REPORT

Tonkin+Taylor

Plan Change Liquefaction Vulnerability Assessment

Three Growth Cells within the Waihi area

Prepared for
Hauraki District Council

Prepared by
Tonkin & Taylor Ltd

Date
October 2023

Job Number
1090294 v1.1

Together we create and sustain a better world
www.tonkintaylor.co.nz

Document control

Title: Plan Change Liquefaction Vulnerability Assessment					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
August 23	1	Final	J. Hong	J. Brzeski	C. Davanna
17/10/23	1.1	Updated Final to capture client feedback	J. Hong	J. Brzeski	C. Davanna

Distribution:

Hauraki District Council 1 electronic copy

Tonkin & Taylor Ltd (FILE) 1 electronic copy

Table of contents

Client summary

The table below summarises the key findings and considerations for three proposed Growth Cells in the Waihi area. We understand this report will assist the plan change application for these three Growth Cells. The information presented must be read in conjunction with the relevant detail in the report's main body.

Ford Road Cell (industrial) – Site A

<table>
<tr><th>Consideration</th><th>Background</th><th>Undulating hills</th><th>Alluvium channel</th></tr>
<tr><td>Level B liquefaction vulnerability assessment [refer Section 3, and 4].</td><td>Based on geomorphic study, and interpretation of CPT and hand auger investigations.
Importance Level 2 and 3, with 50 or 100-year design life.</td><td>Low to Medium liquefaction vulnerability.
This terrain is more likely to fall within the low category, this should be confirmed at the Subdivision stage</td><td>Low to medium vulnerability.
Any loose, saturated sands that are susceptible to liquefaction are likely to be removed during the development of the site, alternatively, this feature may be retained as part of the stormwater disposal and may not be developed.</td></tr>
<tr><td>Preliminary static settlement assessment [refer Section 5].</td><td>Based on geomorphic study, preliminary geotechnical assessment results, address compressible soils.</td><td><u>Low risk</u></td><td><u>Moderate risk</u></td></tr>
<tr><td>Future works [refer Section 6].</td><td>Additional works required.</td><td colspan="2">• Confirmation of liquefaction vulnerability category at subdivision stage.
• Further assessment of the natural hazard risks should be undertaken at the subdivision stage to satisfy the requirements of s106 of the RMA (i.e., slope stability).</td></tr>
</table>

More detailed information is provided in this report in relation to liquefaction assessment and preliminary geotechnical hazard assessment.

Bradford Street Cell (residential) – Site C

Consideration	Background	Undulating hills	Alluvium lowlands	Alluvium channel	Infilled Channels
Level B liquefaction vulnerability assessment [refer Section3, and 4].	**Based on geomorphic study, and interpretation of CPT and hand auger investigations.** **Importance Level 2 and 3, with 50 or 100-year design life.**	**Liquefaction is possible.** This terrain is more likely to fall within the low category, this should be confirmed at the Subdivision stage (see key uncertainties in section **4.8.4**).	**Liquefaction is possible.** This terrain is more likely to fall within the medium category, this should be confirmed at the Subdivision stage. Alternatively, this area may be retained as part of the stormwater treatment and may not be developed.	**Liquefaction is possible.** This terrain is more likely to fall within the medium category. Any loose, saturated sands that are susceptible to liquefaction are likely to be removed during the development of the site. Alternatively, this feature may be retained as part of the stormwater disposal and may not be developed.	**Liquefaction is possible.** The extent and nature of this terrain and the soils underlying it are unknown. It is likely that they will fall within the medium vulnerability category. Any loose, saturated sands that are susceptible to liquefaction are likely to be removed during the development of the site.
Preliminary static settlement assessment [refer Section 5].	**Based on geomorphic study, preliminary geotechnical assessment results, address compressible soils.**	**<u>Low risk</u>**	**<u>Moderate risk</u>**	**<u>Moderate risk</u>**	**<u>High risk</u>**
Future Works **[refer Section 6].**	**Additional works required.**	• Confirmation of liquefaction vulnerability category at subdivision stage. • Further assessment of the natural hazard risks should be undertaken at the subdivision stage to satisfy the requirements of s106 of the RMA (i.e., slope stability). • A pond/old gully feature was observed in the alluvium lower lands/ alluvium channel area; it is recommended that this is assessed by a suitably qualified ecologist.			

More detailed information is provided in this report in relation to liquefaction assessment and preliminary geotechnical hazard assessment.

Waitete Road (low density residential) – Site D

Consideration	Background	Hills and ranges
Level B liquefaction vulnerability assessment [refer Section 3, and 4].	**Based on geomorphic study, and interpretation of CPT and hand auger investigations.** **Importance Level 2 and 3, with 50 or 100-year design life.**	**Liquefaction Damage is Possible.** The assessments indicate these soils are susceptible to liquefaction, however, there are significant uncertainties in this assessment that should be assessed at Subdivision stage. See section 4.8.4 for list of uncertainties. Deep machine drill holes and installing dual piezometers to confirm the actual the groundwater table are recommended.
Preliminary static settlement assessment [refer Section 5].	**Based on geomorphic study, preliminary geotechnical assessment results, address compressible soils.**	**Low to moderate risk**
Future Works **[refer Section 6].**	**Additional works required.**	• Confirmation of liquefaction vulnerability category at subdivision stage. • Further assessment of the natural hazard risks should be undertaken at the subdivision stage to satisfy the requirements of s106 of the RMA (i.e., slope stability).

