were observed within the building footprint below the subgrade level. Differential settlements are expected to be within tolerable range.
	Bearing capacity	The soil conditions achieve geotechnical ultimate bearing capacity typically between 300 kPa and 400 kPa. A geotechnical ultimate bearing capacity of 300 kPa equivalent to NZBC “Good Ground” is considered appropriate for preliminary design purposes.
Recommendations	Earthworks and Site Preparation	Earthworks involving cut and fill up to 1.5m are anticipated to form the building platform at the proposed levels. There is likely to be some existing fill beneath the existing building which will require rework and compaction. All engineered fill should be placed and compacted in general accordance with NZS4431:2022. Compaction targets for site-won fill should be confirmed through laboratory testing and documented within a site -specific compaction specification prepared prior to construction. Any permanent cut and fill batters should be limited to 1V:3H.
	Foundations	The building can feasibly be supported on either shallow foundation system or a piled foundation system. For shallow foundations, assuming earthworks and site preparation are undertaken as per this report, an ultimate bearing capacity of 300 kPa can be adopted for preliminary design. Alternatively, foundation capacities can be determined in accordance with B1/VM4 using the assessed soil parameters. A preliminary method for determining the piled foundation capacity is also provided.
	Construction monitoring	Construction monitoring and observations are required during construction to confirm that the ground conditions are in line with this report and that earthworks compaction criteria is achieved. Preliminary observation hold points are provided.

Contents

1 INTRODUCTION

RCP Ltd on behalf of Te Kotahitanga o Te Atiawa Trust (Te Atiawa) are planning to construct a new visitors centre at 2679 Egmont Road, Taranaki. BCD Group Limited (BCD) has been requested to provide geotechnical engineering services for the project. This report presents a summary of our assessment and provides foundation design and construction recommendations for Te Atiawa.

The scope of this assessment was carried out in accordance with our Short Form Agreement dated 19 April 2023. However, due to ongoing changes to the size and form of the building additional detailed geotechnical analysis may yet be required and therefore this report is intended for information only. A revision suitable for supporting a Building Consent application will be issued once key elements of the design have been finalised.

2 PROJECT DESCRIPTION

It is proposed to demolish the existing building and construct a new visitors centre. Scheme drawings are included in Appendix A which show the overall concept. The current version includes a single storey building that will house the DOC visitors centre, a commercial café, and Manaaki space.

At this stage the finished floor level is proposed between 940.0m RL and 940.6 m RL, and cut and fill earthworks are required to form the building platform. The earthworks volumes are not yet confirmed but cuts and fill are likely to be in the order of 2.5m deep/thick.

Several smaller ancillary structures are proposed as part of the redevelopment.

3 SITE OVERVIEW

3.1 Site Description

The site is located at the road end of Egmont Road on the north-eastern side of Mt Taranaki (refer to Figure G-01 in Appendix B). The subject site is a collection of titles, the legal descriptions and usage follow:

- Part OBJECTID 29946; ID 4678223 - Majority of roadway and carparks.
- Part PT Section 2 Block XIV Egmont SURD; OBJECTID 16195; ID 4644708 – Visitors centre building and some surrounding area.
- Egmont National Park Survey Office Plan 10039 - Surrounding national park.

The existing north Egmont visitors centre is at approximately 940 m RL with the lower carparking areas being at slightly lower elevations. The ground surface is generally gently sloping towards the northeast.

The site is within the jurisdictions of Taranaki Regional Council (TRC) and New Plymouth District Council (NPDC). The site does not have connections to Council reticulation for wastewater and stormwater. It is likely that stormwater will continue to be collected for reuse on within the building with overflows disposed by soak pits. Wastewater is presently directed to the existing dispersal field up slope to the south of the existing visitors' centre.

3.2 Desk Study Geotechnical Assessment

BCD was engaged to complete a preliminary geotechnical desktop study[1] at the early stages of the project prior to the site-specific investigation works. The assessment focused on providing high level geotechnical constraints for the site however included a thorough desktop review of the site history, published geological maps, and known geological hazards associated with Mt Taranaki. The desktop study assessment letter is included in Appendix D.

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[1] BCD Group, 2023. 23-0183 – North Taranaki Visitors Centre Redevelopment – Preliminary Geotechnical Letter, dated 16 March 2023.

4 GEOTECHNICAL INVESTIGATIONS

4.1 Walkover Observations

A site walkover was completed by a BCD geotechnical engineer on 23 February 2023. The purpose of the walkover was to observe the site profile and to check for any soil or rock exposures which might assist with determining the site conditions and geotechnical assessment.

The site is generally positioned on top of a prominent ridgeline and is gently sloping to the northeast. To the southwest, and some distance away, is a large steeper slope that runs sub-parallel with the main ridge line. The upper flanks of the gulley slope appear on average to be approximately 45 degrees, gradually reducing in slope gradient down to the base of the gulley. The site is generally grassed or vegetated beyond the building footprint and there were no noteworthy geomorphic features observed.

Some small soil exposures were observed beneath the viewing platform and along the nature walk track, which indicate that the natural surface soils may comprise tephra ash fall deposits comprising silts with variable sands and gravels.

4.2 Subsurface Investigations

The following investigations were undertaken to evaluate the subsurface conditions at the site between 8 May 2023 & 18 May 2023.

- 5 No. hand augers (HA) up to 3m depth.
- 3 No. machine borehole (MBHs) up to 10 m depth.

All investigations were logged in accordance with the NZGS guidelines "field description of soil and rock" 2005. The site investigations plan and Investigation data can be found in Appendix C.

It is noted that 3 No. hand augers were completed in an area where a roadway was originally proposed. This road extension is no longer part of the development plan. 2 No. hand augers (HA01 and HA02) were also previously completed as part of the high-level assessment in February 2023, however these are away from the proposed development area and are not considered in detail.

4.2.1 Hand Auger Boreholes

Shear strength testing was attempted within fine grained soils. Scala Penetrometer testing was undertaken in sandy soils not suitable for shear vane testing.

During the testing several hand auger holes could not obtain target depths due to the density of the Maero Debris Flow Formation.

4.2.2 Machine Boreholes

Machine boreholes were completed by Hardcore drilling using HQ triple tube coring techniques. SPTs were undertaken at regular 1.5m depth intervals in all boreholes.

5 SUBSURFACE CONDITIONS

Three geotechnical cross sections are included in Appendix C and depict the soil profile across the site. The sections have been developed from the available investigations. A number of separate geotechnical units have been identified and are outline din the following sections.

5.1 Geotechnical Units

The investigations indicate that the existing subsurface conditions beneath the proposed building area comprise of the following geotechnical units:

- Unit 1: Fill
- Unit 2: Stiff SILTs
- Unit 3: loose to medium dense Silty Gravelly Sand
- Unit 4: dense to very dense Sandy Gravels

It is noted that the soil profile comprised highly interbedded silts, sands, and gravels. There was sometimes a high percentage of core loss during the deeper machine borehole drilling which makes it difficult to accurately delineate geological units. To simplify the engineering assessment the non-Tephra soil units are delineated by engineering strength based primarily on SPT testing values.

5.1.1 Unit 1: Fill

Fill was encountered in some site areas only. The fill observed typically comprises medium dense silty gravelly sands and is likely to be material reworked from the Maero Formation mixed with GAP40 and construction debris including broken brickwork.

The greatest thickness of fill was encountered at MBH01 location. Up to approximately 2.5m of fill was observed. It is considered this fill layer is likely to be an isolated wedge associated with formation of the road accessway.

The fill thickness at MBH02 was approximately 0.7m thick. This fill is likely associated with the paved driveway and installation of underground services in the vicinity of the borehole.

It is considered that there is a high probability of being some existing fill beneath the existing building footprint, particularly given there have been older historical buildings in the area.

5.1.2 Unit 2: Stiff Silts

Thin layers of stiff silts were encountered within the soil profile. The soils were described as stiff, low plasticity, silts with some sand and gravel and are representative of Taranaki Tephra Deposits. A consistent band of tephra was encountered at a depth range typically between 9m and 10m below surface level across all three boreholes. A shallow layer of tephra ash was also encountered in MBH03 from 2.6m to 2.9m bgl, however this did not extend across the other boreholes.

5.1.3 Unit 3 - L to MD Silty Gravelly Sand

Loose to medium dense silty gravelly sands were encountered from surface and at varying depths within the boreholes. These are identified in layers of varying thickness interbedded between the other soil units. Refer to geological sections for typical depths and lateral extents.

This unit was typically distinguished by the distinctive yellowish-brown colour and inclusion of silt components. It is inferred as part of the Maero Debris Formation. This unit's core recovery was generally good. SPT results were variable but typically between 8 and 30.

5.1.4 Unit 4 – MD to VD Sandy Gravel

A consistent layer of medium dense to very dense Sandy Gravels was encountered across the site starting from approximately 2m to 4m depth (elevation 935 m RL to 937 m RL) and being about 3 to 4m in thick. This unit was typically distinguished by the greyish colour and more gravelly nature. There was generally a higher percentage of core loss throughout this unit. The SPT results were again variable, however typically between 23 and 50+.

5.2 Groundwater

Groundwater measurements were made in the vicinity of the proposed building during MBH drilling. The MBHs were dipped between 15 and 25 minutes after completion of drilling. Therefore, the dipped water levels observed might not accurately represent a 'static' groundwater table due to the introduction of water during drilling.

Prior to and during the investigations the site experienced significant rainfall which is considered typical for late autumn. Groundwater levels may be higher following periods of prolonged rainfall or significant snow melt. However, the site is positioned on top of a prominent ridgeline, and it is expected that the 'static' groundwater table is generally controlled by the significant gulley features to the southeast and northwest of the site.

Notwithstanding the above, the groundwater level was dipped at depths of between 6 m and 9 m bgl at the time of the investigation in May 2023. For assessment purposes, the groundwater is conservatively assumed to be present at about 6m below ground surface.

It is noted that groundwater was encountered within HA102 at a depth of 2.3m bgl only across the shallow hand auger investigations. When compared with dipped values from the borehole testing this water level appears to be an outlier and is more likely representative of an old flow path or seep created during the construction of the carpark and is considered not indicative of the overall site groundwater table.

6 GEOTECHNICAL ASSESSMENT

This section outlines our geotechnical assessment of the site with respect to geotechnical natural hazards and the assessment of ground conditions with respect to the proposed development.

The following recommendations and opinions are based upon data from observations made on-site, and the investigations undertaken. Inferences about the nature and continuity of subsoils away from the exploration holes are made but cannot be guaranteed.

6.1 Soil Design Parameters

The soil parameters detailed in the table below have been derived based on the in-situ strength testing results and BCDs experience with similar ground conditions. These parameters should be used by the geotechnical and structural designers for the specific engineered design elements. The values presented are considered conservative but reflect the amount of field testing completed.

Table 1: Soil Design Parameters

Soil unit	Unit weight, γ (kN/m³)	Effective cohesion, c' (kPa)	Effective friction angle, ø' (degrees)	Undrained shear strength, su (kPa)
Unit 1 - Existing Fill	18	0	28	-
Unit 2 – Stiff Silts	17	3	28	80
Unit 3 - L - MD Silty Gravelly Sand	18	0	34	-
Unit 4 – MD - VD Sandy Gravel	19	0	38	-
Site-won Engineered Fill	19	0	38	-

6.1.1 Site-Won Engineered Fill

In the vicinity of the proposed building the upper geotechnical unit is dominated by gravelly sands and sandy gravels. This material is relatively well graded with a maximum stone size in the coarse gravel range and includes subangular material. In places the material may include varying amounts of silt.

It is anticipated that this material will be suitable as engineered fill. However, it should be noted that the insitu material is relatively loose and therefore some reduction in volume should be expected where this material is well compacted.

Typical soil strength values for this compacted material in included in Table 1.

6.2 Seismic Soil Behaviour

6.2.1 Site Subsoil Class

Site testing and nearby borehole data from GNS that the site technically falls under the site subsoil class D due to the absence of competent rock within the upper 100m or so. However, it is considered unlikely that the volcanic deposits would behave as per a site class D site.

The site sits on the flanks of Mount Taranaki which is a composite volcanic cone formed from repeat volcanic eruptions (lahar flows, tephra, and lava flows). Based on the expected absence of 'soft' soils throughout the volcanic profile and interbedded nature of the underlying granular soil and rock which forms the volcanic environment, the overall site performance is considered more likely to behave like a class C 'shallow soil site'.

For design purposes a conservative worst case should be adopted. a soil class C should be considered for geotechnical design elements. For structural design elements a site subsoil class D should be adopted.

6.2.2 Soil Liquefaction

The seismic design parameters for geotechnical purposes are presented in Table 1. These are selected based on the MBIE Earthquake Geotechnical Engineering Practise Module 1 (2021) which conservatively assumed site class C seismic actions. The proposed building is an Importance Level 2 structure with a design life of 50 years.

Table 2: Seismic Assessment Parameters

Importance Level	Design Life (years)	Earthquake Magnitude	Limit State	Annual Exceedance Probability (AEP)	PGA (g)
2	50	6.2	SLS	1/25	0.07
			ULS	1/500	0.28

The following considerations are made by BCD with respect to the risk of liquefaction effects for the site:

- The conservative ground water assumed for assessment is at 6m below existing surface level. Therefore, there is a minimum 6m thick non-liquefiable crust present beneath the future building.
- Liquefaction occurring within soil layers below 6m depth would incur only minor to negligible effects at the surface.
- The subsurface soils were deposited in a high energy environment and are therefore less susceptible to liquefication triggering.

For these reasons, we consider the risk of liquefaction triggering at the site to be very low. Based on the liquefaction risk and the site topography, lateral spread is also not considered to be a risk for the building. Therefore, no measures are required for the foundations or to the site to minimise the risk against possible liquefaction effects.

6.3 Slope Stability

The proposed building site gently slopes down toward the northeast, following the natural topography of the surrounding area and is bound on the east by a large gully feature. Currently the proposed building is approximately 15m back from the edge of the gully. Based on our experience with the local soil conditions and the site topography, the site is considered to have adequate global stability against slope failure affecting the proposed building platform. This is on the assumption that localised instability is managed appropriately through suitable cut and fill batter slopes, benching, and retaining walls.

If building layout is revised and the building moves to the east beyond the existing water tanks, then further specific slope stability will be required as the ground profile tends to steepen up towards the east.

6.4 Soil Expansivity

The underlying soil conditions at the anticipated foundation level comprise predominantly silty gravelly sands. The silt matrix is derived from volcanic tephra deposits which are known locally to not experience shrink and swell properties based on their mineralogy. Being predominantly gravelly sand, the foundation soils are not considered to be expansive as defined in NZS3604:2011.

6.5 Static Settlement

The investigations did not identify any potentially compressible soils within the influence zone of the proposed building. The Tephra deposits are thin, stiff, and generally at depth. The other soils are interbedded loose to very dense sands and gravels and are not considered to be compressible. Potential settlement within granular soils is often referred to as 'immediate' settlement, as pore pressures can dissipate quickly. Settlement due to placement of fill on the site granular soils will effectively be completed before the building construction is finished. Any post-construction settlement effects will be due to building loads only.

It is also noted that the proposed building largely overlaps the current visitors centre location and will therefore have been effectively preloaded. Buildings have been present in this general area for more than 100 years.

For the above reasons the risk of any significant post-construction settlement affecting the future building is very low assuming the earthworks and building platform preparation are completed in accordance with this report. Long term total and differential settlements are expected to be within accepted tolerances for the building type.

6.6 Bearing Capacity of Natural Soils

Bearing capacity has been estimated using the B1/VM4 guidelines adopting the soil parameters presented in Table 1 and a range of preliminary foundation sizes. The B1 calculations using an assessed effective friction angle indicate that the insitu soils achieve ultimate bearing strength typically between 300 kPa and 400 kPa.

A bearing capacity sensitivity check has been completed using SPT correlation presented in Bowles (1997) foundation Analysis and Design and the original chart developed by Terzaghi and Peck (1976). The chart (Bowles Figure 4-7) provides allowable bearing capacity for a range of footing sizes that will limit settlement to within 25mm.

Assuming an average SPT N value of 8, an allowable bearing capacity of approximately 125 kPa can be adopted for surface loaded footings with settlement limited to 25 mm. Allowable bearing capacities include a factor of safety of 3 indicating an ultimate bearing capacity of about 375 kPa, which is consistent with the values calculated using B1/VM4. Greater bearing capacity would become available as footings are embedded.

For preliminary design, a geotechnical ultimate bearing capacity of 300 kPa equivalent to NZBC "Good Ground", is considered appropriate. Higher bearing capacities may be available and can be confirmed for specific foundations once the building loads and foundation details have been confirmed.

7 RECOMMENDATIONS

The following sections provides general recommendations suitable for preliminary design of the proposed building. These are based on the current revision of the development plans. The recommendations are preliminary and may require review once the final form of the building has been documented.

Prior to lodging for Building Consent, the final consent plans will need to be reviewed by a member of the BCD geotechnical team.

7.1 Earthworks and Site Preparation

Cut and fill earthworks are required to form the building platform. At this stage it is assumed the building FFL will be between 940.0 m RL to 940.6 m RL. An estimate of up to approximately 2.5m of cut and/or fill may be required to form the building platform based on the available site contour data.

There is likely to be some uncertified fill beneath the existing building. Based on investigation close to the building footprint this material is likely to be gravelly sands and sandy gravels but may include lesser amounts of silts. Generally our testing indicates the sands and gravels are loose, and these will need to be reworked and compacted where the existing fill is to support loads.

7.1.1 Batter slopes

It is recommended that unsupported permanent cut batter slopes are limited to 1V:3H for slopes up to 2m in height. Alternatively designed retaining walls could be utilised to support cuts.

All filling is expected to be supported by the building structure; therefore, no exposed fill batters are expected. However, if permanent fill batters are required for landscaping areas, a maximum slope gradient of 1V:3H is recommended for preliminary purposes.

7.1.2 Fill Compaction

It is expected that site won sandy gravels/gravelly sands can be stockpiled and reused as engineered fill on site.

All engineered fills should be placed and compacted in uniform layers not exceeding 250mm thick and in general accordance with NZS4431:2022. However, compaction targets for site-won materials should be confirmed within a site-specific compaction specification prepared prior to construction. Laboratory standard compaction testing with solid density tests are recommended as part of preparing this specification.

A bulking factor of 0.90 should be adopted for preliminary cut and fill balance purposes i.e. every 1m^3 of in-situ soils excavated will recompact down to 0.9m^3.

Existing uncertified fill will need to be inspected and approved by the geotechnical engineer. It may be possible to compact the fill in place using a heavy roller and avoid significant remedial excavation works. Where this is not effective the thickness of the uncertified fill may need to be reduced to a maximum of 0.6m and the excess treated as new fill as outlined above.

7.1.3 Earthworks Observations

Preliminary hold points for certifying bulk filling works are as follows:

- **Subgrade Inspection** – Confirm subgrade conditions comprise natural soils beneath proposed building footprint.
- **Existing Fill appraisal** – If existing fill is present at subgrade level within building footprint. Confirm in-situ compaction criteria specified achieves the targets. Otherwise confirm removal of unsuitable soils.
- **Subsoil Drainage** – Confirm drainage is installed behind building walls in accordance with the design.
- **Compaction of Engineered fill –** Confirm compaction criteria is achieved in accordance with the earthwork's specification.

The earthworks specification should be prepared prior to the production earthworks to not cause project delays.

7.2 Foundations

Based on an assessment of the site conditions it is considered acceptable to support the proposed building on either:

- Shallow Strip and Pad foundation system bearing on natural soils or engineered fill.
- Shallow and/or deep piled foundation systems

7.2.1 Shallow Strip and Pad Foundations

A shallow foundation solution comprising strip and pad footings is suitable provided that earthworks and foundation preparation are undertaken in accordance with the recommendations of this report.

For preliminary design, an ultimate bearing capacity of 300 kPa can be adopted for design of shallow foundations that have centric, vertical loads. Specific foundation bearing capacities can be determined using B1/VM4 and the values outlined in Table 1.

Where footings are founded on compacted fill the soil strength values for site-won engineered fill can be used where the thickness of fill below the foundation is at least two-times the footing width. Where the relative thickness is less, specific bearing capacity will need to be determined using methods developed for two-layer systems (Meyerhof & Hanna, or similar).

7.2.2 Piled Foundations

A piled foundation is considered suitable depending on the designers' requirements. Specific pile design can be completed in accordance with B1/VM4 using the values outlined in Table 1.

Based on the geotechnical sections (Appendix C) there is a thick band of sandy gravel at a depth of approximately 5m. SPT N-values in this material range from 23 to 50+ indicating it is dense to very dense and would be suitable as a founding layer for deep heavily loaded piles. Piles should be founded at least three diameters into this unit to mobilise the full pile strength.