More detailed information is provided in this report in relation to liquefaction assessment and preliminary geotechnical hazard assessment.

1 Introduction

Based on a "high" growth scenario, the Hauraki District's population is projected to increase by 8 % over the ten-year period between 2018 and 2028, from 20,650 to 22,300 people. After 2028, the population is projected to continue increasing but at a lesser rate, reaching 23,695 people by 2048. Of the three towns in the District, Waihi is forecast to remain the largest town, with a projected population of 5,840 by 2048[1].

At the end 2017, HDC undertook an analysis of the capacity of currently zoned land to accommodate growth in Waihi. There are only potentially 150 greenfield lots available for residential development and only potentially 70 lots for industrial development. Therefore, as part of the Hauraki District Growth Strategy 2050, the district is planning to provide more residential and industrial lands for Waihi in the near future[1].

Hauraki District Council (HDC) required site-specific investigation and assessment to better understand the natural hazards (in particular, the liquefaction hazard) within each block of land and how this may impact the proposed change in land use. The liquefaction assessment was required to be completed in accordance with the MBIE Planning and Engineering Guidance[2], comprising a Level B "Calibrated Desktop Assessment" for the four blocks of land that HDC are interested in rezoning. The four blocks of land were named Site A to Site D, and this report will cover Site A (Ford Road site), C (Bradford Street Site) and D (Waitete Road site) only. Site B (Wharry Road site) will be covered in a separate report[3].

HDC engaged Tonkin & Taylor Ltd (T+T) (202303IFS PO142784 dated 09 March 2023) to undertake:

- Geotechnical investigations to characterise the liquefaction vulnerability within the proposed rezoning areas – Site A, C and D only. The details of the three sites are listed in Section 1.1 below.
- Interpretation of ground conditions and assessment of liquefaction vulnerability for all three sites.
- Assess soft soil and settlement hazards for all three sites.
- Assess the effect of these hazards on the proposed residential and industrial development and on the proposed new infrastructure for all three sites.
- Outline further work, if required, to refine the understanding of liquefaction and other geotechnical risks within the proposed rezoning areas for all three sites.

This geotechnical report presents the findings of the Level-B liquefaction assessment and high-level soft soil and settlement assessment completed for use by HDC as part of Plan Changes for proposed rezoning area – Site A, C and D. The level of assessment is sufficient to characterise the liquefaction vulnerability within these three areas considered for the plan change and guide the requirements for assessment of liquefaction effects during subsequent design stages. Further testing will be required to support further development stages.

1.1 Site description

The three sites in this report that HDC is interested in rezoning are all located within Waihi. The locations of these three areas are presented in Appendix A Figure 1: Site Plan – Overview. The sites

[1] Hauraki District Growth Strategy Te Rautaki Whakatipu 2050 issued by Hauraki District Council.
[2] Ministry of Business, Innovation and Employment (MBIE) guidelines on Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE/MfE/EQC 2017).
[3] Plan Change Liquefaction Vulnerability Assessment Wharry Road Growth Cell issued July 2023 by Tonkin & Taylor Ltd., job reference number 1090294.v00.

are described below. Please note that the extents of the assessments at Site A and Site C differ from the extents described in the IFS as per the subsequent instructions of HDC[4][5] .

Site A (Ford Road Cell) covers an area of approximately 118,323 m^2 and is legally described as Section 290 Block XVI Ohinemuri Survey District, Lot 3 and Lot 4 Deposited Plan South Auckland 33318. This area is currently classified as a *Rural* zone, and HDC is planning to rezone this area as an *Industrial* zone. The site comprises undulating topography within gentle to moderately dipping southern and western-facing slopes that extend to Waimata stream approximately 20 meters from the site boundary. At present, there are a number of industrial/commercial and residential buildings presented adjacent to the southeast site boundary (shown in Figure 1.1).

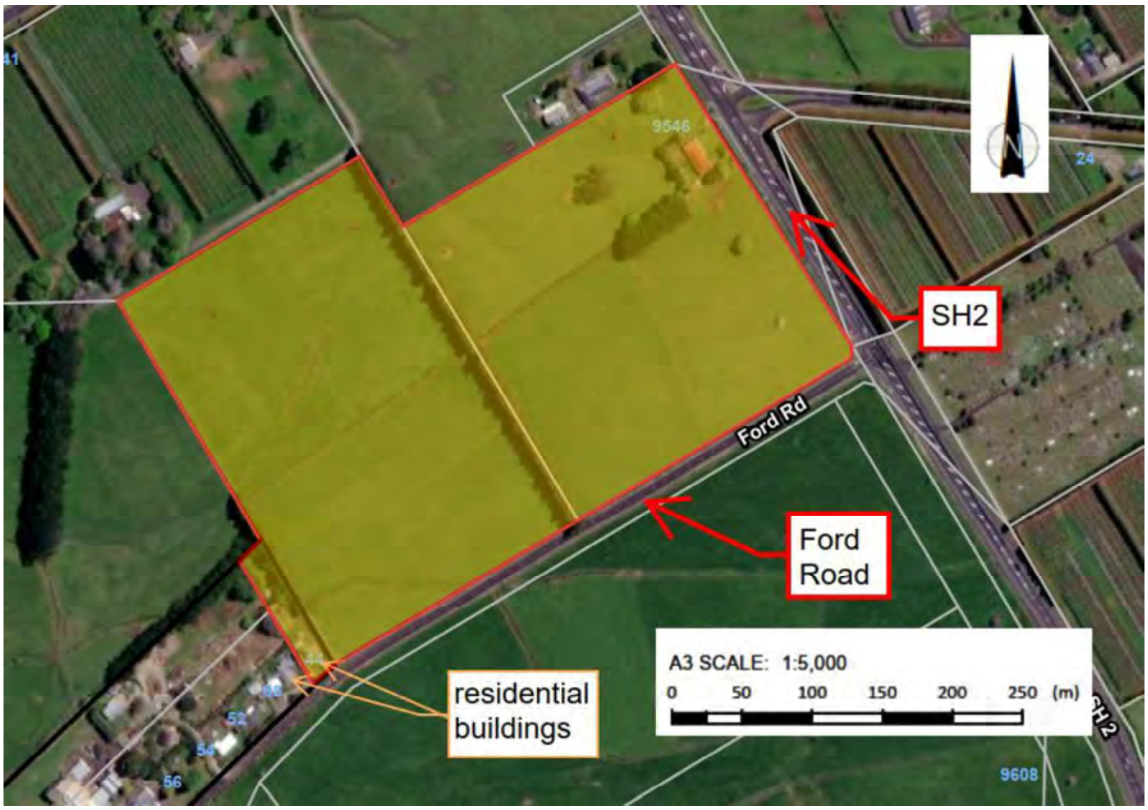


Figure 1.1: Site A Location Plan

Site C (Bradford Street Cell) covers an area of approximately 84,300 m^2 and comprises Lot 1 and part of 3 DP 478174, Lot 1, Lot 2 and Lot 3 DP 337064 and Lot 1 and Lot 2 DP 571193. This area is currently classified as a *Low Density Residential* zone, and HDC is planning to rezone this area as a *Residential* zone. The site comprises undulating topography with gentle slopes dipping towards Ohinemuri river located approximately 25 meters to the south of sites extent at elevation 78.5 m RL. A corrugated iron farming shed is situated in the northwestern corner of Lot 3 DP 337064. At the centre of Lot 3 DP 478174 is a large pond (shown in Figure 1.2). Historical aerial photographs suggest that part the rezone area has been subject to earthworks between the 1970s and 2010s as shown in Figure 1.3 to Figure 1.7. These earthworks appear to have infilled part of an old gully system culminating in the formation of a small pond between 1990s and early 2000s towards the base of the old gully feature. The council GIS shows an open drain running north to south through the

[4] Email from Leigh Robcke (Hauraki District Council) to Jia Hong on 11 April 2023 10:50 am.
[5] Email: Comments on T+T geotechnical report in relation to Waihi Plan Change Proposal, from Louise Cown (4Sight Consulting) to Jia Hong on 20 September 2023 10:02 am.

central part of the site. Historical aerial photos suggest this has been infilled in the late 2000s with the old channel being replaced by a pipe. Currently, there are several occupied residential buildings on the site.

Figure 1.2: Site C Location Plan

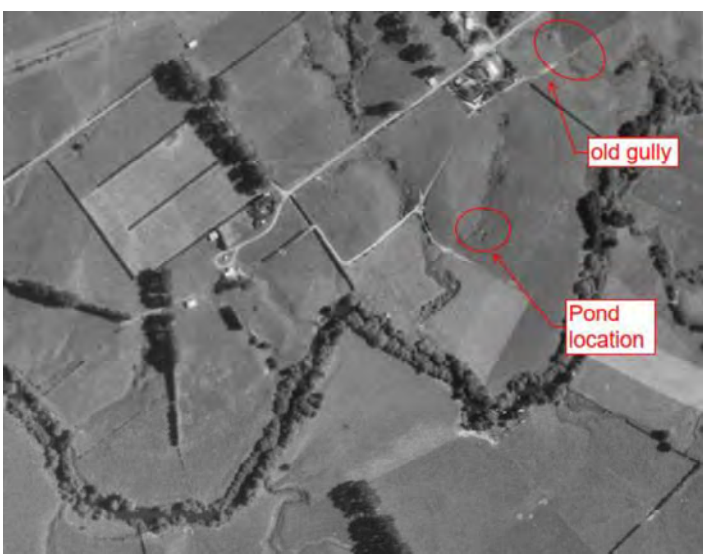


Figure 1.3: Photo from 1975

Figure 1.4: Photo from 1991

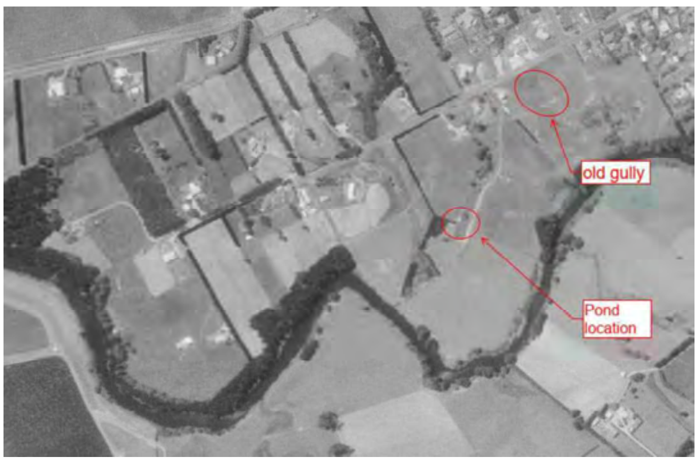


Figure 1.5: Photo from 1999

Figure 1.6: Photo from 2004

Figure 1.7: Photo from November 2016 indicated the location of the earth worked area