The following empirical correlations between SPT N-Values and ultimate pile capacity was developed by Meyerhof and can be used for preliminary design based on end bearing only:

$$q_f = 40N\,(D_b/B)\ \text{kN/m}^2 \quad \text{(limited to 400N).}$$

Where D_b is the embedment depth into the founding (strong) layer, and B is the pile diameter (A_b = end area).

Assuming a 250mm diameter pile founded 750mm into the dense sandy gravel at 5m depth with N=23, the ultimate capacity would be approximately 12,800 kN/m^2, indicating the ultimate pile strength would be q_fA_b = 600kN (approx.). Shallower soils have average N-values of 8 and where the same sized pile is founded at 2m the ultimate capacity would be 2,500 kN/m^2, and the strength would be 120 kN (approx.)

For preliminary design the settlement of the pile can be assumed to be 1% of the pile diameter.

8 SAFETY IN DESIGN

This section outlines the safety in design considerations with respect to geotechnical matters for our current understanding of the project. We recommend that these are incorporated into the project risk register.

The Principal and Contractor(s) must comply with the Health and Safety at Work Act (2016). If controls are required it is the responsibility of the contractor to implement the controls, or to satisfy the project manager and any applicable consenting authorities that the alternative addresses the Hazard and reduces the Risk to an acceptable level.

WorkSafe New Zealand has produced a Good Practice Guideline for Excavation Safety (2016). If the controls in Table 3 differ from the WorkSafe Guideline then Table 3 shall have precedence, unless further assessed by a Chartered Professional Engineer on behalf of the contractor.

Table 3: Safety in Design Summary

Hazard	Initial Risk	Controls	Residual Risk
Excavations Cut and fill earthworks up to 1.5 m to create the building platform.	**Medium**	Cut batters to 1V:3H.	**Low**
Existing Buried Fill Existing fill / unknown objects may be present within building area from historical buildings and demolitions.	**Medium**	Site specific earthworks compaction specification prepared prior to construction. Geotechnical engineer appraisal of existing fill. Compaction trial using heavy equipment is recommended to confirm compaction targets can be achieved.	**Low**
Groundwater Not expected to be encountered during site works.	**Low**	Review controls should groundwater be encountered.	**Low**
Heavy Plant / Stockpiles Heavy plant operating above excavations or steep slopes. Stockpiles placed on excavations or steep slopes.	**Medium**	Do not stockpile materials above existing cuts. Heavy plant not to operate in close proximity to existing cuts	**Low**
Services There is high probability of services that served the existing buildings to be found. Potential live for electrical cables.	**High**	The contractor shall obtain service plans and locate all services prior to commencing works. Ensure all old redundant services are in-active and cut off prior to demolition.	**Low**
Soil Contamination BCD is not aware of any soil contamination assessment of this site. Cultural impact of introducing new soil material to the site.	**Medium**	Obtain environmental advice is if potentially contaminated soil or groundwater is found. Re-use site won material where possible. Co-ordinate introduction of new materials to site with Te Atiawa.	**Low**

9 REFERENCES

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MBIE (2021). Ministry of Business, Innovation & Development & New Zealand Geotechnical Society (NZGS), Earthquake Geotechnical Engineering Practice, Modules 1 – 6.

MBIE AS/VM B1 (2019). Ministry of Business, Innovation & Employment. Acceptable Solutions and Verification Methods for New Zealand Building Code Clause B1 Structure. Amendment 18.

Neall V.E. (1972). Tephrochronology and Tephrostratigraphy of western Taranaki (N108-109, New Zealand). Report No. 4, Volume 15, New Zealand Journal of Geology And Geophysics.

Townsend, D., Kamp, P., & Vonk, A. (2008). Geology of the Taranaki area. Institute of Geological & Nuclear Sciences.

10 REPORT LIMITATIONS

The recommendations and opinions made in this report are based upon data from observations made on-site, conducted hand augers, and in-situ soil strength testing at discrete locations. Inferences about the nature and continuity of subsoils away from the exploration holes are made but cannot be guaranteed. Actual conditions onsite may vary more gradually or abruptly than that inferred from the investigations. Steps can be taken to reduce the likelihood of unexpected conditions arising onsite. As the soil conditions are created and vary by natural processes and human activity, the report is based on soil conditions at the time of the investigation. Soil conditions onsite can change, particularly after long periods of time from the date of investigation.

This report has been prepared for our client for their purposes and the regulatory authority in relation to the consent application within the scope of this report. It is based on our understanding of the proposed development. Should any changes to the nature of the development occur, BCD should be asked to provide comment on the ongoing applicability of recommendations made in this report. It is not to be relied upon or used out of context by any other person without reference to BCD Group Ltd. The reliance by other parties on the information or opinions contained in this report shall, without prior review and agreement in writing, be at such parties' sole risk. To avoid misinterpreting this report, we recommend that the assistance of geotechnical professionals familiar with the project and scope of this report is maintained.

Engineering design and/or engineering design recommendations have been made based on the information provided to BCD. Should these recommendations be used for construction, BCD are to sight approved Building Consent drawings to ensure compliance with recommendations made within this report. If a Producer Statement 4 or construction observation is required from BCD (see BCD report and/or consent requirements from council), we are to be contacted prior to construction to outline appropriate inspection milestones.

This report covers geotechnical considerations only. We recommend the proposed works be checked against current District and Regional Council plans or checked by a registered planner

APPENDIX A - Selected Project Drawings

A11 FLOOR PLAN

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FOR INFORMATION

FOR CO-ORDINATION
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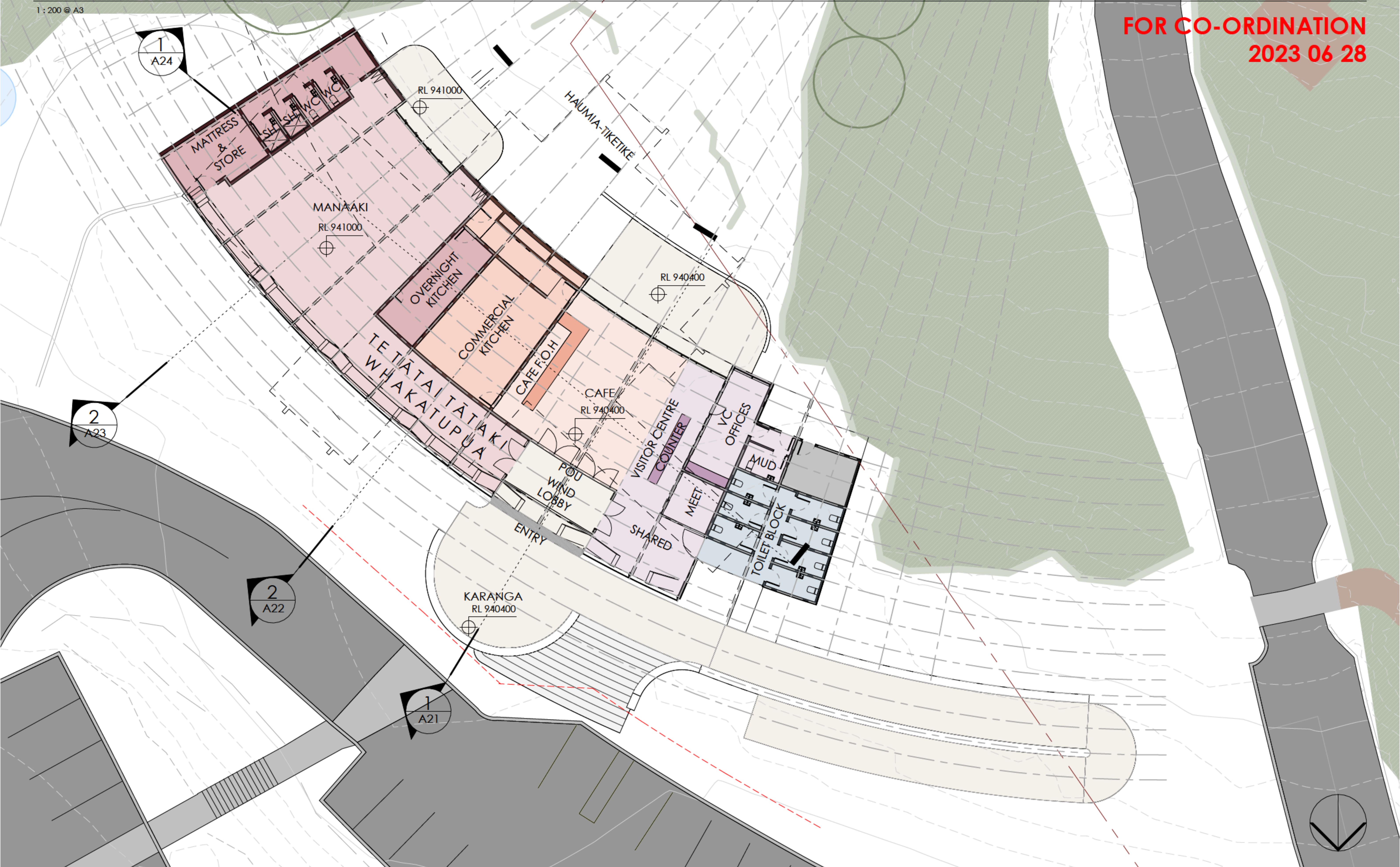

NORTH TARANAKI VISITORS CENTRE, TE PAPA-KURA-O-TARANAKI
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| **TOA** | TĀMAKI MAKAURAU OFFICE ARCHITECTURE LTD

1 : 200 @ A3

FOR INFORMATION

FOR CO-ORDINATION
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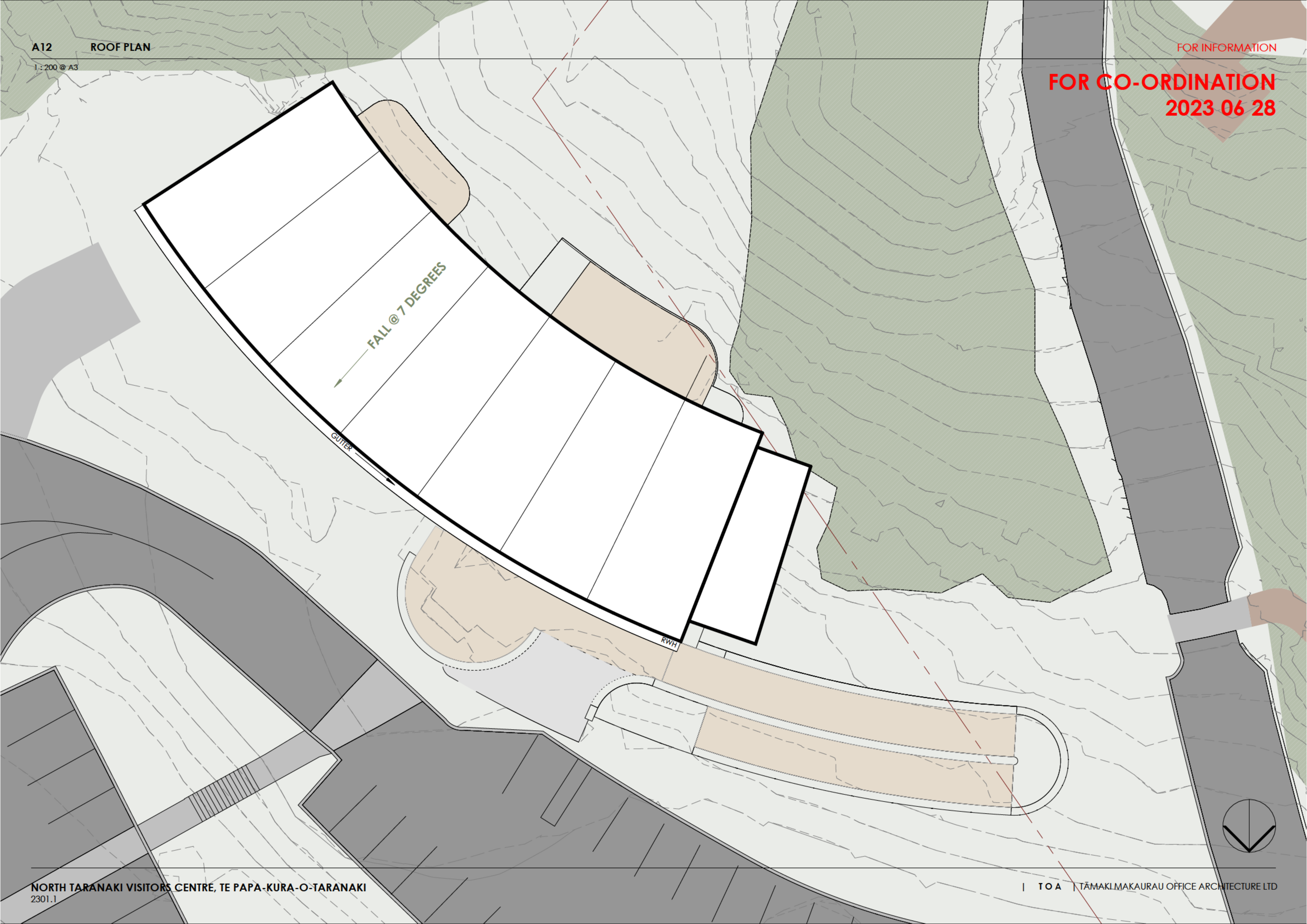

A13 **FLOOR PLAN - UNDERCROFT**

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FOR INFORMATION

FOR CO-ORDINATION
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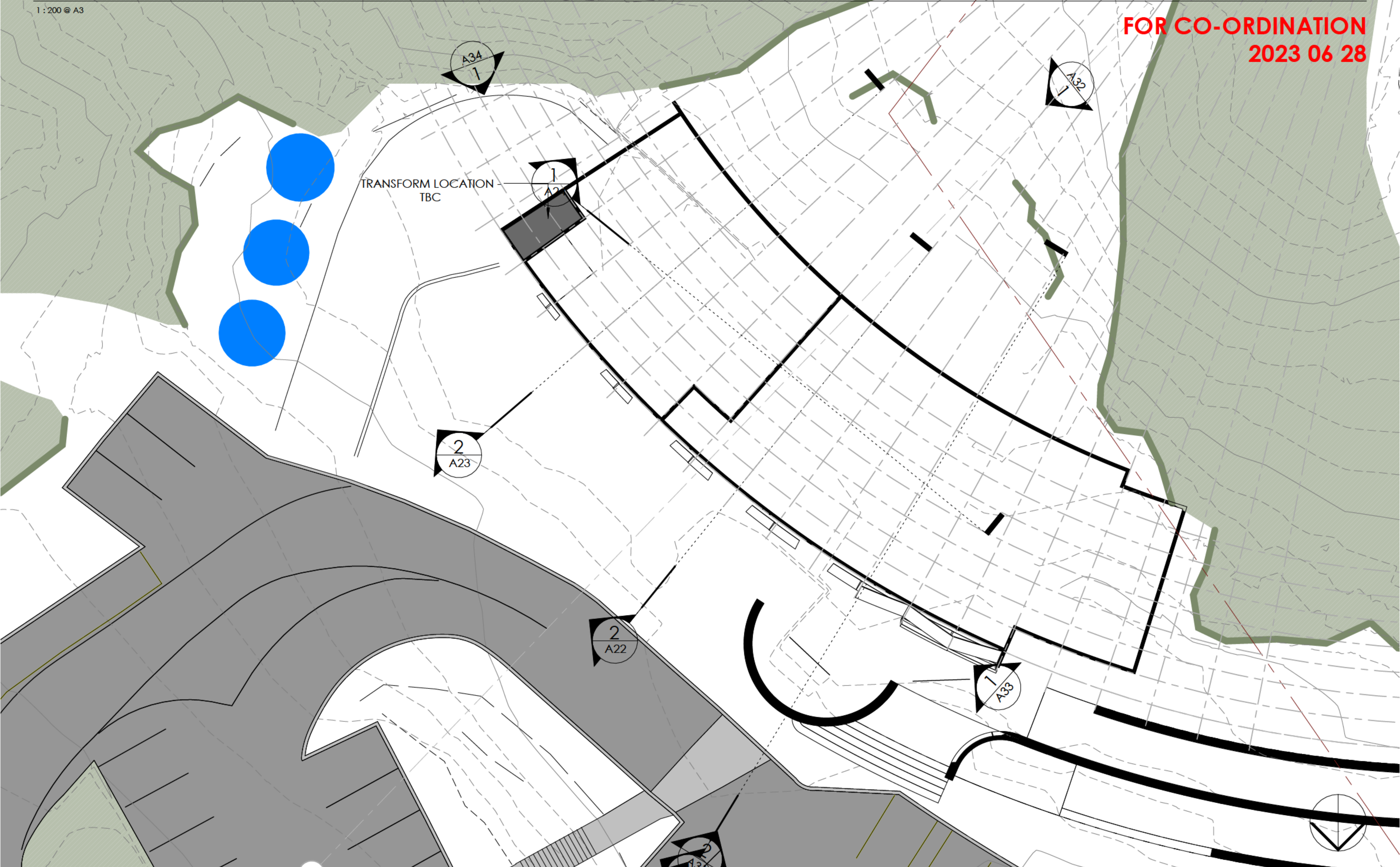

NORTH TARANAKI VISITORS CENTRE, TE PAPA-KURA-O-TARANAKI
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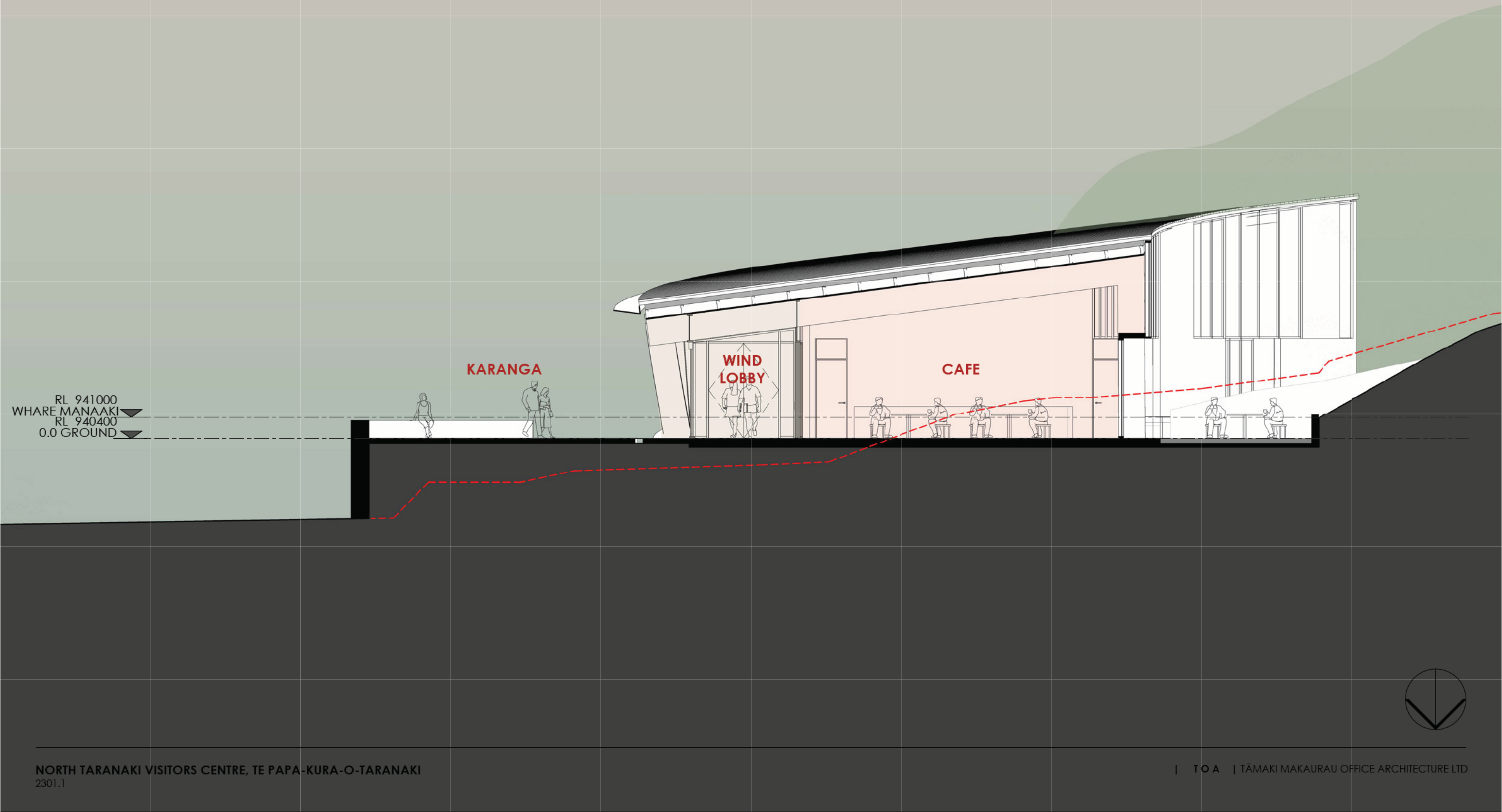
FOR INFORMATION