Photograph 1.1: Site C pond (at the centre of Lot 3 DP 478174)

Site D (Waitete Cell) overs an area of 6,079 m^2 and is legally described as LOT 6 DP 519074. This area is currently classified as a *Rural* zone. The site is situated on a relatively flat section with moderately dipping slopes on the northern and western property boundaries. The north-facing slope is approximately 9 m high and slopes towards the north on a 30° angle (shown in Photograph 1.2), and the west-facing slope is approximately 12 m high and slopes towards the west on a 20° angle (shown in Photograph 1.3). A residential building occupies this site in the south-eastern corner and accesses Thorn Road to the east (shown in Figure 1.8). Historical aerial photographs suggest that the site may have been subject to a landslide early 1940s as shown in Figure 1.9.

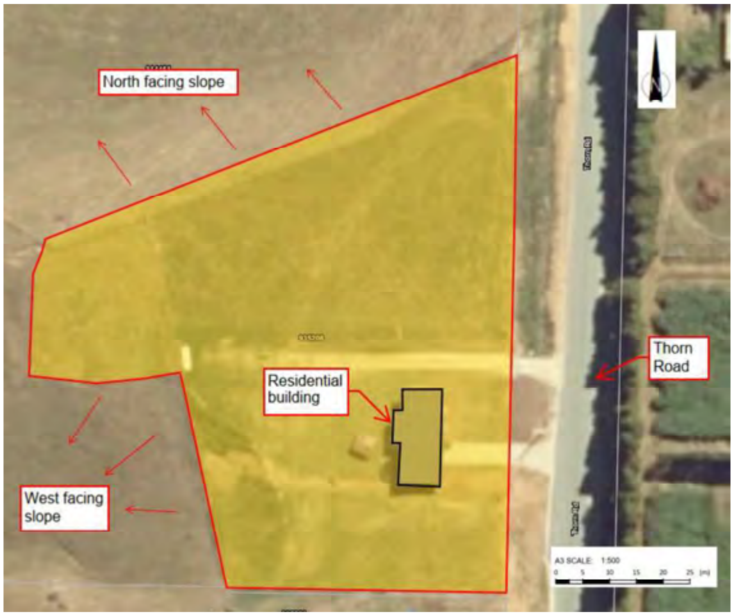


Figure 1.8: Site D Location Plan

Photograph 1.2: Site D north facing slope

Photograph 1.3: Site D west facing slope

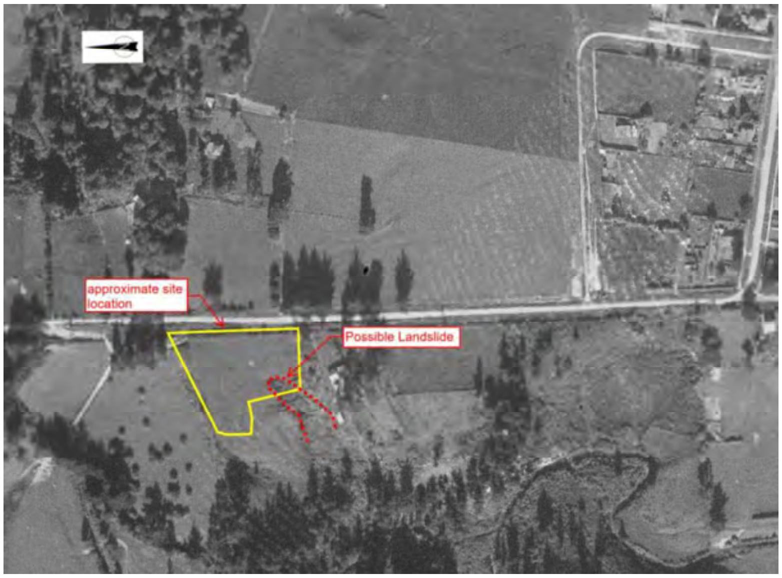


Figure 1.9: Photo from 1999

1.2 Proposed development

The proposed developments for these sites are currently unknown. But it will likely comprise Importance Level 2 & 3 structures only, including but not limited to the following:

- Residential properties (Site C and D) – for the areas that intend to rezone as residential zones.
- Industrial properties (Site A only) – for the area that is intended to be rezoned as industrial zones.
- Recreational reserves and green spaces.
- Stormwater infrastructure, including but not limited to drainage channels, soakage/attenuation basins and culverts, swales.
- Potable water supply and wastewater infrastructure.
- Roads.