FOR CO-ORDINATION
2023 06 28

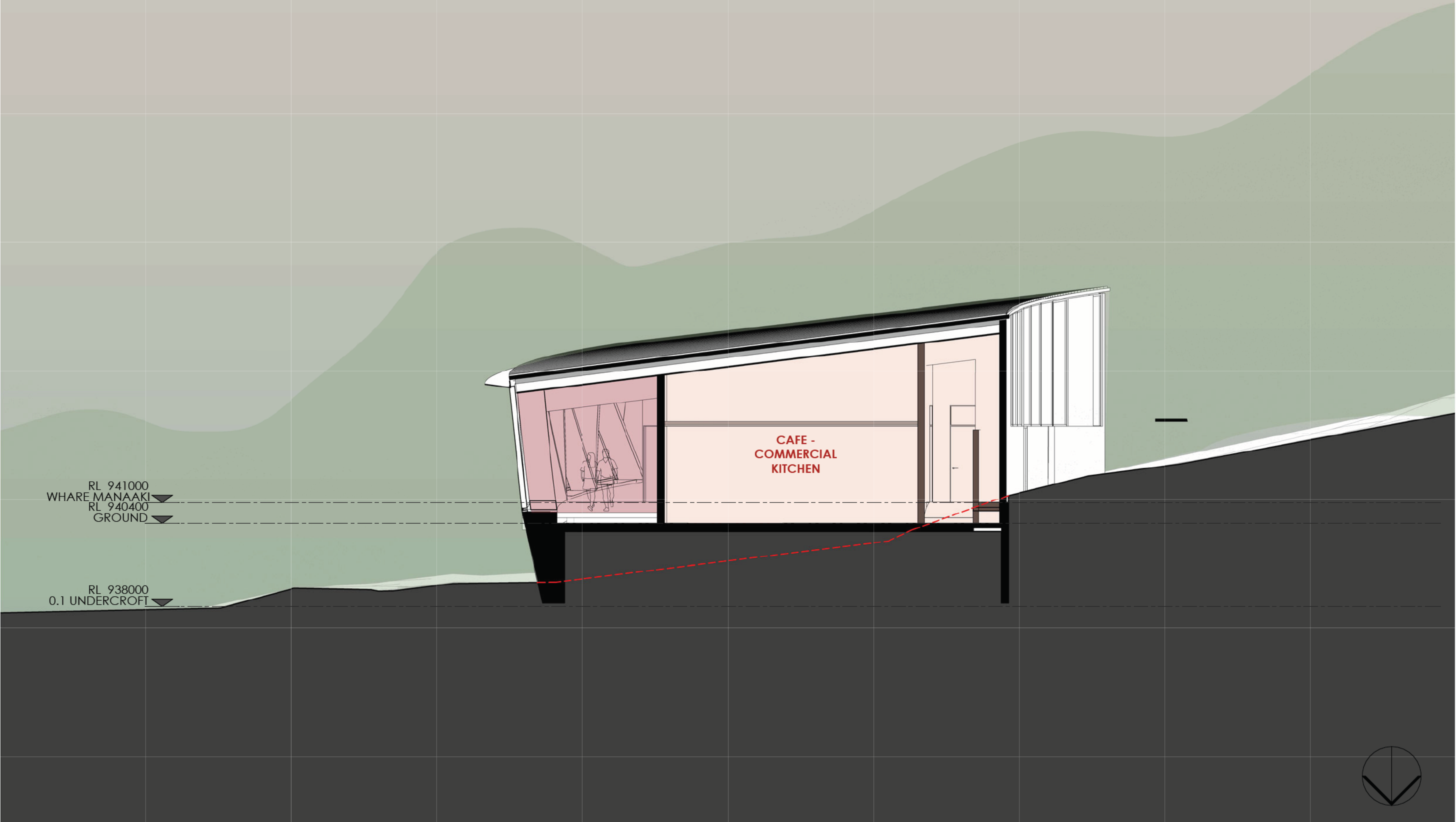


A22 SECTION

1 : 100 @ A3

FOR INFORMATION

**FOR CO-ORDINATION
2023 06 28**

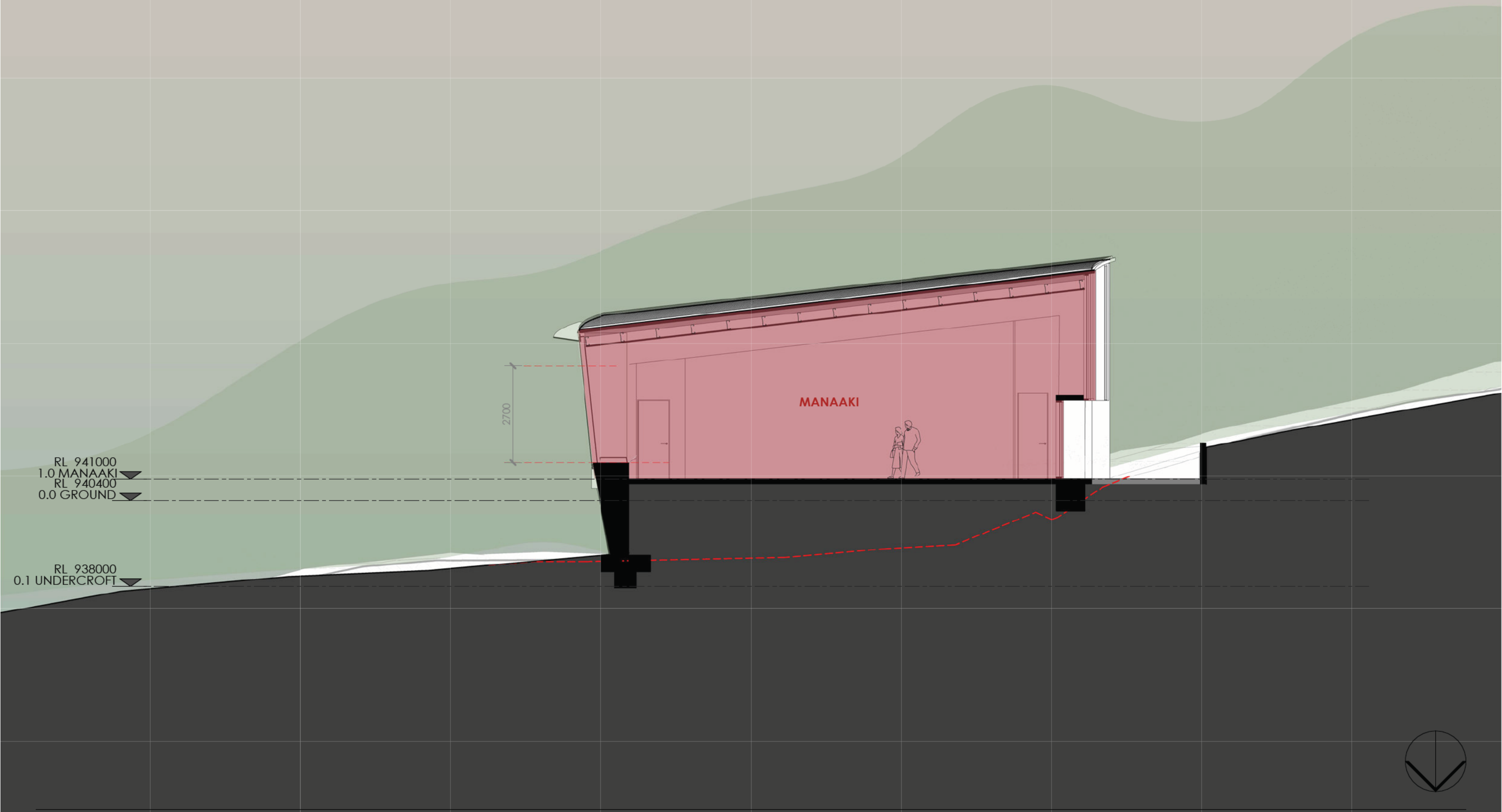


A23 SECTION

1 : 100 @ A3

FOR INFORMATION

FOR CO-ORDINATION
2023 06 28

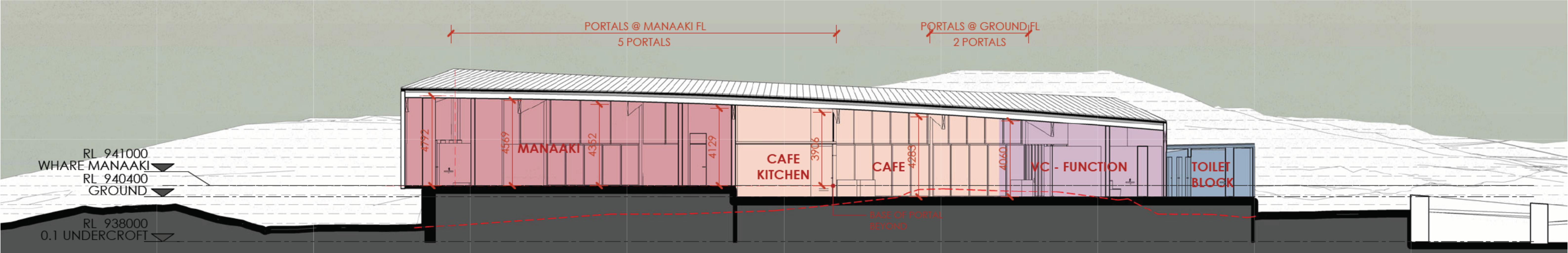


A24 **SECTION**

FOR INFORMATION

1 : 200 @ A3

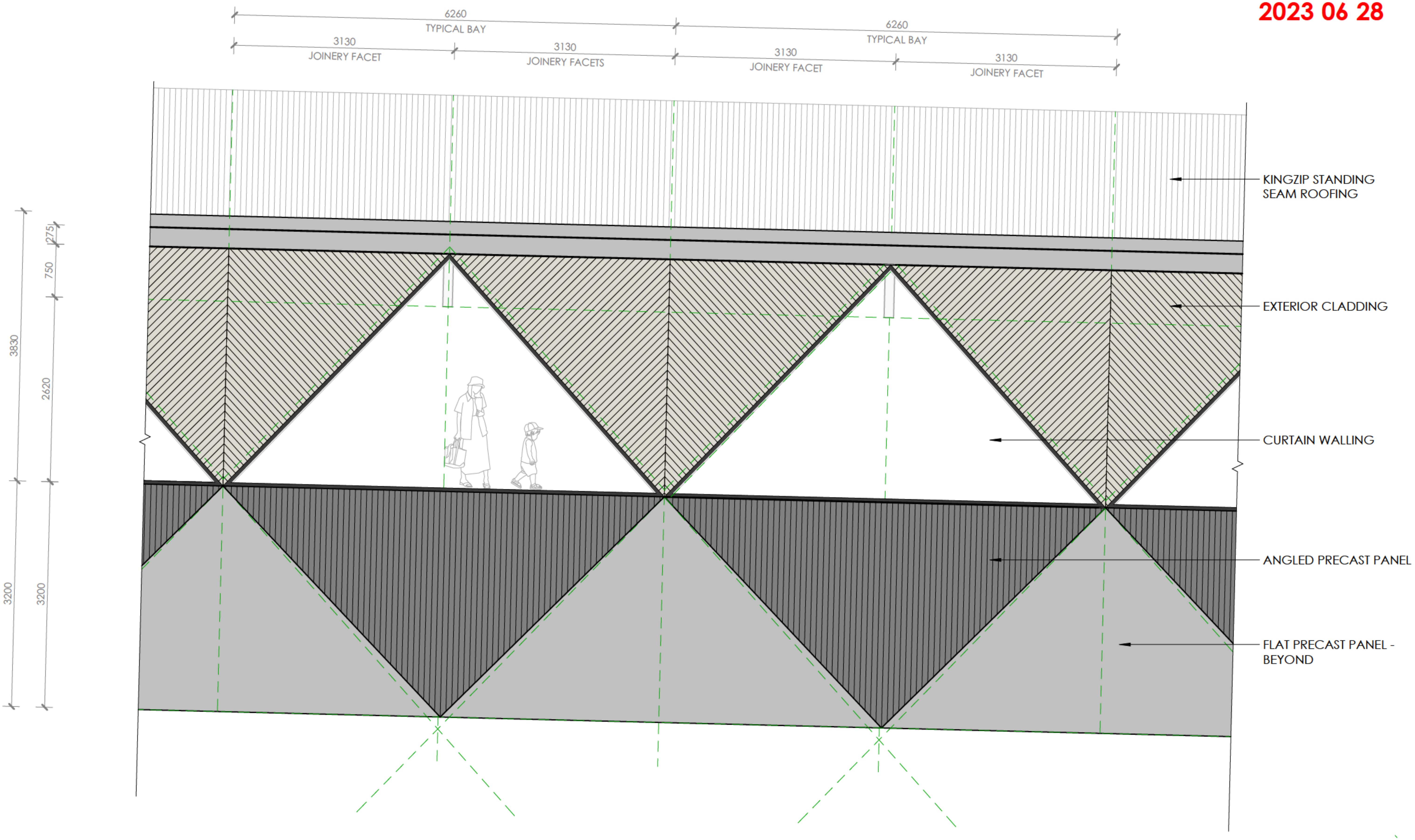
FOR CO-ORDINATION
2023 06 28



NOTE:
ALL DIMENSIONS TO UNDERSIDE OF PORTAL
ON MAUNGA SIDE FROM ASSOCIATED
FLOOR LEVEL

1 : 50 @ A3

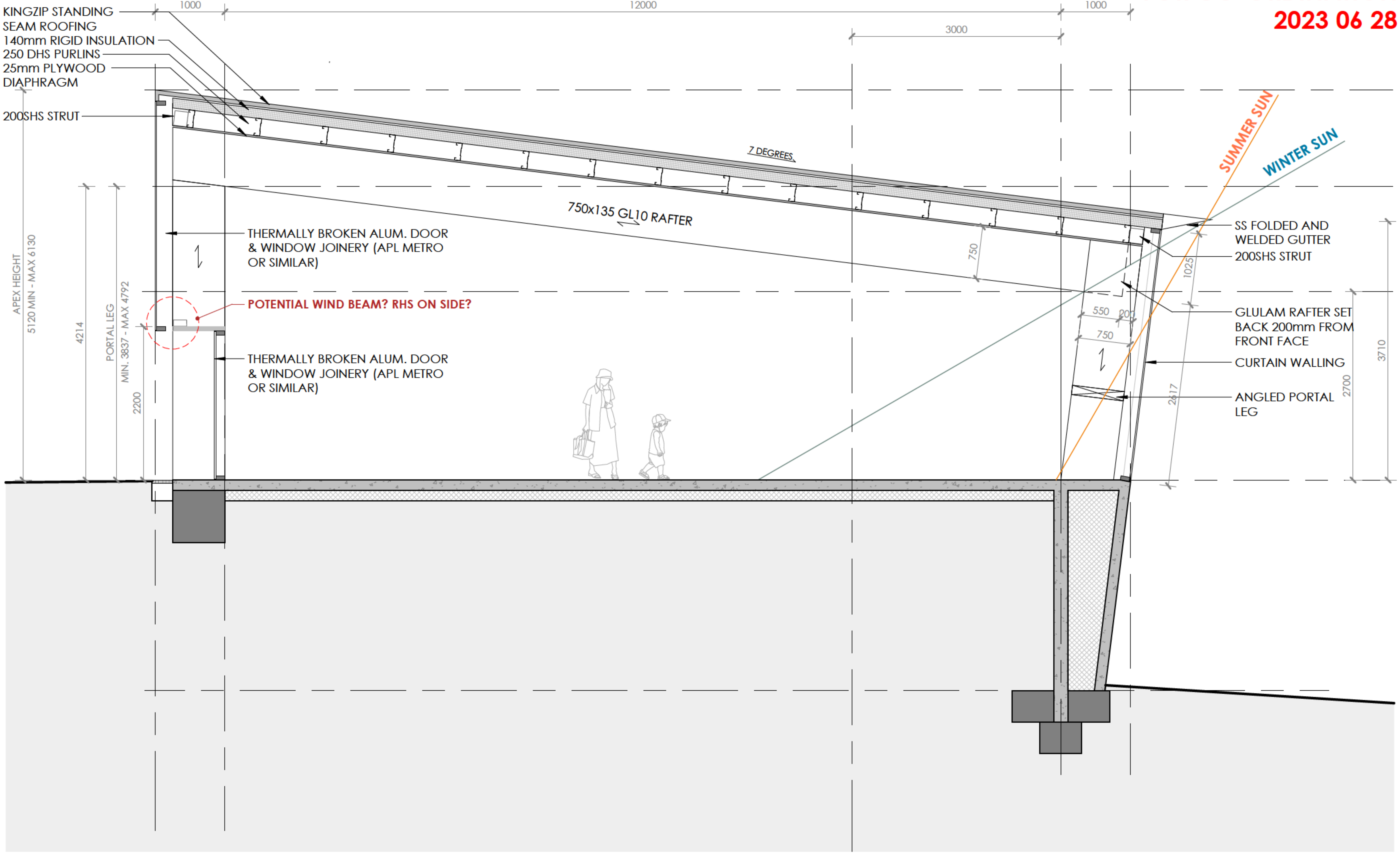
FOR CO-ORDINATION
2023 06 28



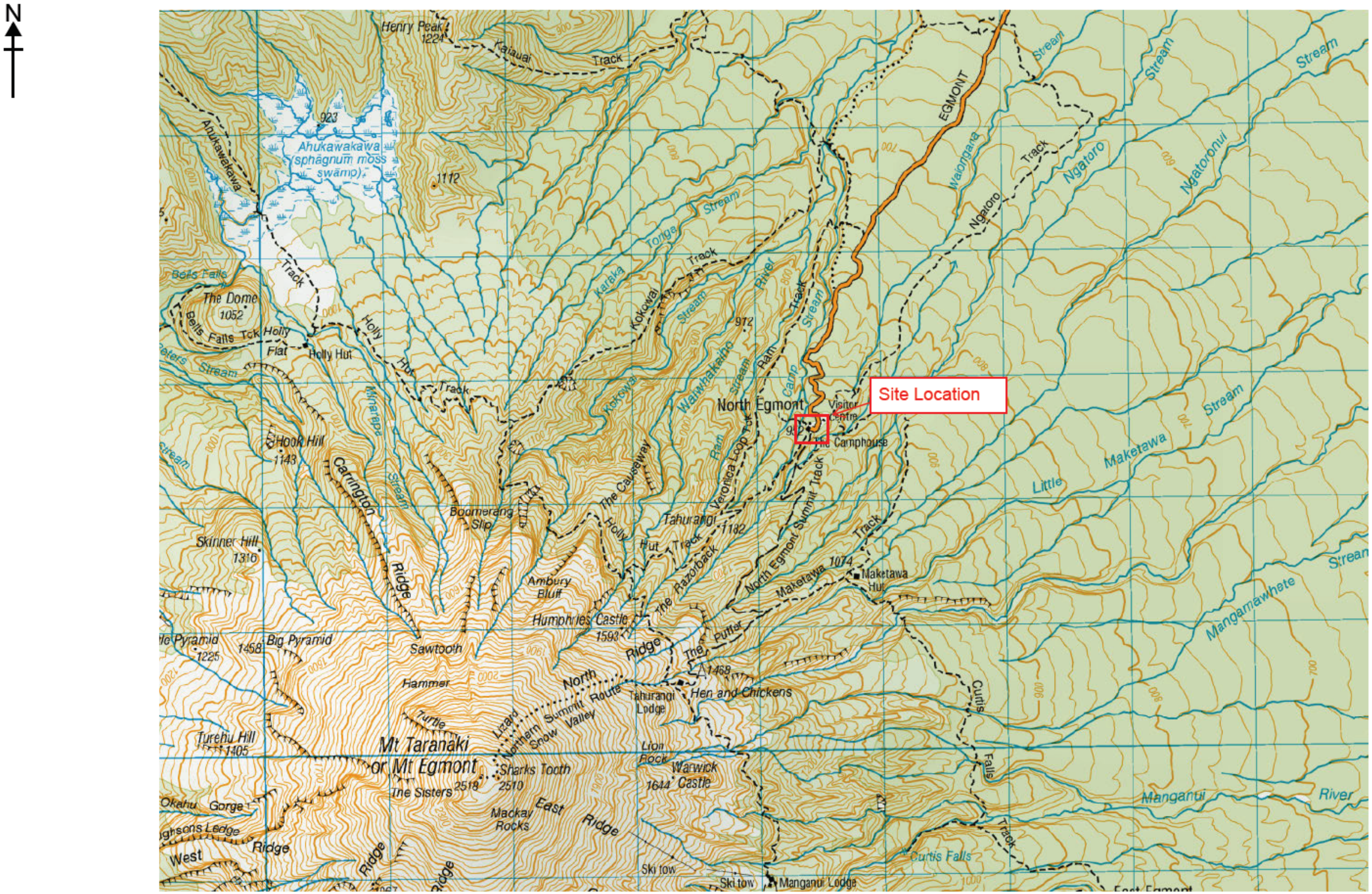
C9-12 STUDY - CROSS SECTION

1 : 50 @ A3

**FOR CO-ORDINATION
2023 06 28**

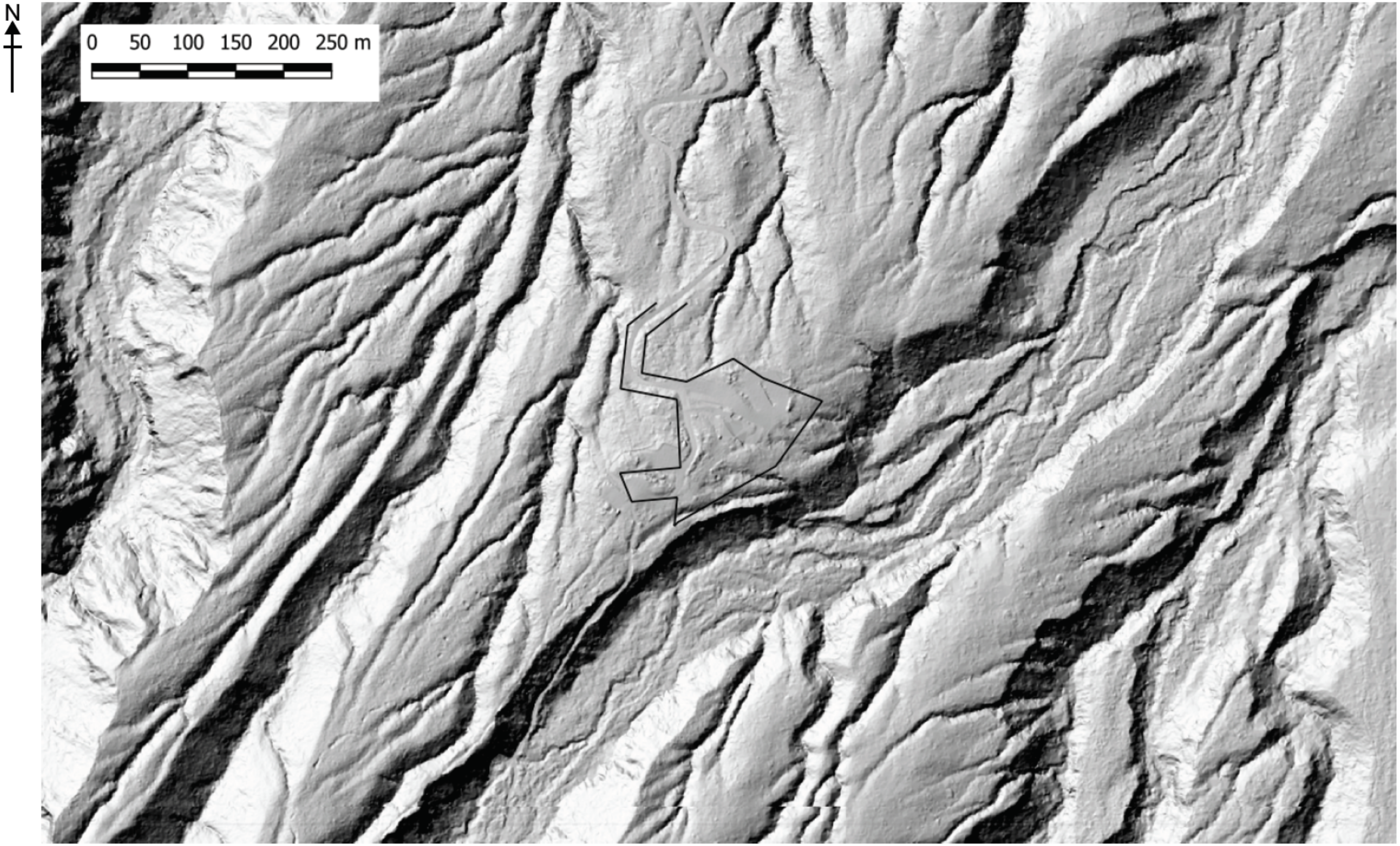


APPENDIX B - Site Details



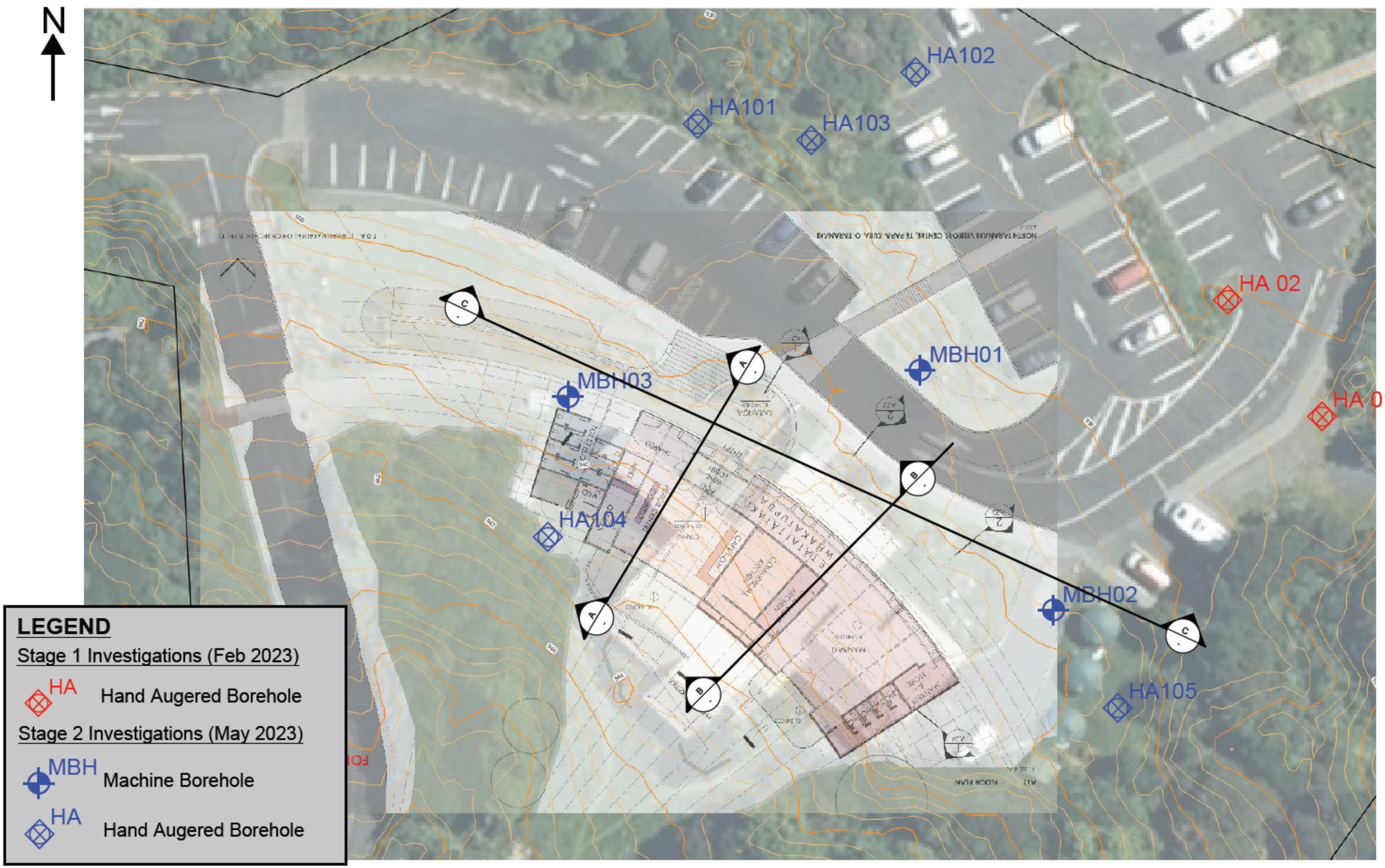
Site Location

Consultant	Client	Project Title	Sheet Title	Drawn: 7-03-2023	Scale:	At:
	TE KOTAHITANGA O TE ATIAWA	NORTH EGMONT VISITORS CENTRE REDEVELOPMENT	SITE OVERVIEW - TOPOGRAHIC MAP	Engineer: MSB		
				Job No: 23-0183	Sheet No: G-01	Revision: 01



Consultant	Client	Project Title	Sheet Title			
BCD GROUP	TE KOTAHITANGA O TE ATIAWA	NORTH EGMONT VISITORS CENTRE REDEVELOPMENT	SITE LIDAR PLAN	Drawn: 7-03-2023	Scale:	At:
				Engineer: MSB		
				Job No: 23-0183	Sheet No: G-02	Revision: 01

APPENDIX C - Investigation Data and Site Plan



LEGEND

Stage 1 Investigations (Feb 2023)

🟥 HA Hand Augered Borehole

Stage 2 Investigations (May 2023)