2 Geotechnical information

2.1 Geology and faulting

Site A (Ford Road Cell) the published geological map of the area indicates that the site is underlain by the Ignimbrites of the Ohinemuri and Matua Subgroups. The ignimbrites are overlain by Late Pleistocene tephra. The approximate location of the site is presented in Figure 2.1 below.

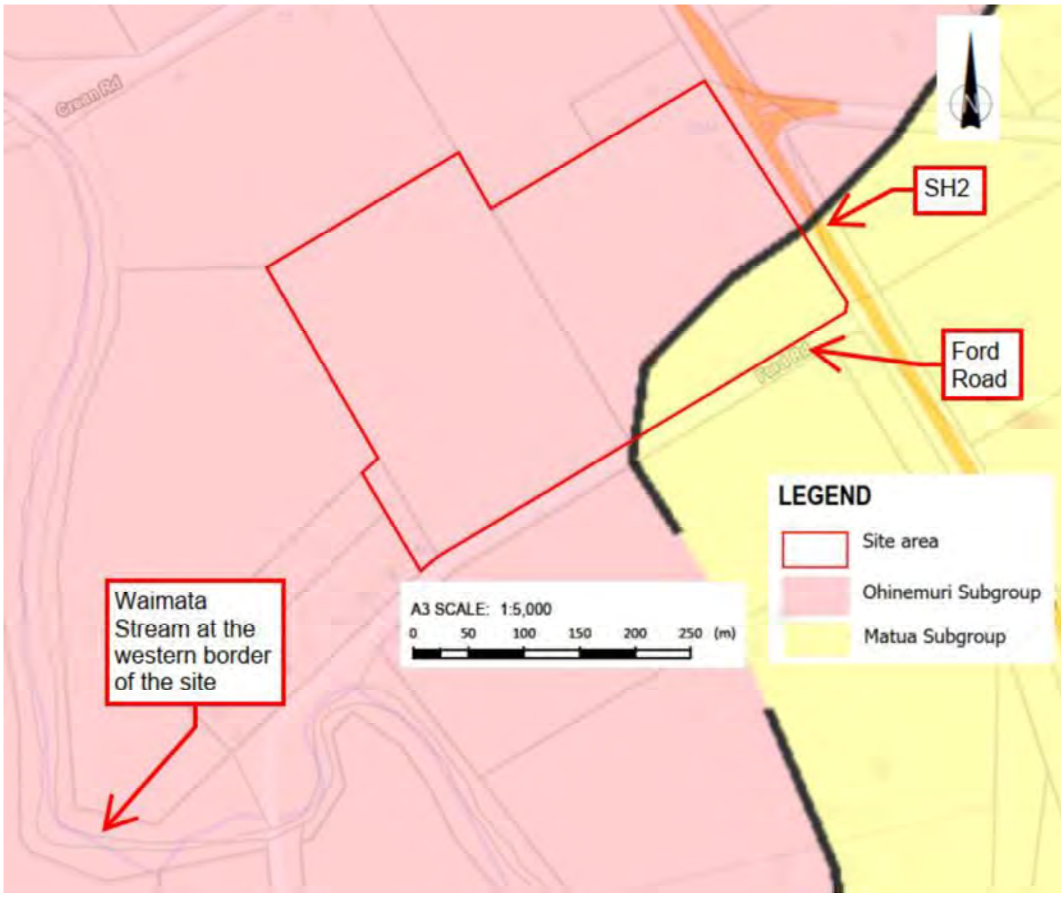


Figure 2.1: Site A geological setting (TTMapviewer, 2023, GNS Geology layer[6])

Site C (Bradford Street Cell) the published geological map of the area indicates that the majority of the site is underlain by the Whitianga Group welded, pumice-rich ignimbrite of the Ohinemuri Subgroup within the Coromandel Volcanic Zone. The map indicates that the southernmost corner of the site is underlain by Holocene sediments of the Ruahihi Formation of Tauranga Group. Late Pleistocene tephra is noted to blanket the whole sequence. The approximate location of the site is presented in Figure 2.2 below.

[6] R.L.Brathwaite & A.B.Christie 1996: Geology of the Waihi area, Institute of Geological & Nuclear Sciences 1:50,000 geological map 21. 1 sheet + 64 p. Lower Hutt, New Zealand. Institute of Geological & Nuclear Sciences Limited.

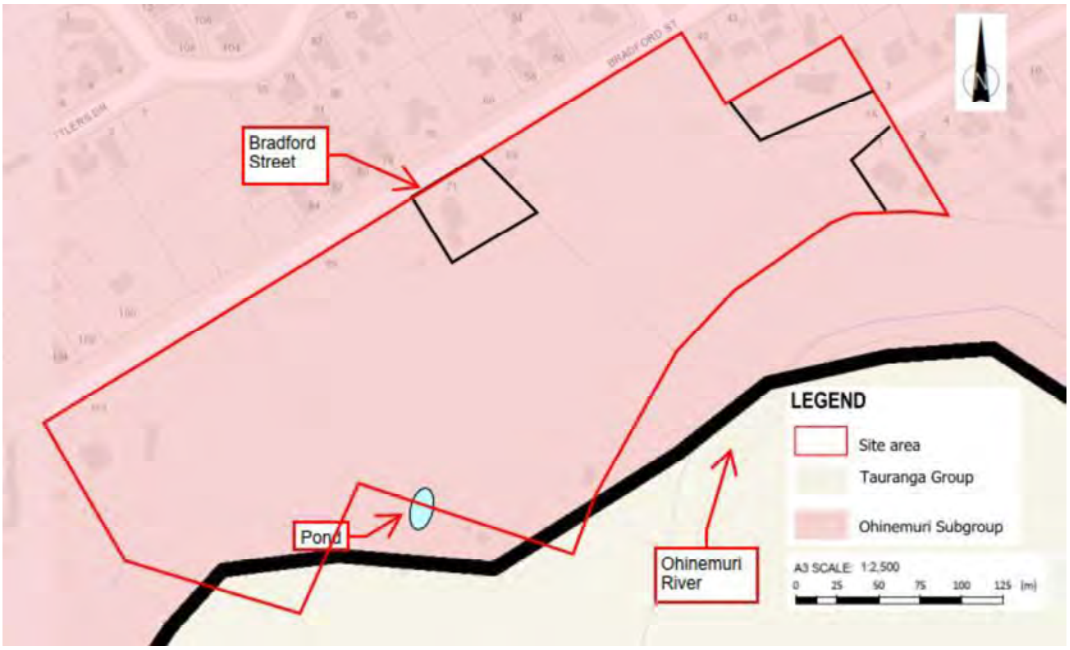


Figure 2.2: Site C geological setting (TTMapviewer, 2023, GNS Geology layer[7])

Site D (Waitete Cell) the published geological map of the area indicates that the site is underlain by the Waipapu Formation of the Waiwawa Subgroup (Coromandel Group) which is noted to comprise phyric, plagioclase-two-pyroxene andesite and dacite, locally with quartz phenocrysts and minor tuff breccia, crystal tuff and lacustrine sediments. Late Pleistocene tephra is noted to blanket the whole sequence. The approximate location of the site is presented in Figure 2.3 below.

Figure 2.3: Site D geological setting (TTMapviewer, 2023, GNS Geology layer[8])

---

[7] R.L.Brathwaite & A.B.Christie 1996: Geology of the Waihi area, Institute of Geological & Nuclear Sciences 1:50,000 geological map 21. 1 sheet + 64 p. Lower Hutt, New Zealand. Institute of Geological & Nuclear Sciences Limited.

[8] R.L.Brathwaite & A.B.Christie 1996: Geology of the Waihi area, Institute of Geological & Nuclear Sciences 1:50,000 geological map 21. 1 sheet + 64 p. Lower Hutt, New Zealand. Institute of Geological & Nuclear Sciences Limited.

2.2 Geomorphology

Geomorphic terrains have been defined and mapped using geological maps, topographic data and aerial photography to help identify landforms that could be susceptible to liquefaction. The geomorphic mapping details are presented in Table 2.1 and further discussed below, and the preliminary geomorphic maps are presented in Appendix C.

Table 2.1: Preliminary geomorphic mapping summary

Geomorphic Terrains identified	Undulating Hills	Alluvial lowlands	Alluvial channel	Hills and ranges	Infilled channel
Site A	90 %	N/A	10 %*	N/A	N/A
Site C	86 %	7 %	3 %	N/A	4 %
Site D	N/A	N/A	N/A	100 %	N/A

Note: Historic fill was identified on-site. However, we believe it is likely to be shallow, and it is expected to be underlain by alluvial channel deposits. Hence, the historic fill terrain was not included in this table, and we have considered the underlying geology instead.

These details of the identified geomorphic terrains are described as follows:

- **Undulating hills:** This geomorphic terrain represents gently undulating, elevated land. This terrain is generally associated with land that slopes less than 10 degrees (excluding the boundary of this terrain with alluvial lowlands terrain and alluvial channel terrain, which is generally associated with steeply sloping land). The undulating hills terrain typically has rock at or near the ground surface, and therefore, it is less likely to contain soils that are susceptible to liquefaction.
- **Alluvial lowlands:** This geomorphic terrain represents generally flat, low-lying land that is positioned at a lower elevation than the undulating hills terrain. This terrain has been influenced by recent fluvial processes and is likely associated with depositional environments associated with the surrounding Stream (i.e., the Ohinemuri River). This terrain may include areas of wetland and soft/loose soils. This terrain is likely to comprise Holocene-aged alluvial sediments that are susceptible to liquefaction.
- **Alluvial channel:** This channel landform feature acts as a drainage path from the undulating hills terrain into the alluvial lowlands terrain. It is likely that recent alluvial processes have formed this terrain, and as a result, this terrain could contain soft and/or loose soils that are susceptible to liquefaction.
- **Hills and ranges:** This geomorphic terrain represents steeply sloping elevated land positioned at a higher elevation than the undulating hills terrain. This terrain is generally associated with steeply sloping land (slope angles more than 10 degrees) and elevated, flat land surfaces (noting that Site D only contains the elevated flat land surface). The hills and ranges terrain typically has rock at or near the ground surface, and therefore, it is less likely to contain soils that are susceptible to liquefaction.
- **Infilled channel:** This geomorphic terrain represents land that shows some evidence of being influenced by human activities. Historical aerial imagery and site observations suggest that this terrain may be associated with historic earthworks/filling. We consider it likely that the Alluvial channel terrain underlies the Infilled channel terrain, and as a result, this terrain could contain soft and/or loose soils that are susceptible to liquefaction and settlement.