⊕ MBH Machine Borehole

🟦 HA Hand Augered Borehole

Consultant

BCD GROUP

Client

Te Kotahitanga o te Atiawa

Project Title

North Taranaki Visitors Center

Sheet Title

Site Investigation Plan

Drawn: 03/07/2023

Scale: NTS

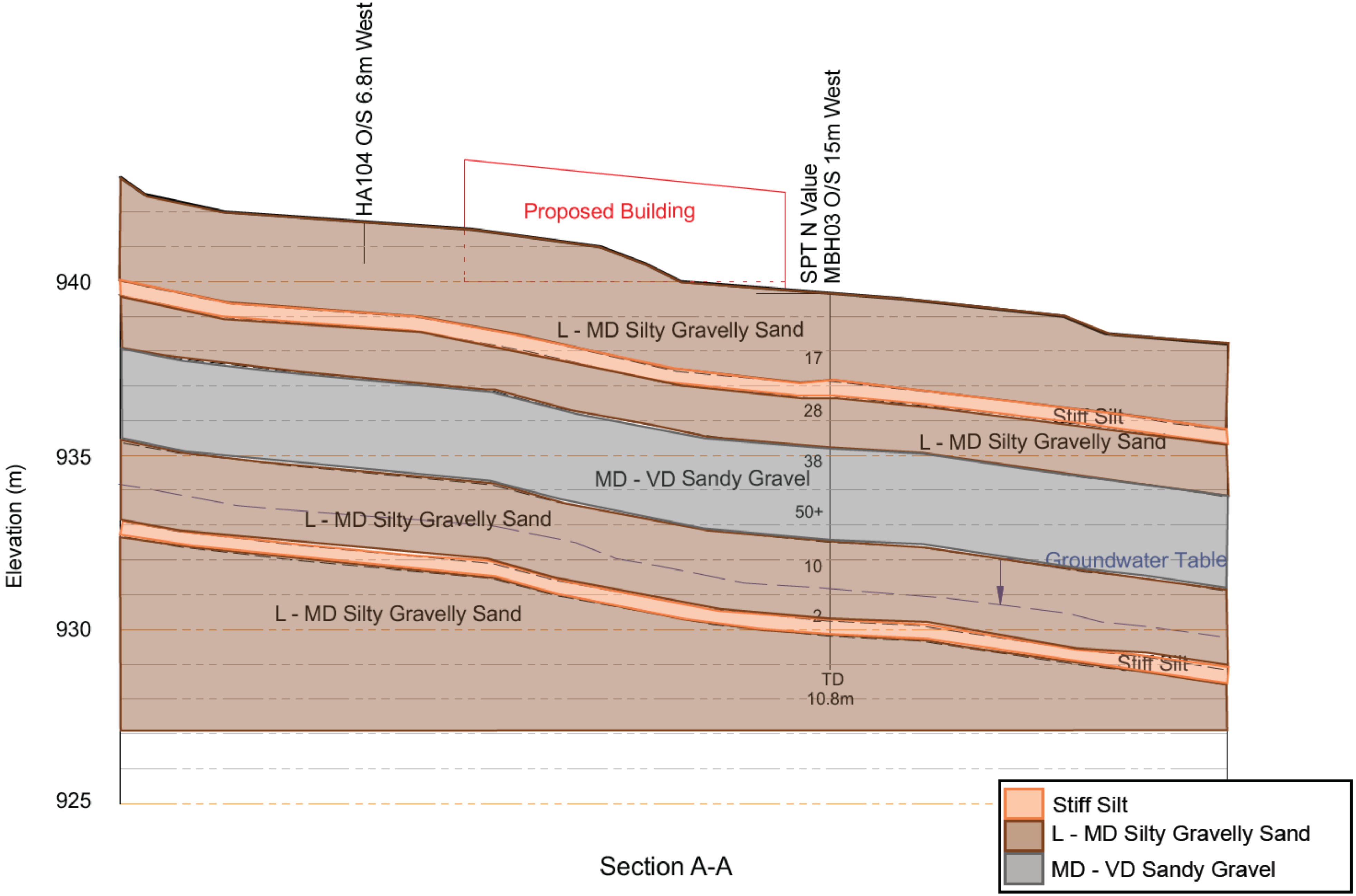
At: NA

Engineer: RV

Job No: 23-0183

Sheet No: G-01

Revision: 01



Section A-A

Levels are in terms of
TARANAKI DATUM 1970

NOTE: BOUNDARIES AND
DIMENSIONS AND AREAS
ARE APPROX MATE AND
SUBJECT TO ALTERATION
BY FINAL SURVEY

BCD GROUP

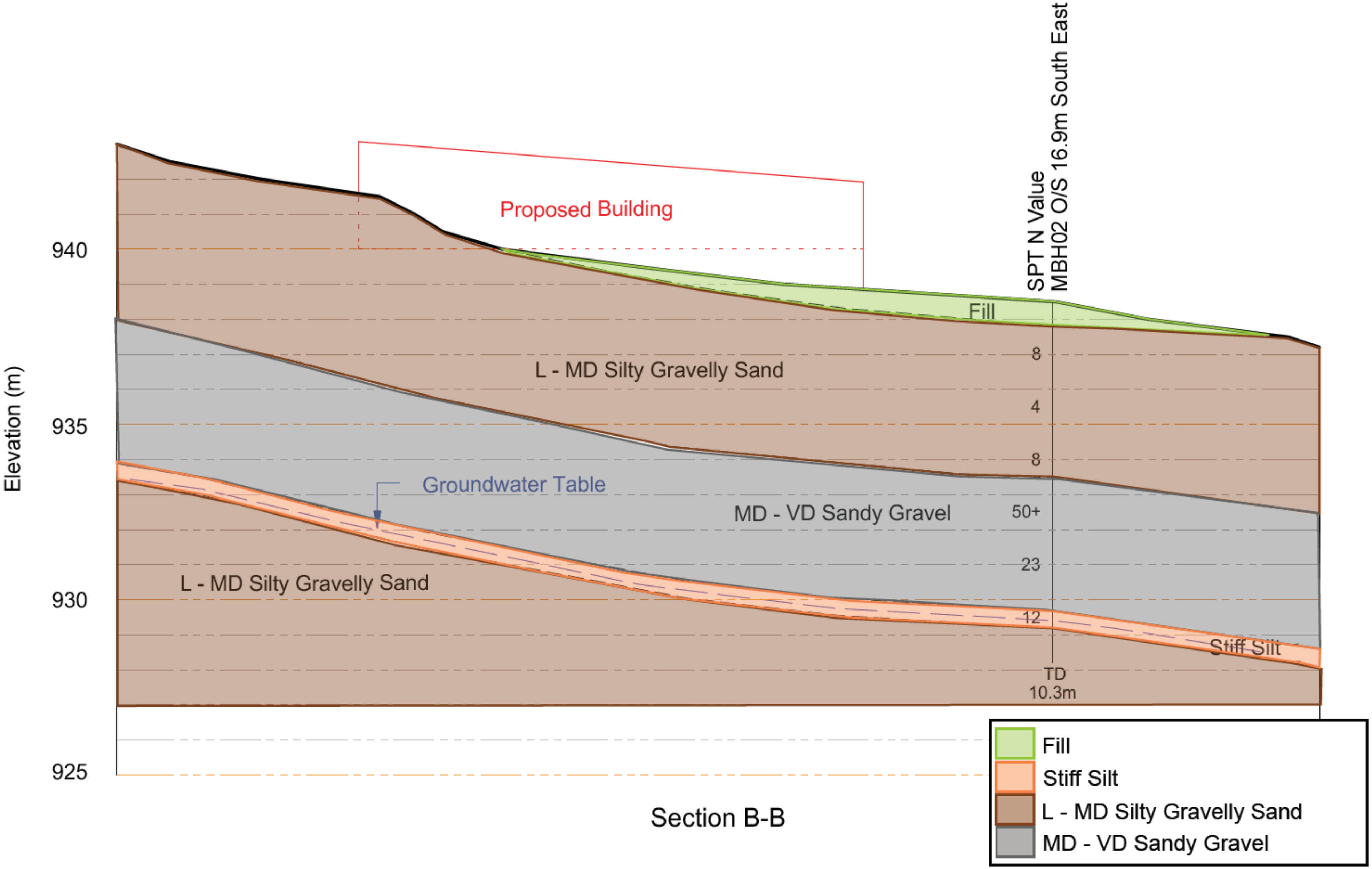
Sheet
GEOTECHNICAL CROSS SECTION

Project Title
NORTH TARANAKI VISITORS CENTRE
2879 EGMONT ROAD, EGMONT NATIONAL PARK

Rev	Date	by	Reason
1	26/06/2023	RV	For Building Consent

Drawn: 26/06/2023 Scale: 1:150 at A4
Engineer: RV Area:
Job No: 23-0183
Sheet No: SK001G
Revision 1

all dimensions to be verified on site before making any shop drawings
or commencing any work.



Section B-B

Levels are in terms of
TARANAKI DATUM 1970

NOTE: BOUNDARIES AND
DIMENSIONS AND AREAS
ARE APPROX MATE AND
SUBJECT TO ALTERATION
BY FINAL SURVEY

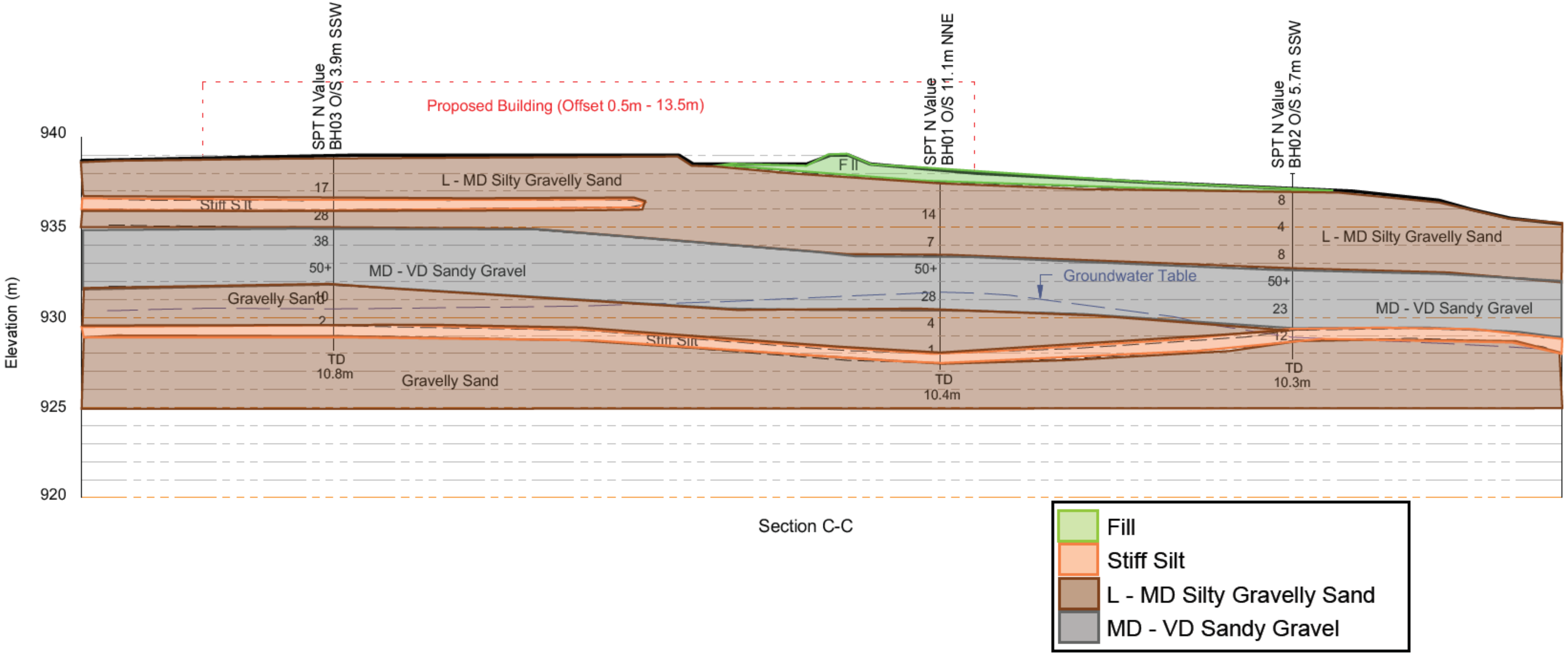
Sheet
GEOTECHNICAL CROSS SECTION

Project Title
NORTH TARANAKI VISITORS CENTRE
2879 EGMONT ROAD, EGMONT NATIONAL PARK

Rev	Date	by	Reason
1	26/06/2023	RV	For Building Consent

Drawn: 26/06/2023 Scale: 1:150 at A4
Engineer: RV Area:
Job No: 23-0183
Sheet No: SK002G
Revision 1

all dimensions to be verified on site before making any shop drawings
or commencing any work.



Section C-C

Levels are in terms of
TARANAKI DATUM 1970

NOTE: BOUNDARIES AND
DIMENSIONS AND AREAS
ARE APPROX MATE AND
SUBJECT TO ALTERATION
BY FINAL SURVEY

BCD GROUP

Sheet
GEOTECHNICAL CROSS SECTION

Project Title
NORTH TARANAKI VISITORS CENTRE
2879 EGMONT ROAD, EGMONT NATIONAL PARK

Rev	Date	by	Reason
1	26/06/2023	RV	For Building Consent

Drawn: 26/06/2023
Scale: 1:300 at A4
Engineer: RV
Area:

Job No:	Sheet No:	Revision
23-0183	SK003G	1

all dimensions to be verified on site before making any shop drawings
or commencing any work.

Soil Description

Log Identification: BH01 (Page 1 of 2)

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R L. (meters)	Geological Unit	Investigation Method	Rock strength	Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	Groundwater Level
	TOPSOIL; dark blackish brown.		FILL	HQ3	NA		67%								
	Gravelly **SAND** with trace silt; brown. Medium dense.Gravel, Fine to coarse subangular to subrounded. Sand, fine to coarse.														
	- At 0.4m becoming brownish grey.														
0.5		937.0													
	- 0.8m to 1.8m grades to Sandy **SILT**; grey. Sand, fine to coarse.														
1.0		936.5													
1.5		936.0					33%								
				SPT			55%	**14**	1	1	3	4	4	3	
2.0		935.5													
	- At 2.2m containing broken bricks.			HQ3			100%								
	Gravelly **SAND**; light yellowish brown. Loose, well graded. Gravel; Fine to coarse, subangular. Sand; fine to coarse.		MAERO DEBRIS FLOWS												
2.5		935.0					83%								
							0%								
3.0		934.5													
	- At 3.3m containing some silt, becoming brown, wet.			SPT			0%	**7**	1	2	2	3	1	1	
3.5		934.0													
				HQ3			175%								
	- At 3.8m containing trace andesitic cobbles.														
4.0		933.5													
							14%								
	- 4.2m to 4.8m core loss.														
4.5		933.0													
	Gravelly **SAND**; brownish grey. Very dense, well graded. Gravel; fine to coarse, subangular. Sand; fine to coarse.			SPT			55%	**50+**	4	8	8	15	4		
5.0		932.5													
				HQ3			25%								
5.5		932.0													

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270118° 174.096127°
Date of investigation: 10/05/2023

Job Number: 23-0183
Surface R.L. 937.5m
Logged By: RV
Checked By: MSB

Soil Description

Log Identification: BH01 **(Page 2 of 2)**

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R L. (meters)	Geological Unit	Investigation Method	Rock strength	Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	Groundwater Level
	(CONT) Gravelly SAND; brownish grey. Very dense, well graded. Gravel; fine to coarse, subangular. Sand; fine to coarse.		MAERO DEBRIS FLOWS	HQ3	NA		0%								
6.0	- At 6.1m groundwater table. Dipped approximately 15 min after drilling.	931.5					100%								▮
	- At 6.3m becoming medium dense.			SPT			30%	**28**	1	2	4	7	8	9	▮
6.5		931.0													▮
7.0		930.5		HQ3			50%								▮
7.5	Sandy **GRAVEL** with trace silt; yellowish brown. Gravel; fine to coarse, subangular. Sand; fine to coarse.	930.0													▮
				SPT			67%	**4**	1	1	1	0	2	1	▮
8.0	**SAND** with trace silt; dark grey. Poorly graded. Sand; medium to coarse.	929.5													▮
				HQ3			73%								▮
8.5		929.0													▮
	Silty **SAND** with some gravel; yellowish brown. Sand; fine to coarse. Gravel; fine to caorse, subangular.														▮
9.0		928.5													▮
	SILT with trace sand & gravel; brown. Stiff, wet, low plasticity. Sand; fine to coarse. Gravel; fine, subangular, pumiceous.			SPT			100%	**1**	1	0	0	0	0	1	▮
9.5		928.0													▮
				HQ3			100%								▮
10.0	Gravelly **SAND**; light yellowish brown. Loose. Sand; fine to coarse, pumiceous. Gravel; fine to coarse, subangular, pumiceous.	927.5													▮
	END OF BOREHOLE AT 10.4m - Target Depth														▮
10.5		927.0													▮
11.0		926.5													▮

Notes:

1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

BCD GROUP

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270118° 174.096127°
Date of investigation: 10/05/2023

Job Number: 23-0183
Surface R.L. 937.5m
Logged By: RV
Checked By: MSB

P \23-0183 North Taranaki Visitors Centre\060 BCD Geotechnical\062 Investigations\2 Logs\Machine Borehole\[23-05-12 BH01 23-0183.xlsx]BH01

Borehole ID	BH01	**Contractor**	Hardcore Drilling
Date Started	10/05/2023	**Drilling Method**	HQ Drilling
Date Finished	10/05/2023	**Logged By**	RV
Termination Depth	10.4m	**Checked By**	MSB
Ground Water Level	6.1m		

Box 1: 0.0m – 3.8m

Box 2: 3.8m – 7.8m

Box 3: 7.8m – 10.4m

Soil Description

Log Identification: BH02 **(Page 1 of 2)**

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R L. (meters)	Geological Unit	Investigation Method	Rock strength	Field Test Data			SPT results						Groundwater Level
						Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	
	Sandy **GRAVEL**; grey. Sand; fine to coarse. Gravel; fine to coarse, subrounded to subangular. Likely AP40 metal.		FILL	HQ3	NA		80%								
0.5		937.5													
	Silty gravelly **SAND**; brown. Loose, well graded. Gravel; fine to coarse, subangular to subrounded. Sand; fine to coarse.		MAERO DEBRIS FLOWS												
1.0		937.0					80%								
1.5		936.5		SPT			100%	**8**	1	2	3	0	1	4	
				HQ3			25%								
2.0	Sandy **GRAVEL**; dark brownish grey. Loose, well graded. Sand; fine to coarse. Gravel; fine to coarse, subangular.	936.0													
2.5		935.5													
							0%								
3.0		935.0		SPT			67%	**4**	1	1	1	1	2	0	
				HQ3			40%								
3.5	- At 3.5m becoming light brownish grey.	934.5													
							42%								
4.0		934.0													
	Silty **SAND** with some gravel; yellowish brown. Loose, well graded. Sand; fine to coarse. Gravel; fine to coarse, subangular.														
4.5		933.5		SPT			22%	**8**	3	4	1	1	3	3	
	- At 4.7m 100mm thick pumice lense.														
				HQ3			66%								
5.0	- At 5.0m becoming very dense.	933.0													
							0%								
5.5		932.5													

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270351° 174.096236°
Date of investigation: 08/05/2023

Job Number: 23-0183
Surface R.L. 938m
Logged By: RV
Checked By: MSB

P \23-0183 North Taranaki Visitors Centre\060 BCD Geotechnical\062 Investigations\2 Logs\Machine Borehole\[23-05-12 BH02 23-0183.xlsx]BH02

Soil Description

Log Identification: BH02 (Page 2 of 2)

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R L. (meters)	Geological Unit	Investigation Method	Rock strength	Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	Groundwater Level
	(CONT) Silty SAND with some gravel; yellowish brown. Loose, well graded. Sand; fine to coarse. Gravel; fine to coarse, subangular.		MAERO DEBRIS FLOWS	HQ3	NA		0%								
							0%								
6.0	SPT Refusal with 35mm to go.	932.0		SPT			44%	50+	14	16	14	16	14	6	
6.5		931.5		HQ3			29%								
	- At 6.7m 100mm thick brown silt lense. - At 6.8m becoming light greyish brown.														
7.0		931.0													
							50%								
7.5	- At 7.5m becoming medium dense	930.5		SPT			89%	23	2	6	7	7	6	3	
8.0		930.0		HQ3			41%								
	- At 8.2m sand becoming coarse, gravel becoming subangular, becoming brown.														
8.5		929.5													
	SILT with some sand; brown. Stiff, low plasticity. Sand; fine.														
9.0		929.0		SPT			100%	12	1	1	2	2	4	4	
	- At 9.1m groundwater table. Dipped approximately 25 min after drilling.														🟦
	Sandy **GRAVEL**; light greyish brown. Sand; fine to coarse. Gravel; fine to coarse, subangular, pumiceous.														🟦
9.5		928.5		HQ3			100%								🟦
10.0		928.0													🟦
	- At 10.1m containing Andesite cobbles and or boulders.														🟦
	END OF BOREHOLE AT 10.3m - Target Depth														🟦
10.5		927.5													🟦
11.0		927.0													🟦

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

BCD GROUP

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270351° 174.096236°
Date of investigation: 08/05/2023

Job Number: 23-0183
Surface R.L. 938m
Logged By: RV
Checked By: MSB

Project Name: North Taranaki Visitors Centre
Project Number: 23-0183

Borehole ID	BH02	**Contractor**	Hardcore Drilling
Date Started	08/05/2023	**Drilling Method**	HQ Drilling
Date Finished	08/05/2023	**Logged By**	RV
Termination Depth	10.3m	**Checked By**	MSB
Ground Water Level	9.1m		

Box 1: 0.0m – 3.1m

Box 2: 3.1m – 7.1m

Box 3: 7.1m – 10.3m

Soil Description

Log Identification: BH03 (Page 1 of 2)

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R L. (meters)	Geological Unit	Investigation Method	Rock strength	Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	Field Test Data: SPT results: N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	Groundwater Level
	TOPSOIL; dark blackish brown.		TS												
	Gravelly **SAND**; light brown. Medium dense. Gravel; Fine to coarse, subangular to subrounded. Sand; fine to coarse.														
0.5		938.5													
				HQ3			53%								
1.0		938.0													
1.5		937.5													
	- At 1.6m becoming grey.						56%								
								17	1	2	5	5	4	3	
2.0		937.0		SPT			44%								
							94%								
			MAERO DEBRIS FLOWS	HQ3	NA										
2.5		936.5													
	SILT with trace sand & gravel; brown. Stiff, low plasticity. Sand; fine to coarse, pumiceous. Gravel; fine to coarse, subangular to subrounded, pumiceous.														
3.0	Gravelly **SAND**; light brown. Medium dense. Gravel; Fine to coarse, subangular to subrounded. Sand; fine to coarse.	936.0													
							50%								
								28	3	4	6	7	8	7	
3.5		935.5		SPT			78%								
	- At 3.8m with some andesitic cobbles.						78%								
4.0		935.0													
				HQ3											
4.5		934.5					93%								
								38	6	9	9	11	10	8	
5.0		934.0		SPT			67%								
	- At 5.2m with red staining.														
	Sandy **GRAVEL**; brownish grey. Very dense, well graded. Gravel; fine to coarse, subangular. Sand; fine to coarse.			HQ3			50%								
5.5		933.5													

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270130° 174.095691°
Date of investigation: 11/05/2023

Job Number: 23-0183
Surface R.L. 939m
Logged By: RV
Checked By: MSB

Soil Description

Log Identification: BH03 (Page 2 of 2)

Drill Rig: Hard Core Drilling Wireline HQ

Depth (meters)	Field Description	Elevation R.L. (meters)	Geological Unit	Investigation Method	Rock strength	Torvane Strength (kPa)	Core Recovery (%) & RQD (if rock)	N Value	75 mm	75 mm	75 mm	75 mm	75 mm	75mm	Groundwater Level
6.0		933.0	MAERO DEBRIS FLOWS	HQ3	NA		40% 50%								
	SPT refusal due to bouncing. - At 6.4m becoming cobbly sandy **GRAVEL**; grey.			SPT			0%	50+	1	1					
6.5		932.5													
7.0		932.0		HQ3			90%								
7.5		931.5					0%								
	Gravelly **SAND**; light brown. Medium dense. Gravel; Fine to coarse, subangular to subrounded. Sand; fine to coarse, pumiceous. - At 8.0m sand no longer pumiceous. Becoming brown.			SPT			44%	10	2	1	2	1	3	4	
8.0		931.0													
8.5	- At 8.5m groundwater table. Dipped approximately 20 min after drilling. - 8.5m - 9.0m coreloss.	930.5		HQ3			17%								🟦
9.0		930.0													🟦
	SILT; dark brown. Stiff, moist, low plasticity.			SPT			44%	2	1	1	1	1	0	0	🟦
9.5		929.5													🟦
	Gravelly **SAND** with some silt; yellowish brown. Medium dense. Gravel; Fine to coarse, subangular to subrounded. Sand; fine to coarse.			HQ3			59%								🟦
10.0		929.0													🟦
10.5		928.5													🟦
	END OF BOREHOLE AT 10.8m - Target Depth														🟦
11.0		928.0													🟦

Field Test Data; SPT results

Notes:

1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
3. Undrained shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.

BCD GROUP

Project name: North Taranaki Visitors Centre
Site location: North Taranaki Visitors Centre
Coordinates: -39.270130° 174.095691°
Date of investigation: 11/05/2023

Job Number: 23-0183
Surface R.L. 939m
Logged By: RV
Checked By: MSB

Project Name: North Taranaki Visitors Centre
Project Number: 23-0183

Borehole ID	BH03	**Contractor**	Hardcore Drilling
Date Started	11/05/2023	**Drilling Method**	HQ Drilling
Date Finished	11/05/2023	**Logged By**	RV
Termination Depth	10.8m	**Checked By**	MSB
Ground Water Level	8.5m		

Box 1: 0.0m – 4.1m

Box 2: 4.1m – 7.0m

Box 3: 7.0m – 10.8m

Soil Description

Log Identification: HA01

R.L.
Taranaki: 935 m

Coordinates:
1694593 E, 5652670 N

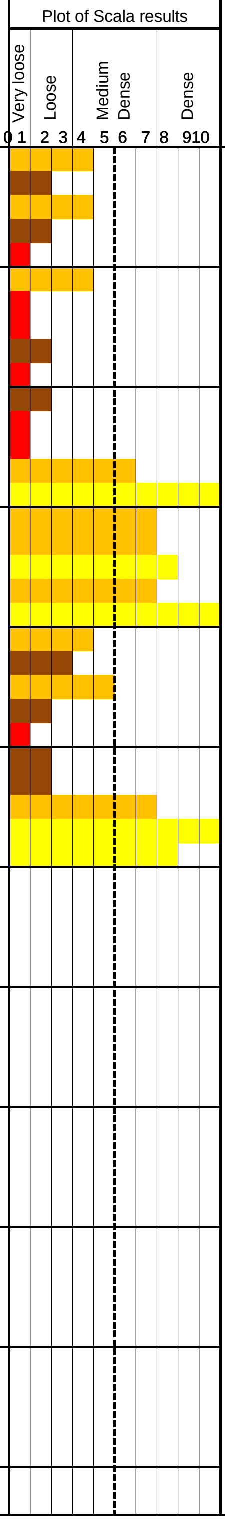
Field Description

Depth (meters)	Field Description	Geological Unit
0.0–1.2	**SILT** with some gravels and trace clay; brown. Very stiff to hard, moist, low plasticity; gravels, fine to medium, sub-angular.	FILL
1.2–1.6	**SILT** with trace sand; brown. Very stiff, moist, low plasticity; sand, fine to medium	TBA?
1.6–1.8	Silty **SAND**; brown. Medium dense, moist, fine to coarse grained.	
	End of hand auger at 1.7m - effective refusal *TBA = Taranaki Brown Ash	

Groundwater not encountered during testing

Field Test Data

Depth (meters)	Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity	Scala Penetrometer (blows per 100mm drop) Blow count
				4
				2
	190+			4
				2
				1
0.5	163	20	8.0	4
				1
	190+			1
				2
				1
1.0				2
				1
	109	39	2.8	1
				6
				10
1.5				7
				7
				8
				7
				11
2.0				4
				3
				5
				2
				1
2.5				2
				2
				7
				10
				8
3.0				
3.5				
4.0				
4.5				
5.0				
5.5				



Groundwater Level

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
3. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
4. Vane shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.
5. Scala Penetrometer testing (where reported) has been carried out in general accordance with NZS 4402 Test 6 5.2.
6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

BCD GROUP

Job Number: 23-0183
Client: Te Atiawa

Shear Vane ID:3236
Calibration Expiry Date: 7/06/2023
Shear Vane Factors: A: 1.359

Location: North Egmont Visitors Centre Carpark

Date of investigation: 21/02/2023

Logged By: MSB **Checked By: JA**

Soil Description

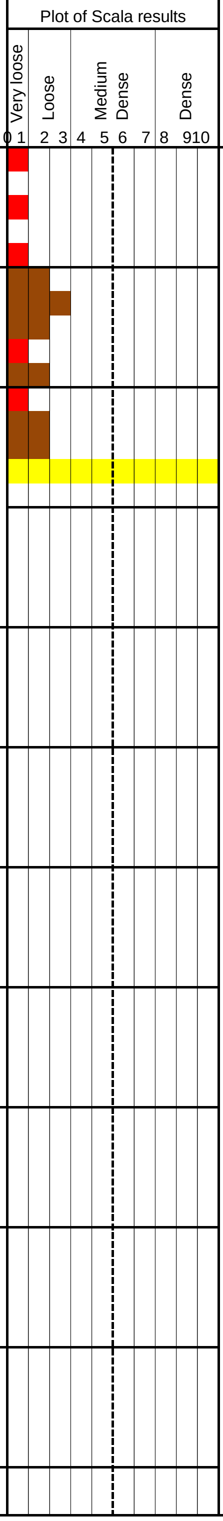
Log Identification: HA02

R.L.
Taranaki: 935 m

Coordinates:
1694584 E, 5652684 N

Field Test Data

Investigation method	Depth (meters)	Field Description	Geological Unit	Depth (meters)	Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity	Scala Penetrometer (blows per 100mm drop) Blow count	Groundwater Level
		TOPSOIL; SILT; dark brown. Moist, non plastic.	TOPSOIL					1	
								0	
								1	
					190+	0		0	
		SAND, grey. Loose, moist, fine to medium grained.	FILL?					1	
	0.5			0.5				2	
								3	
								2	
								1	
								2	
	1.0			1.0				1	
								2	
								2	
		At 1.3 m - grades to dense						20+	
		End of hand auger at 1.4 m - effective refusal							
	1.5			1.5					
		*TBA = Taranaki Brown Ash							
	2.0			2.0					
	2.5			2.5					
	3.0			3.0					
	3.5			3.5					
	4.0			4.0					
	4.5			4.5					
	5.0			5.0					
	5.5			5.5					
		Groundwater not encountered during testing							



Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
3. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
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Job Number: 23-0183
Client: Te Atiawa

Shear Vane ID:3236
Calibration Expiry Date: 7/06/2023
Shear Vane Factors: A: 1.359

Location: North Egmont Visitors Centre Carpark

Date of investigation: 21/02/2023

Logged By: MSB **Checked By: JA**

Soil Description

Log Identification: HA101

R.L.
Taranaki: 937m

Coordinates:
-39.269907° 174.095877°

Investigation method: Hand Auger (50mm Diameter)

Depth (meters)	Field Description	Geological Unit	Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity
0.0 – 0.1	**TOPSOIL**; dark blackish brown. Wet.	TS			
0.1 – 0.6	Sandy gravelly **SILT**; dark brown. Stiff, wet, no plasticity, moderately sensitive. Sand; fine to coarse. Gravel; fine, subangular.	MDF*	93	28	3.3
0.6 – 1.1	Gravelly **SAND** with trace silt; greyish brown. Loose to dense, wet. Gravel; fine to medium, subangular. Sand; fine to coarse. - At 0.8m becoming grey. - At 1.0m becoming very dense.	MDF*			
1.1	**END OF BOREHOLE AT 1.1m - Refusal**				