A T+T geotechnical engineer visited Site D on 28 March 2023. Site walks were also undertaken based on the previous instruction in the permitted area within or adjacent to Site A and Site C between March and April. The site walkover observations at site D generally support the geomorphic desktop study findings for each site listed above. Following the site walkover, aerial photography and contour data were used for sites A and C to extend the geomorphic mapping to the amended areas.

Investigations at Site C and the adjacent Site A encountered shallow refusals within 3 m below the surface. Investigations completed at Site D were refused at a deeper depth due to the thick residual soil layer. However, most of the sites were covered by ankle-height vegetation with some scattered trees; therefore, some geomorphic features may have been obscured. No evidence of significant ground instability, such as landslide scarps and tension cracks, was visible at the time of the site walkover.

For sites A and C, the lower areas in the alluvial lowlands and alluvial channel geomorphic zones presented potential wetland features. It is our understanding that HDC has engaged a suitably qualified ecologist to assess these areas.

3 Project specific investigations

Geotechnical investigations were carried out at the project site on 27 to 28 March and 13 to 14 April 2023. The investigations comprised:

Site A

- Two hand augered boreholes.
- Two cone penetration tests (CPTs).

Site C

- Three hand augered boreholes.
- Five cone penetration tests (CPTs).

Site D

- Two hand augered boreholes.
- Two cone penetration tests (CPTs).

Actual investigation locations were selected by T+T based on access and presence of buried services.

Based on the MBIE Planning and Engineering Guidance, a minimum total number of three deep investigations are required for each geological sub-unit for a Level B assessment. Therefore, the deep investigations at Site A and Site D were slightly under the required number due to issues with access approvals and changes to the site extents. However, we believe there is a sufficient number of tests for the required development scenario for the following reasons:

Site A

No investigations have been undertaken within the updated extents of Site A. Two CPTs and two hand augers were undertaken within the area of the original site extents. Based on the geomorphic mapping and predominant geological unit, these are expected to be representative of the updated extents of Site A.

In addition, a further two CPTs located 750 m away from the site in the same geological unit and geomorphic terrain are available on the New Zealand Geotechnical Database (NZGD). These indicate similar results to the investigations undertaken adjacent to the Site A.

Site D

The site is relatively small and is formed of a consistent geological unit within the same geomorphic terrain. Investigations indicate similar performance.

Plans of completed investigations for three different sites are presented in Figure A-1, C-1 and D-1, Appendix A. The locations of the investigations were determined using handheld GPS provided by WSP and T+T staff. The elevations of the investigations were estimated using HDC Lidar information.

The following sections describe and summarise the geotechnical investigations undertaken for this project.

3.1.1 Hand augered boreholes

The drilling of seven hand augered boreholes across three different sites was undertaken on 27 to 28 March and 14 April 2023. The works were carried out by T+T geotechnical engineers. In situ shear strength testing was undertaken at 0.5 m intervals throughout the soil horizon.

Please note that the hand auger numbering is not continuous, and a few numbers have been omitted. This is due to some planned tests that could not be undertaken due to site access issues at the time of the investigation.

Investigation locations are presented in Figure A-1, C-1 and D-1, Appendix A. Summary borehole logs are presented in Appendix B. A summary of the hand auger borehole details is presented in Table 3.1 below.

Table 3.1: Hand augered borehole summary

HA ID	Location (NZTM)		Ground Surface Elevation* RL (m)	Depth (m)
	Easting (m)	Northing (m)		
A HA03	1852621	5855704	99.5	2.6
A HA04	1852640	5855416	96	3.1
C HA01	1850995	5856868	93.5	2.0
C HA02	1850951	5856685	93	2.8
C HA03	1850864	5856725	87	3.0
D HA01	1850182	5858672	140	2.5
D HA02	1850234	5858621	140	4.0
* Elevations are presented in Moturiki 1953 Datum based on HDC Lidar information.				

3.1.2 Cone Penetration Tests

The pushing of nine Cone Penetration Tests (CPTs) across three sites was undertaken by WSP Global from 13 to 14 April 2023. All the CPTs undertaken at Site A and Site C hit refusal at shallow depths between 0.5 m and 9.6 m due to the cone terminating on or within hard, impenetrable strata such as rock or a dense sand layer. Second drilling attempts were undertaken at the CPT04 location due to CPT04 refusal at 0.5 m bgl, and subsequently, CPTC04A was advanced to 9.6m depth. Site D investigations reached greater depths with CPTD01 having refusal at 17.6m bgl, and CPTD02 meeting the target depth of 20 m.

PVC standpipe piezometers were installed in four of the CPT holes (CPTA04, CPTC03, CPTC06 and CPTD02). The PVC standpipe piezometers were installed to monitor the potential perched water and regional groundwater levels. The monitoring details will be discussed in the next section.

The CPT locations are presented on Figure A-1, C-1 and D-1, Appendix A. CPT logs are appended in Appendix B. A summary of the CPTs and termination depths is presented in Table 3.2 below.