MDF* = MAERO DEBRIS FLOWS

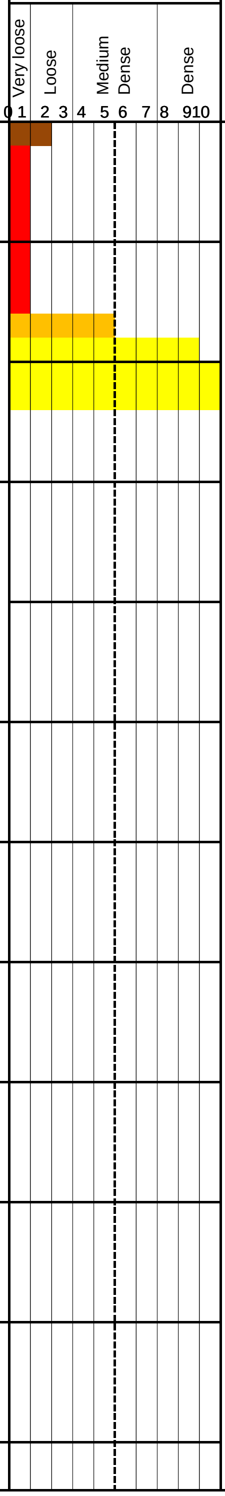
Groundwater not encountered during testing

Field Test Data

Scala Penetrometer (blows per 100mm drop)

Depth (meters)	Blow count
0.0 – 0.1	2
0.1 – 0.2	1
0.2 – 0.3	0.5
0.3 – 0.4	0.5
0.4 – 0.5	1
0.5 – 0.6	1
0.6 – 0.7	1
0.7 – 0.8	1
0.8 – 0.9	5
0.9 – 1.0	9
1.0 – 1.1	19
1.1 – 1.2	20+

Plot of Scala results



Groundwater Level

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
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6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

BCD GROUP

Job Number: 23-0183
Client: Te Kotahitanga o Te Atiawa

Shear Vane ID:3663
Calibration Expiry Date: 23/11/2023
Shear Vane Factors: A: 1.558

Location: North Taranaki Visitors Centre

Date of investigation: 18/05/2023

Logged By: RV **Checked By: MSB**

Soil Description

Log Identification: HA102

R.L.
Taranaki: 935m

Coordinates:
-39.269830° 174.096102°

Investigation method: Hand Auger (50mm Diameter)

Depth (meters)	Field Description	Geological Unit
0.0	**TOPSOIL**; dark blackish brown. Moist.	UF
	Gravelly **SILT** with some sand; light brown. Moist, low plasticity.	UF
	SAND some gravel; brownish grey. Moist. Sand; fine to coarse. Gravel; fine, subrounded.	UF
0.5	**SILT** with some sand & gravel; dark brown. Stiff to very s iff, moist, low plasticity, insensitive to moderately sensitive. Sand; fine to coarse. Gravel; fine, subrounded. - At 0.8m with brown mottling. - At 0.9m becoming wet. - At 1.1m becoming brown. - At 2.0m containing some amorphous organics. Black streaks.	MAERO DEBRIS FLOWS
	Gravelly **SAND**; light brown. Medium dense, saturated. Sand; fine to coarse. Gravel; fine to medium, subrounded, pumiceous. **END OF BOREHOLE AT 2.4m - Refusal**	MAERO DEBRIS FLOWS

UF = UNCONTROLLED FILL

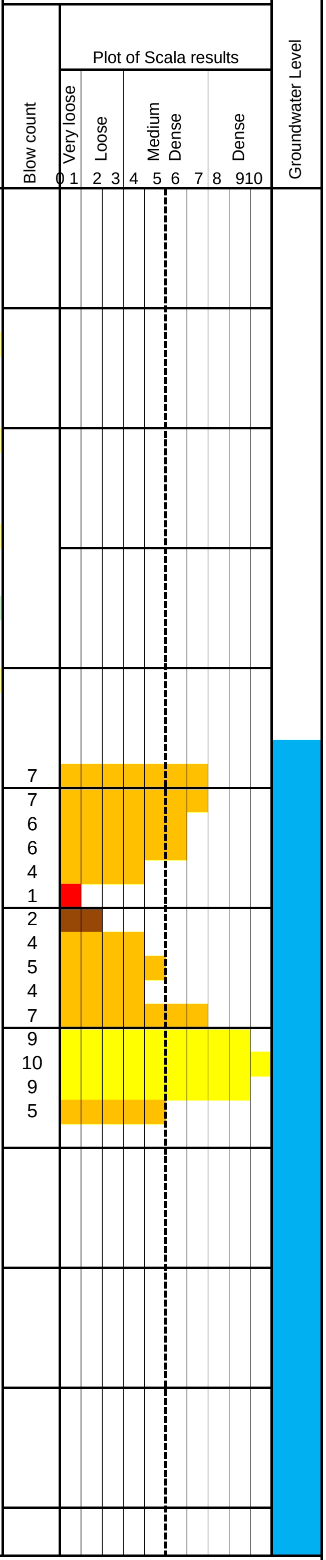
Groundwater encountered at 2.3m during testing.

Field Test Data

Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity
62	16	4.0
97	28	3.4
125	31	4.0
65	37	1.8
62	31	2.0

Scala Penetrometer (blows per 100mm drop)

Depth (meters)	Blow count
2.4–2.5	7
2.5–2.6	7
2.6–2.7	6
2.7–2.8	6
2.8–2.9	4
2.9–3.0	1
3.0–3.1	2
3.1–3.2	4
3.2–3.3	5
3.3–3.4	4
3.4–3.5	7
3.5–3.6	9
3.6–3.7	10
3.7–3.8	9
3.8–3.9	5



Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
3. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
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5. Scala Penetrometer testing (where reported) has been carried out in general accordance with NZS 4402 Test 6 5.2.
6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

Job Number: 23-0183
Client: Te Kotahitanga o Te Atiawa

Shear Vane ID:3663
Calibration Expiry Date: 23/11/2023
Shear Vane Factors: A: 1.558

Location: North Taranaki Visitors Centre

Date of investigation: 18/05/2023

Logged By: RV **Checked By: MSB**

Soil Description

Log Identification: HA103

R.L.
Taranaki: 935.5m

Coordinates:
-39.269919° 174.096010°

Investigation method: Hand Auger (50mm Diameter)

Depth (meters)	Field Description	Geological Unit
0.0	Organic **SILT**; dark blackish brown. Moist. Rootlets. - At 0.3m rootlets absent. - At 0.7m with light brown streaks. - At 0.8m becoming wet.	TOPSOIL FILL?
1.7	Sandy **GRAVEL**; yellowish brown. Medium dense, wet. Sand; fine to coarse. Gravel; fine to medium, angular to subangular. - At 2.0m becoming grey.	MDF*
2.1	**END OF BOREHOLE AT 2.1m - Refusal** MDF* = MAERO DEBRIS FLOWS	

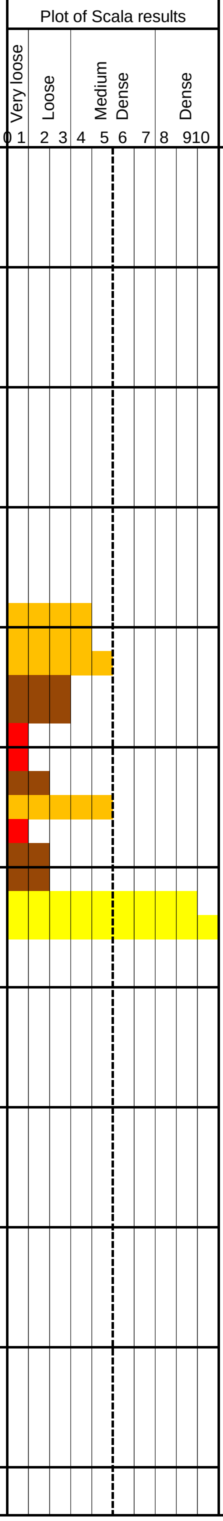
Groundwater not encountered during testing

Depth (meters): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5

Field Test Data: Peak Vane Shear Strength (kPa) | Residual Vane Shear Strength (kPa) | Sensitivity | Groundwater Level

Scala Penetrometer (blows per 100mm drop)

Blow count:
4
4
5
3
3
1
1
2
5
1
2
2
9
20+



Notes:
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6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

Job Number: 23-0183
Client: Te Kotahitanga o Te Atiawa

Shear Vane ID:3663
Calibration Expiry Date: 23/11/2023
Shear Vane Factors: A: 1.558

Location: North Taranaki Visitors Centre

Date of investigation: 18/05/2023

Logged By: RV **Checked By: MSB**

Soil Description

Log Identification: HA104

R.L.
Taranaki: 941.5m

Coordinates:
-39.270257° 174.095675°

Investigation method: Hand Auger (50mm Diameter)

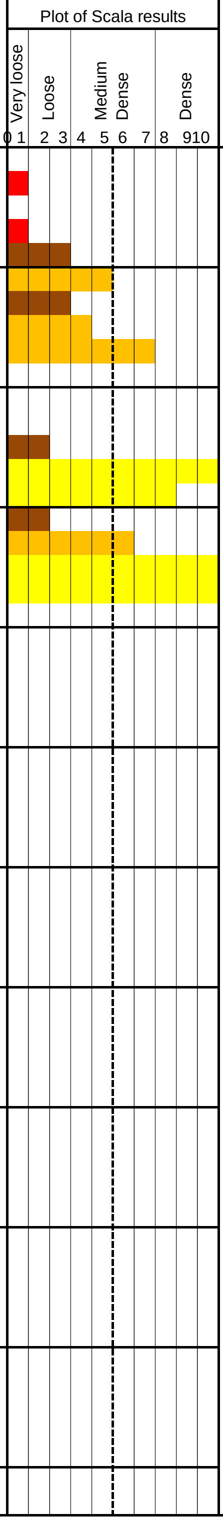
Depth (meters)	Field Description	Geological Unit
0.0	**TOPSOIL**; dark blackish brown. Moist. Rootlets.	TS
	SILT with trace sand; grey. Very stiff, moist, low plasticity, extra sensitive. Sand; fine to coarse. At 0.3m with red staining.	MDF*
0.5	**SAND**; dark grey. Medium dense, wet, well graded. Sand; fine to coarse. - At 0.5m containing some gravel & silt, brownish grey. Gravel; fine to medium subangular to subrounded. - At 0.7m silt content decreasing to trace. - At 0.9m silt absent. - At 1.0m becoming grey.	MDF*
1.2	**END OF BOREHOLE AT 1.2m - Refusal**	

MDF* = MAERO DEBRIS FLOWS

Groundwater not encountered during testing

Field Test Data

Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity	Scala Penetrometer (blows per 100mm drop) Blow count	Groundwater Level
			0	
			1	
125	16	8.0	0	
			1	
			3	
			5	
			3	
			4	
			7	
			OB	
			OB	
			OB	
			2	
			10	
			8	
			2	
			6	
			10	
			20+	



Notes:
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7. Shear vane results are multiplied by factor A and plus factor B where applicable

Job Number: 23-0183
Client: Te Kotahitanga o Te Atiawa

Shear Vane ID:3663
Calibration Expiry Date: 23/11/2023
Shear Vane Factors: A: 1.558

Location: North Taranaki Visitors Centre

Date of investigation: 18/05/2023

Logged By: RV **Checked By: MSB**

Soil Description

Log Identification: HA105

R.L.
Taranaki: 937m

Coordinates:
-39.270410° 174.096323°

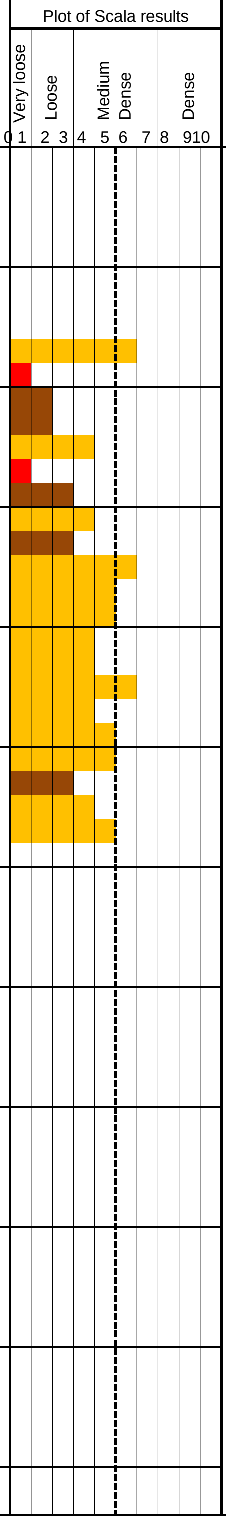
Investigation method	Depth (meters)	Field Description	Geological Unit
Hand Auger		**TOPSOIL**; dark blackish brown. Moist. Roo lets.	TS
		Silty **SAND** with some gravel; dark grey. Moist. Sand; fine to coarse. Gravel; fine, subangular to subrounded.	MDF*
	0.5	Sandy **GRAVEL**; light brown. Moist. Sand; fine to coarse. Gravel; fine, subangular to subrounded. At 0.4m containing trace silt, becoming dark brown.	
		END OF BOREHOLE AT 0.8m - Refusal	
	1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5		
		Groundwater not encountered during testing	

Field Test Data

Peak Vane Shear Strength (kPa) | Residual Vane Shear Strength (kPa) | Sensitivity | Groundwater Level

Scala Penetrometer (blows per 100mm drop)

Blow count
6
1
2
2
4
1
3
4
3
6
5
5
4
4
6
4
5
5
3
4
5



Notes:
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Job Number: 23-0183
Client: Te Kotahitanga o Te Atiawa

Shear Vane ID:3663
Calibration Expiry Date: 23/11/2023
Shear Vane Factors: A: 1.558

Location: North Taranaki Visitors Centre

Date of investigation: 18/05/2023

Logged By: RV **Checked By: MSB**

APPENDIX D - Preliminary Geotechnical Letter

16/03/2023

Te Kotahitanga o Te Atiawa
c/- RCP Limited

Attention: [redacted]
Via Email:

RE: 23-0183 - North Taranaki Visitors Centre Redevelopment – Preliminary Geotechnical Letter

1 INTRODUCTION

BCD Group have been engaged by RCP Ltd on behalf of Te Kotahitanga o Te Atiawa Trust (Te Atiawa) to provide a preliminary desktop geotechnical review of the above reference property. Redevelopment of the Visitors Centre is currently proposed on the site however the client would like to ascertain any geotechnical limitations to developing on this site. This assessment focuses on the large-scale geotechnical hazard that could potentially impact the site.

This letter should be utilised for preliminary guidance only, a detailed site-specific geotechnical investigation and assessment will be required at part of the future development design stage.

2 SITE OVERVIEW

2.1 Site Description

The site is located at the road end of Egmont Road on the north-eastern side of Mt Taranaki (refer to Figure A-01 in Appendix A). The existing north Egmont visitors centre is at approximately 940 m RL with the lower carparking areas being at slightly lower elevations. The ground surface is generally gently sloping towards the northeast.

2.2 Site History

RCP Ltd has provided BCD with several snips from Mt Taranaki historical books and some relevant information. The key findings are described below:

- The North Egmont Old House was first constructed in 1892 at the lower tier.
- The North Egmont Chalet was constructed at the approximate location of the current visitor's centre in 1912. The Chalet was demolished in 1977.
- The Aerial photo from 1976 shows that multiple buildings have been developed with roading and carparking areas which are similar to current day.
- The current visitors centre was opened in 1980 with a redevelopment completed 2010. The Beca plans provided to BCD indicate the current carparking areas where the future building may be located are formed in cut and fill earthworks.

3 DESKTOP REVIEW

BCD have reviewed the following information as part of the desktop study portion of this assessment:

- LINZ Lidar data
- Published Geologic Maps
- Known Geological Hazards

The Local Authority GIS and New Zealand Geotechnical Database were also reviewed, however no relevant information was obtained.

3.1 Site Topography

The Taranaki LIDAR data for the site area has been downloaded and reviewed to determine the location and extent of slopes in the wider area surrounding the site. The crest of the larger gulley slopes are evident in the Lidar data.

The LIDAR data has been converted into site contours at major (2 m) and minor (0.5 m) intervals as shown on the site investigation plan in Appendix A. The Lidar visualisation of the wider area is also presented in Appendix A. This visual clearly shows the site being located on a ridgeline with larger gulley features to the south-east and north-west.

3.2 Published Geological Maps

The 1:250K Geological map (Townsend D, et al, 2008) indicates that the location of the visitors centre is on a ridgeline underlain by Holocene Lahar flow deposits of the Kahui formation. This formation is described as Multiple beds of andesitic conglomerate and sand, some with broken tree trunks and branches, and pyroclastic flow deposits. These deposits are estimated at between 7,000 and 12,000 years of age.

The geological map indicates that the large gulley to the southeast of the site is underlain by more recent Holocene igneous rock deposits of the Maero Formation. These are described as multiple beds of unconsolidated andesitic conglomerate and sand, with minor pyroclastic flow deposits. These deposits are estimated at less than 1,000 years old.

It is well understood that various lahar and avalanche deposits, lava flows, pyroclastic flows form from the geological landscape up Mt Taranaki.

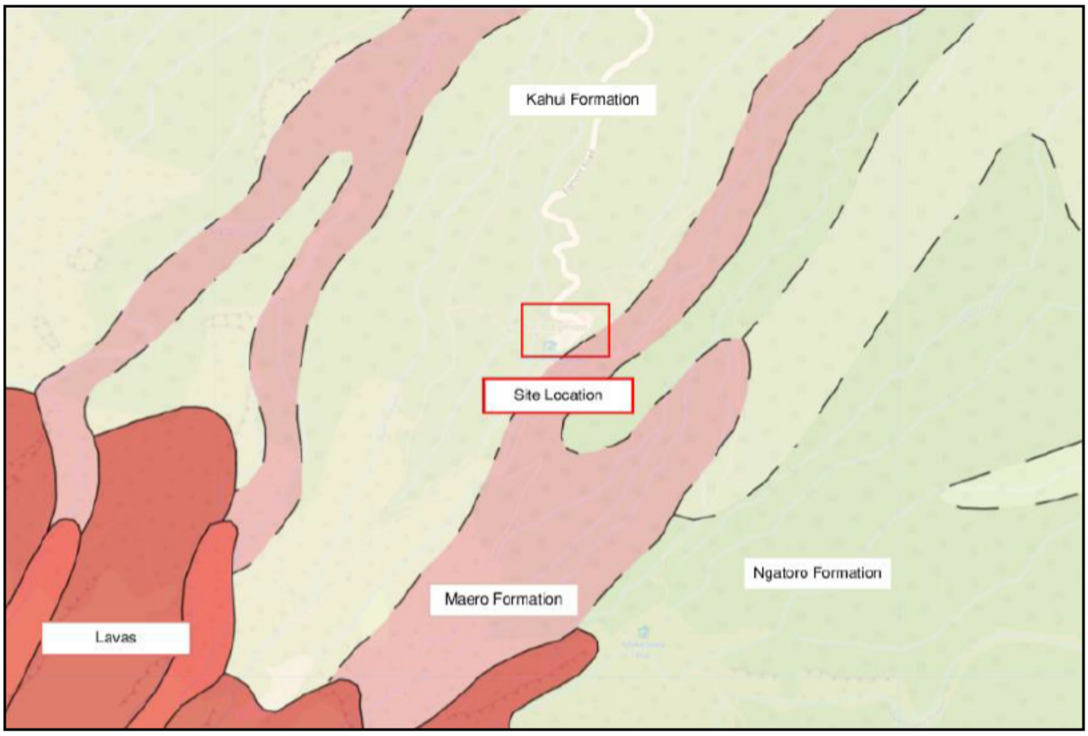

Figure 1: Published Geological Map – Zoomed in on Site location (Source GNS)

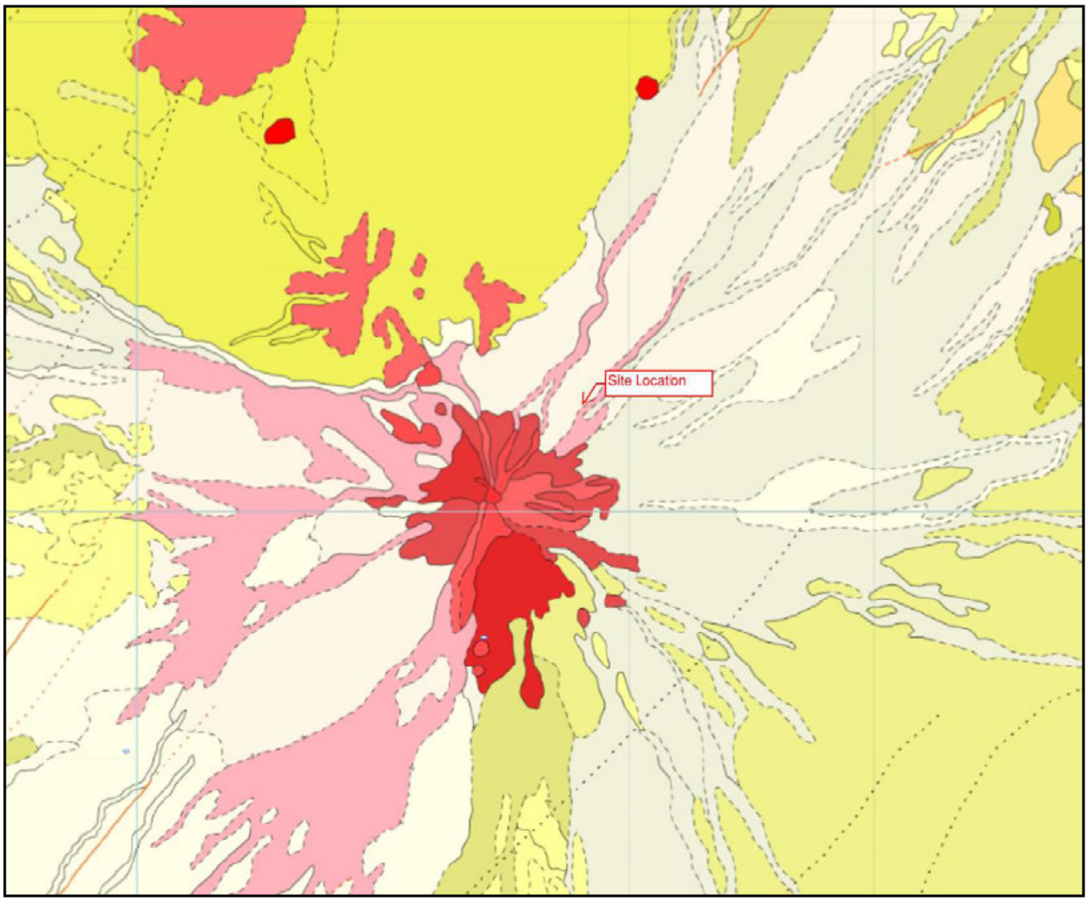

Figure 2: Published Geological Map Showing Taranaki Volcanic Cone (Source GNS)

3.3 Volcanic Hazards

Due to the site location in proximity to Mt Taranaki, there are additional volcanic Hazards for the site which are noted and described below. Though historically considered dormant, recent studies indicate that Mt Taranaki displays more frequent record of activity. Major eruptions are estimated to have occurred every 500 years, with minor eruptions occurring every 90 years on average. The last major eruption occurred around 1655. It is estimated that there is between 30% and 40% probability of an eruption in the next 50 years.

Whilst it may not be possible to eliminate the volcanic hazards through design, the risk and consequence of these hazards can be assessed.

There are various sources of information available about the volcanic hazards on Taranaki. The Civil Defence Emergency Management (CDEM) Taranaki website provides up to date infographics about the key volcanic hazards. The website also provides a detail of the volcanic activity alert levels and the evacuation zones. Figure 3 has been taken from the CDEM website and depicts the various zones set out by CDEM. The Visitors Centre is located within the Red volcanic evacuation zone. This zone comprises the area most at risk from life threatening hazards. It is stated that people who remain in this zone during a significant eruption are unlikely to survive.

Figure 3: Taranaki Volcanic Evacuation Zones (source CDEM Taranaki

An earlier paper written for the Ministry of Civil Defense (Neall V.E. and Alloway, B.V. 1991) titled "Volcanic Hazards at Egmont Volcano" provides a more in-depth discussion of the key volcanic hazards. The volcanic hazards for the site are summarised based on this paper and the relevant hazard maps presented below.

3.3.1 Pyroclastic Flow and Lateral Blast Hazard Zone

Pyroclastic flows are fast moving clouds of rocks, steam and hot ash. These flows are deadly and highly destructive causing fatal burns and injuries. The principal danger to life and property from pyroclastic flows is likely to be residents and buildings within 15km of the top of the Mt Taranaki.

The outer limits of the hazard zones presented below were constructed based on the distribution of pyroclastic flows in the last 15,000 years. The site is located within the Pyroclastic Flow and Lateral Blast Hazard Zone A as shown on Figure 4. Zone A has been mapped as an area that is likely to be affected most severely and most frequently.

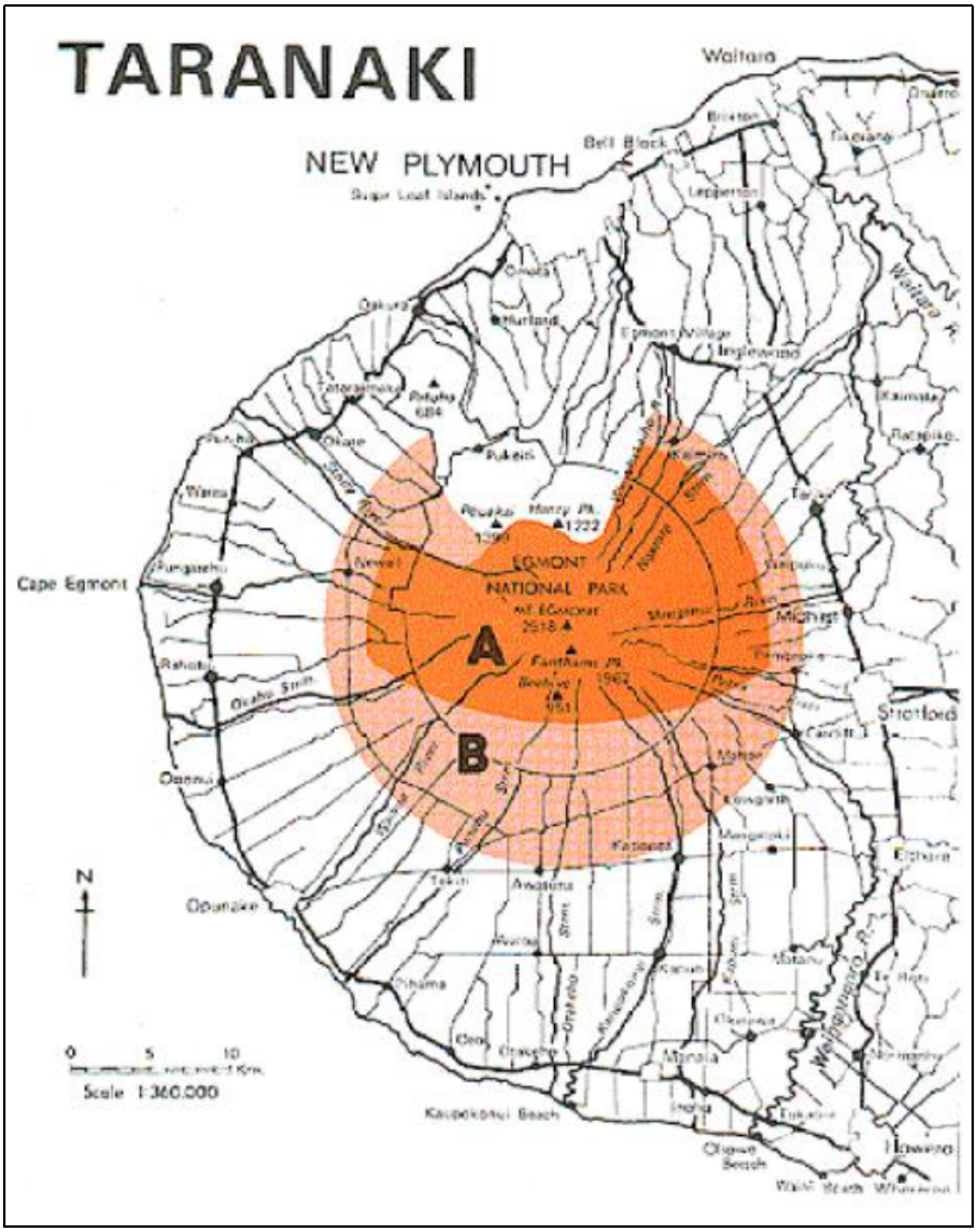

Figure 4: Pyroclastic Flow and Lateral Blast Hazard Zones

3.3.2 Landslide, Lahar, and Associated Flood Hazard Zone

It is known that landslide and lahar deposits form the topography of the Mt Taranaki area. The chances of any given area being affected by a lahar decrease with increasing distance from the volcano and to lesser extent, with increasing distance from the main drainage channels. Historically, the gorges and river channels within the confines of the national park are where many lahars have travelled and where the greater risk would lie.

The site is within the Landside and Lahar Hazard Zone A as shown on Figure 5. It has been mapped as an area that is likely to be affected most severely and most frequently. The average incidence interval of landslides and lahars within Zone A ranges from 1 Per 500 years to 1 Per 3000 years.

Though stratigraphic records show reoccurring periods of frequent large lahar events, Mt Taranaki and the surrounding ring plain have not been inundated by lahars in historically recorded times. Whilst the site is positioned in an area of high risk based on proximity to the likely lahar and landslide paths, the risk of the site being affected is reduced due to the localised topography.

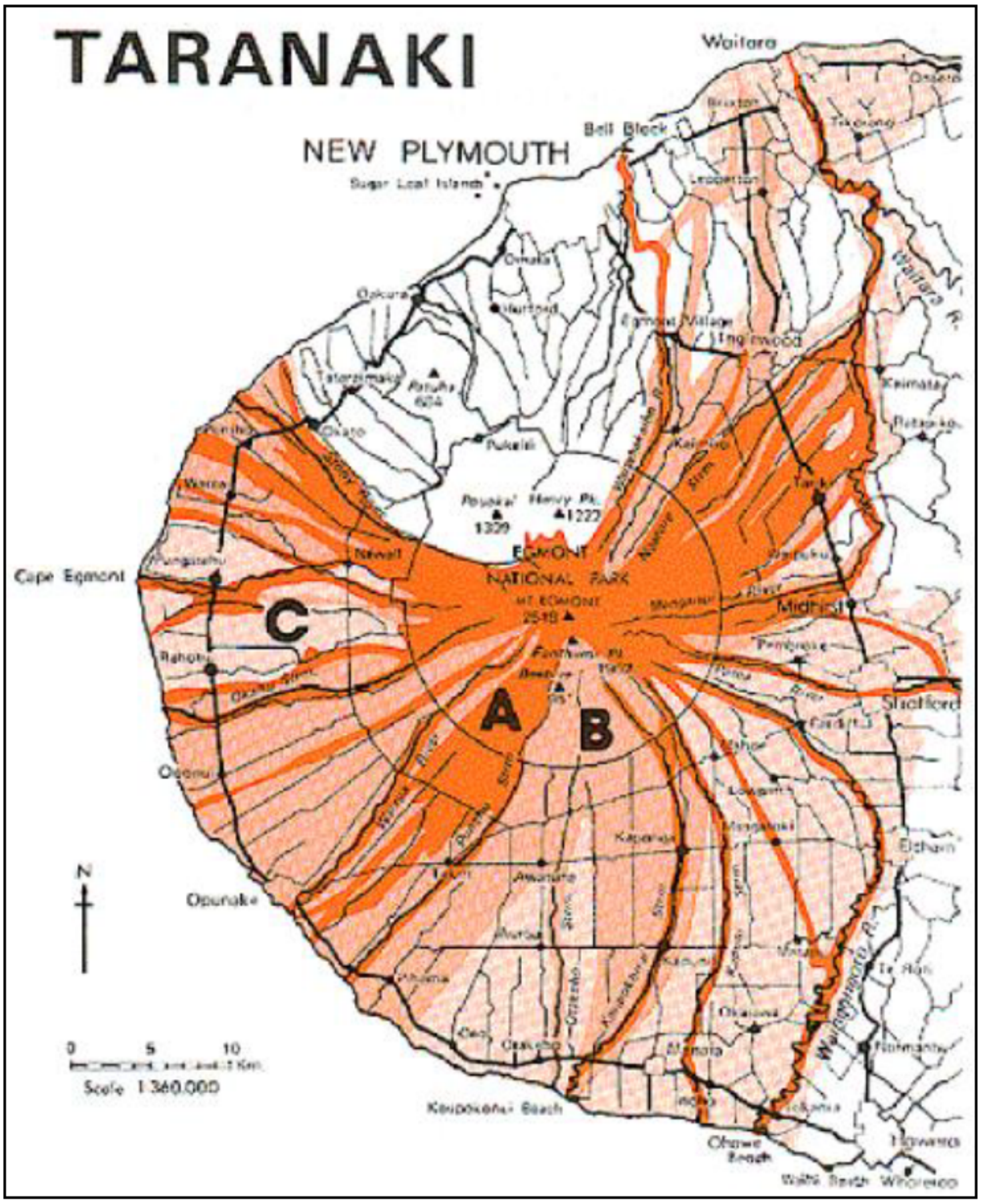

Figure 5: Landslide, Lahar, and associated flood Hazard Zones

3.3.3 Tephra Hazard Zone

Volcanic Tephra is made of rock fragments and particles ejected by volcanic eruption. Volcanic Tephra near to the volcanic cone is often comprised more coarse grained materials. Volcanic ash is a fine grained volcanic tephra which is known to be present across the wider Taranaki area.

The site is within Zone A on the Tephra hazard map shown in Figure 6. This is described where the thickness of Tephra could be in excess of 0.25 m during an eruption and extends about 21 km to the NNE and SSE from the summit. The primary hazard from Tephra is due to accumulation of the deposit which leads to weight/additional loads on buildings, although there is a risk to health were 'dust/associated gases' are breathed in.

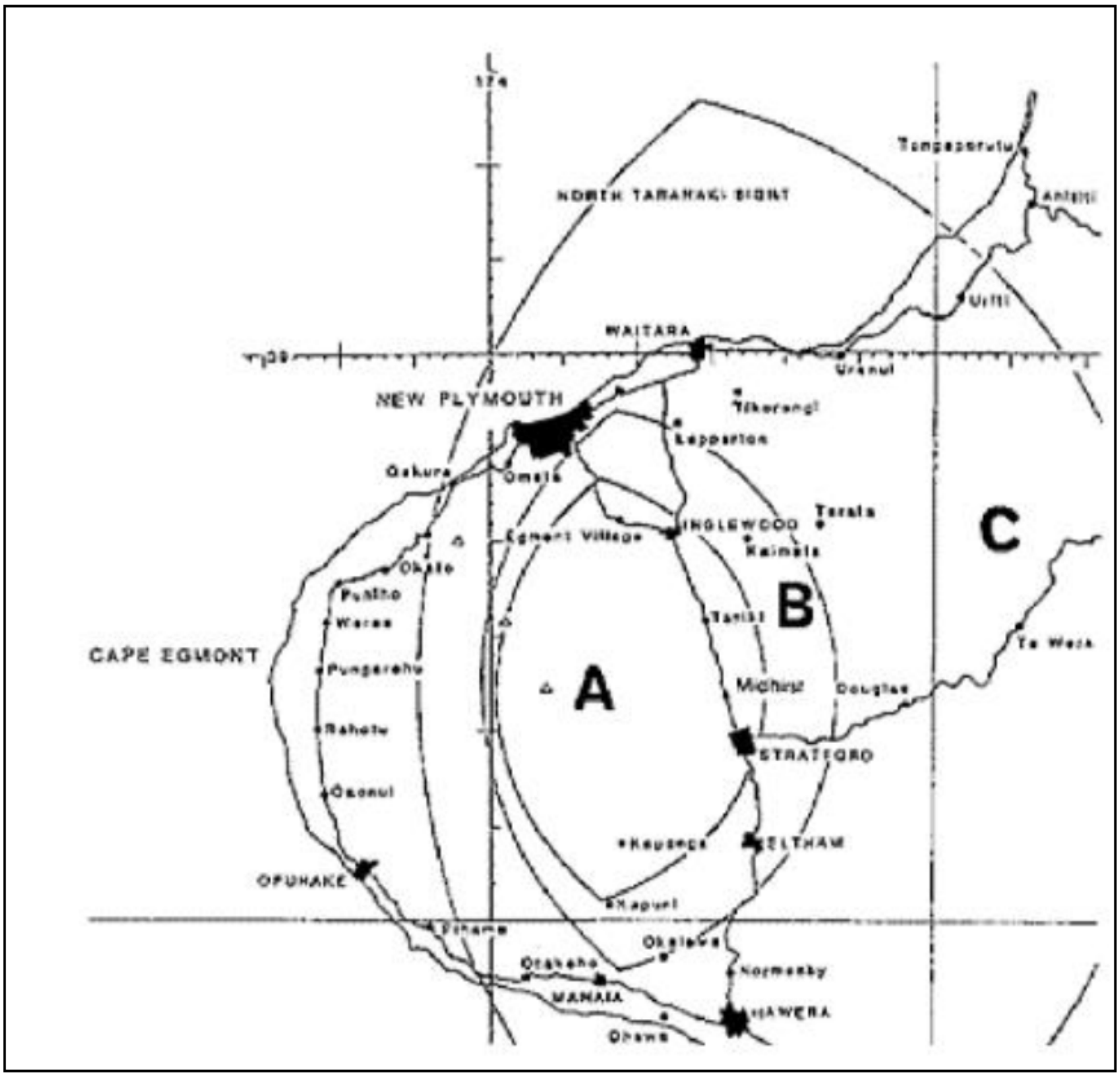

Figure 6: Tephra Deposit Hazard Maps

3.3.4 Lava flows and Lava Dome hazard Zone

The hazard map was not available from the sourced paper. It is noted that the lava flows historically have been erupted from the central vents of the Egmont crater and Fanthams Peak as indicated on the published geological map in Figure 2 and are indicates to have erupted about 14,000 years ago. These are regarded as the likely source areas for future lava flows. Lava flows are expected to travel slowly but move over large distances.

It is anticipated that the site is likely within the mapped lava flow hazard zone. However, the localised topography of the site being on the ridgeline would significantly reduce the likelihood of the site being affected by lava flows.

4 GEOTECHNICAL INVESTIGATIONS

4.1 Site Walkover

A site walkover was completed by a BCD geotechnical engineer on 23 February 2023. The purpose of the walkover was to observe the site profile and to check for any soil or rock exposures which might assist with high level geotechnical assessment and constraints.

The site is generally positioned on top of a prominent ridgeline and is gently sloping to the northeast. To the southwest, and some distance away, is a large steepened slope that runs sub-parallel with the main ridge line. The site is generally grassed or vegetated beyond the building footprint and there were no noteworthy geomorphic features observed.

Some small soil exposures were observed beneath the viewing platform and along the nature walk track, which indicate that the natural surface soils comprise tephra ash fall deposits comprising silts with variable sands and gravels.

4.2 Subsoil Profile

BCD completed preliminary investigations comprising 2 No. Hand Augured boreholes with Scala penetrometer testing located approximately in the potential location for the future visitors centre. The boreholes were extended to effective refusal with Scala testing to 3 m depth or prior refusal. The soils were logged in accordance with NZGS guidelines. The investigation locations are included on the site plan in Appendix A. The logs are included in Appendix B.

The following subsurface conditions are inferred based on the preliminary site investigations:

Existing Fill

The surface materials within HA01 are inferred as existing fill comprising silt with some gravels up to 1.2 m depth. Shear vane readings within the silts indicate the fill is generally very stiff with undrained shear strengths greater than 160 kPa.

In HA02 location, A poorly graded fine to medium grained sand was encountered below the topsoil and extending up to 1.5 m below ground surface. This material is considered likely to be existing fill which may have been imported as part of the current works. An average Scala penetrometer reading of 2 blows per 100 mm was recorded within this sand layer indicating it is relatively loose.

Tephra deposits

The underlying soil within HA01 appeared to be Tephra deposits comprising very stiff silts and medium dense to dense sands. The tephra seemingly becomes more sandy and gravelly with depth. The Scala penetrometer results indicate that there may be interbedded layers of more silty and sandy deposits.

Deep Soils / Rock

The nature of the deeper soils or rock was not able to be confirmed by hand auger investigations. Scala refusal was encountered at 1.5 m depth in HA02; however Scala refusal was not encountered within 3 m depth in HA01 location. The depth to a non-compressible soil/rock layer is likely variable across the site.

5 HIGH LEVEL GEOTECHNICAL ASSESSMENT

An assessment of the key geotechnical constraints is completed based on the desk study assessment, investigation data and BCDs knowledge of the site conditions. The following are noted:

- The subject site is within the higher risk zones for volcanic natural hazards. Some of the hazard risks are reduced due to the site location and localised topography, being up on the ridgeline surrounded by larger gulley features on either side. However, due to the historic frequency of activity from the volcano there is a low likelihood of rockfall, avalanche and lahar flows affecting the site.

- The potential building site around the lower carpark is likely underlain by existing fill which may vary in nature and strength. Some of the fills encountered display high strength indicating they may have been well compacted as part of the historical site works. However, the sandy fill encountered has lower Scala readings indicating that it may not have been placed to an engineered standard. The full extent and depth of fill is not able to be determined with limited investigations, however existing fill is up to 1.5 m thick in places.

- Loose, potentially disturbed soils were encountered down to about 1.5 m in depth. The proposed building may require either piled foundations extending into the competent natural ground, or ground remediation to form engineered fill platform suitable to support a shallow foundation system.

- If ground remediation is adopted as the preferred solution, the existing fill materials are likely suitable for reuse on site as engineered fill subject to preparation of an earthwork's specification.

- There is not expected to be any highly compressible soils within the natural soil profile. The risk of static settlement affecting the building is considered very low to negligible provided the building is founded in natural ground or well compacted engineered fill.

- The risk of liquefaction and lateral spreading effects is considered low due to the nature of deposition, the strength of the underlying soils, the site topography and absence of a groundwater water table.

- The risk of global slope stability issues are low due to the sites generally gentle gradient and the distance from the steepened slopes.

Further detailed geotechnical assessment will be required for the building once the location is confirmed and as part of the consenting process. Further hand auger testing may prove futile given the soil conditions. It is recommended that machine boreholes and test pit excavations are utilised for the site-specific investigations to more accurately assess the soil conditions.

6 LIMITATIONS

This report has been prepared for our client for preliminary informational purposes only. It is based on our understanding of the proposed development which is in the very early stages of design. A detailed geotechnical assessment will be required as part of consenting for the development.

The recommendations and opinions made in this report are based upon data from observations made on-site, conducted hand augers, and in-situ soil strength testing at discrete locations. Inferences about the nature and continuity of subsoils away from the exploration holes are made but cannot be guaranteed. Actual conditions onsite may vary more gradually or abruptly than that inferred from the investigations.

The reliance by other parties on the information or opinions contained in this report shall, without prior review and agreement in writing, be at such parties' sole risk. To avoid misinterpreting this report, we recommend that the assistance of geotechnical professionals familiar with the project and scope of this report is maintained.

Regards,

[redacted]

Geotechnical Engineer
BE(Hons), MEngNZ
BCD Group Ltd

Reviewed and approved for release by:

[redacted]

Chartered Geotechnical Engineer
BSc(Hon) Geology, MEngSc, CMEngNZ, CPeng
BCD Group Ltd

Appendices:

- Appendix A – Site Plans
- Appendix B – Geotechnical logs

APPENDIX A - Site Plans

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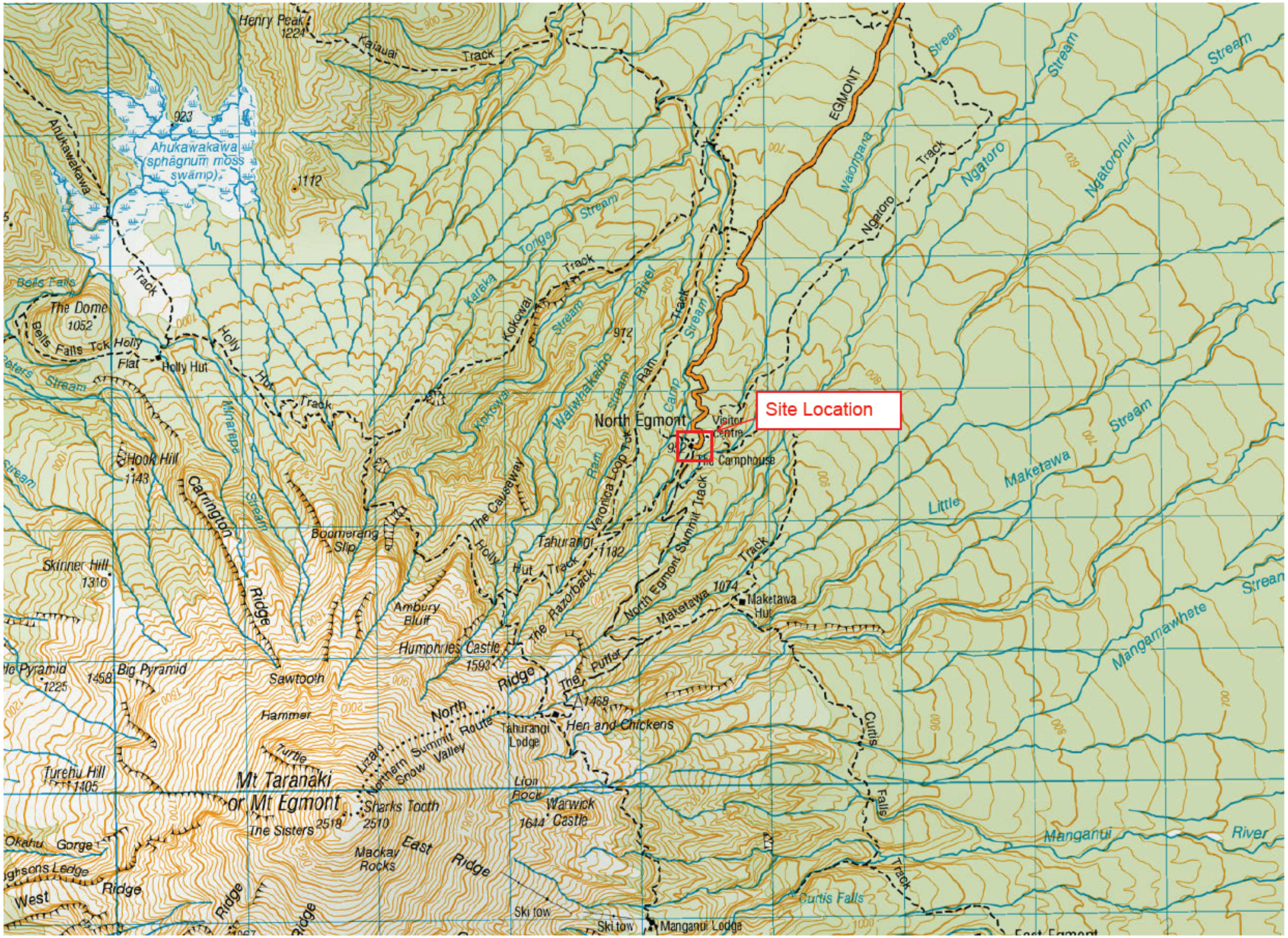


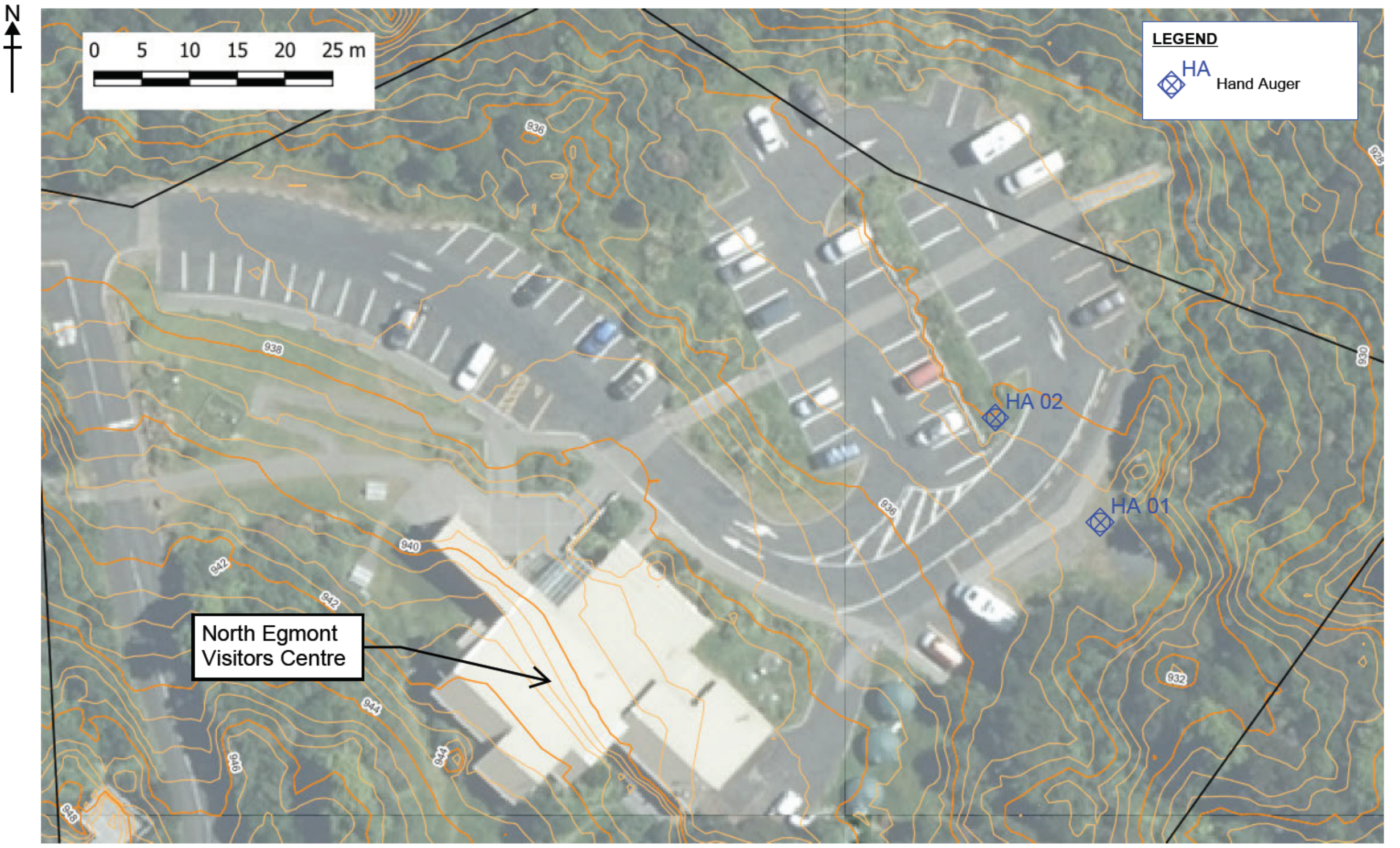
Consultant

Client
TE KOTAHITANGA O TE ATIAWA

Project Title
NORTH EGMONT VISITORS CENTRE REDEVELOPMENT

Sheet Title
SITE OVERVIEW - TOPOGRAHIC MAP

Drawn: 7-03-2023
Scale:
At:
Engineer: MSB
Job No: 23-0183
Sheet No: A-01
Revision: 01



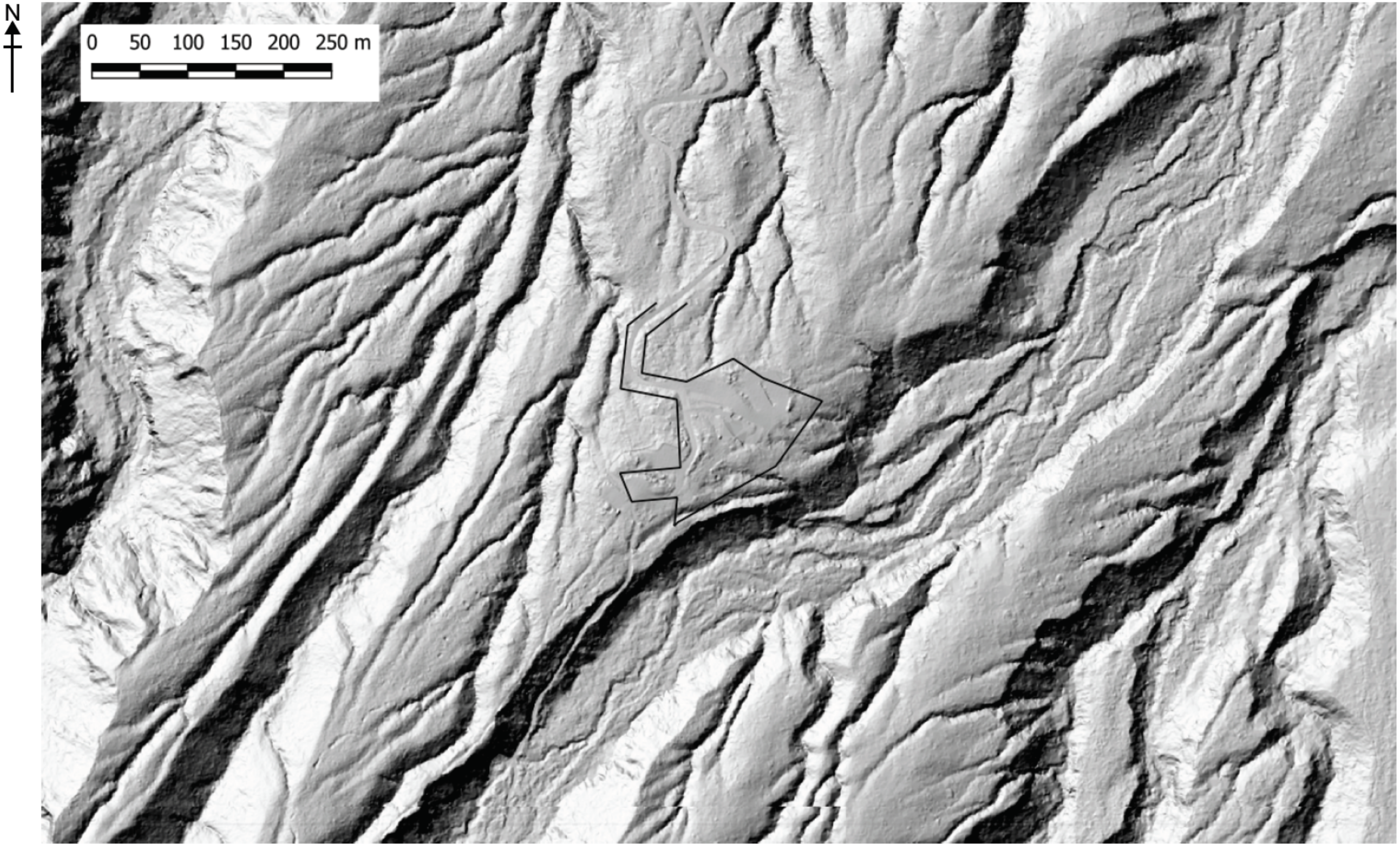
Consultant
BCD GROUP

Client
TE KOTAHITANGA O TE ATIAWA

Project Title
NORTH EGMONT VISITORS CENTRE REDEVELOPMENT

Sheet Title
SITE INVESTIGATION PLAN

Drawn: 7-03-2023
Scale:
At:
Engineer: MSB
Job No: 23-0183
Sheet No: A-02
Revision: 01



Consultant

Client
TE KOTAHITANGA O TE ATIAWA

Project Title
NORTH EGMONT VISITORS CENTRE REDEVELOPMENT

Sheet Title
SITE LIDAR PLAN

Drawn: 7-03-2023
Scale:
At:
Engineer: MSB
Job No: 23-0183
Sheet No: A-03
Revision: 01

APPENDIX B - Investigation Logs

Soil Description

Log Identification: HA01

R.L.
Taranaki: 935 m

Coordinates:
1694593 E, 5652670 N

Investigation method	Depth (meters)	Field Description	Geological Unit	Depth (meters)
	0.0 – 1.2	**SILT** with some gravels and trace clay; brown. Very stiff to hard, moist, low plasticity; gravels, fine to medium, sub-angular.	FILL	0.5, 1.0
	1.2 – 1.6	**SILT** with trace sand; brown. Very stiff, moist, low plasticity; sand, fine to medium	TBA?	1.5
	1.6 – 1.7	Silty **SAND**; brown. Medium dense, moist, fine to coarse grained.		
	1.7	**End of hand auger at 1.7m - effective refusal** *TBA = Taranaki Brown Ash		2.0

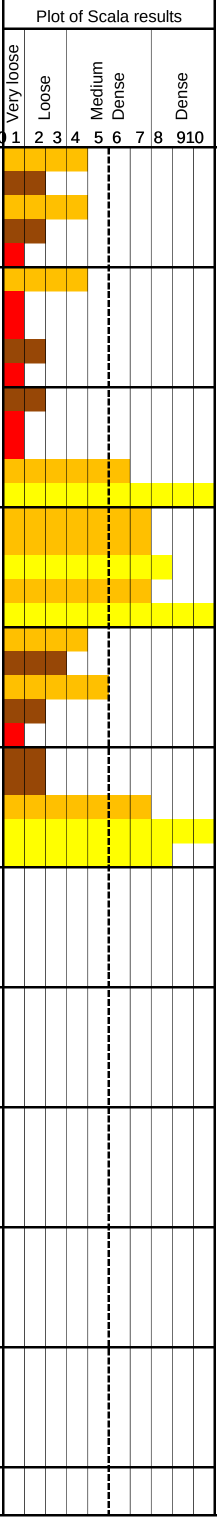
Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5

Groundwater not encountered during testing

Field Test Data

Scala Penetrometer (blows per 100mm drop)

Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity	Blow count
			4
			2
190+			4
			2
			1
163	20	8.0	4
			1
190+			1
			2
			1
			2
			1
109	39	2.8	1
			6
			10
			7
			7
			8
			7
			11
			4
			3
			5
			2
			1
			2
			2
			7
			10
			8



Groundwater Level

Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
3. Soils have been described in general accordance with NZ Geomechanics Society "Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes", December 2005
4. Vane shear strengths (where reported) have been corrected in general accordance with NZ Geotech Society Inc. "Guideline for Hand Held Shear Vane Test", August 2001.
5. Scala Penetrometer testing (where reported) has been carried out in general accordance with NZS 4402 Test 6 5.2.
6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

Job Number: 23-0183
Client: Te Atiawa

Shear Vane ID:3236
Calibration Expiry Date: 7/06/2023
Shear Vane Factors: A: 1.359

Location: North Egmont Visitors Centre Carpark

Date of investigation: 21/02/2023

Logged By: MSB **Checked By: JA**

Soil Description

Log Identification: HA02

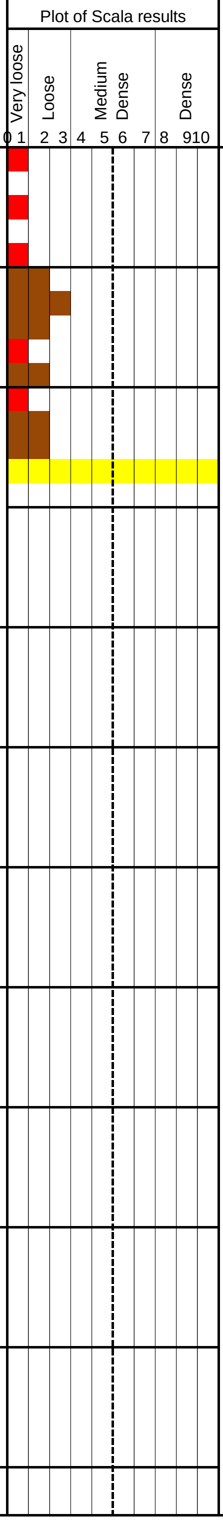
R.L. Taranaki: 935 m

Coordinates: 1694584 E, 5652684 N

Field Test Data

Scala Penetrometer (blows per 100mm drop)

Investigation method	Depth (meters)	Field Description	Geological Unit	Depth (meters)	Peak Vane Shear Strength (kPa)	Residual Vane Shear Strength (kPa)	Sensitivity	Blow count	Groundwater Level
		TOPSOIL; SILT; dark brown. Moist, non plastic.	TOPSOIL					1	
								0	
								1	
					190+	0		0	
	0.5	**SAND**, grey. Loose, moist, fine to medium grained.	FILL?	0.5				1	
								2	
								3	
								2	
								1	
	1.0			1.0				2	
								1	
								2	
								2	
		At 1.3 m - grades to dense						20+	
	1.5	**End of hand auger at 1.4 m - effective refusal** *TBA = Taranaki Brown Ash		1.5					
	2.0			2.0					
	2.5			2.5					
	3.0			3.0					
	3.5			3.5					
	4.0			4.0					
	4.5			4.5					
	5.0			5.0					
	5.5	Groundwater not encountered during testing		5.5					



Notes:
1. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.
2. OB refers to hand auger over bored. HW refers to scala falling under the weight of the hammer. TS refers to topsoil.
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6. Coordinates (where reported) are presented in decimal degrees to a accuracy of ±5m.
7. Shear vane results are multiplied by factor A and plus factor B where applicable

Job Number: 23-0183
Client: Te Atiawa

Shear Vane ID:3236
Calibration Expiry Date: 7/06/2023
Shear Vane Factors: A: 1.359

Location: North Egmont Visitors Centre Carpark

Date of investigation: 21/02/2023

Logged By: MSB **Checked By: JA**