Table 3.2: CPT Summary

CPT ID	Location (NZTM)		Ground Surface Elevation* RL(m)	Termination depth (m)	Reason for termination
	Easting (m)	Northing (m)			
CPTA04	1852626	5855704	99.5	6.6	Tip resistance (10t)
CPTA05	1852640	5855416	96	3.1	Tip resistance (qc 40+ MPa)
CPTC01	1850966	5856901	91.5	8.7	(qc/inclination)
CPTC02	1850995	5856868	91	6.9	(inclination)
CPTC03	1850949	5856682	86.5	5.5	Tip resistance (10t)
CPTC04	1850879	5856749	89	0.5	(qc/inclination)
CPTC04A	1850880	5856751	89	9.6	Tip resistance (10t)
CPTC06	1850875	5856827	94	4.5	Tip resistance (10t)
CPTD01	1850182	5858672	140	17.6	Tip resistance (10t)

CPT ID	Location (NZTM)		Ground Surface Elevation* RL(m)	Termination depth (m)	Reason for termination
	Easting (m)	Northing (m)			
CPTD02	1850234	5858621	140	20	Target depth
* Elevations are presented in Moturiki 1953 Datum as determined using HDC Lidar information. Please note that the elevations on the logs were based on handheld GPS and may not be consistent with those listed in this table.					

3.1.3 Groundwater monitoring

3.1.3.1 Groundwater levels

Standpipe piezometers were installed in four of the CPT holes. Groundwater levels within the piezometers were recorded using an electronic dip meter. The recorded groundwater levels are presented below in Table 3.3. The groundwater levels were recorded on 11 May 2023 after prolonged rainfall.

Table 3.3: Groundwater levels measured within standpipe piezometers

CPT hole ID	Piezometer installation depth (mbgl)	Groundwater depth (mbgl) and groundwater elevation in bracketed values (mRL) *	
		27/04/2023	11/05/2023
CPTA04	2.6 m	2.6 m (96.9 mRL)	1.6 m (97.9 mRL)
CPTC03	2.5 m	2.5 m (84 mRL)	Damaged by landowner
CPTC06	2.6 m	2.6m (91.4 mRL)	Damaged by landowner
CPTD02	2.6 m	2.6 m (137.4 mRL)	2.0 m (138 mRL)
*Elevations are presented in Moturiki 1953 Datum as determined by using HDC Lidar information.			

The groundwater levels monitored in the shallow standpipe piezometers were checked against the groundwater levels encountered within the hand auger holes. At sites A and C, the groundwater levels measured within standpipe piezometers are broadly consistent with those measured in hand augers and site observations. Therefore, a groundwater table between 2.6 m and 2.8 m bgl was adopted for Site A and a groundwater table between 1.4m and 2.8m bgl was adopted for Site C.

At site D, due to the low permeable layers encountered below the bottom of the piezometer installed at CPTD02, the groundwater table recorded in CPTD02 will likely be perched. We believe the permanent groundwater level is likely to be controlled by the Waitete Stream approximately 20 m below the site, however, this cannot be confirmed by hand augers or CPTs. We suggest that additional investigation should be undertaken to confirm the static groundwater level for Site D by undertaking machine drilled boreholes and installing dual piezometers. The hand auger investigation indicated the standing groundwater table is likely to be below 4 m bgl. Therefore, a groundwater table at 4 m was adopted for the liquefaction assessment at site D.

The groundwater levels which were dipped straight after drilling the hand auger boreholes and pushing the CPTs are recorded in Table 3.4.

Table 3.4: Groundwater levels measured straight after drilling/pushing

Investigation ID	Date dipped	Dipped Water Level depth (mbgl)	Dipped Water Level elevation (mRL)
CPTA04	13/04/2023	2.4 (Dry)	Below 97.1
CPTA05	13/04/2023	2.8	93.2
CPTC01	14/04/2023	1.5	90

Investigation ID	Date dipped	Dipped Water Level depth (mbgl)	Dipped Water Level elevation (mRL)
CPTC02	14/04/2023	1.5	89.5
CPTC03	14/04/2023	2.8 (Dry)	Below 83.9
CPTC04	14/04/2023	0.5 (Dry)	Below 88.5
CPTC04A	14/04/2023	1.4	87.6
CPTC06	14/04/2023	2.1 (Dry)	Below 91.9
CPTD01	13/04/2023	1.0 (Dry)	Below 139.5
CPTD02	13/04/2023	2.5 (Dry)	Below 137.5
A HA03	13/04/2023	2.6 (Dry)	Below 96.9
A HA04	28/03/2023	3.0 (Dry)	Below 93
C HA01	27/03/2023	2.0 (Dry)	Below 91.5
C HA02	28/03/2023	1.8	91.2
C HA03	28/03/2023	1.8	85.2
D HA01	27/03/2023	2.5 (Dry)	Below 137.5
D HA02	27/03/2023	4.0 (Dry)	Below 136
*Elevations are presented in Moturiki 1953 vertical datum as determined by using HDC Lidar information.			

3.1.4 Geotechnical model

Table 3.5, Table 3.6 and Table 3.7 below present the soil profiles inferred from the available geotechnical data for the three sites discussed in this report.

Table 3.5: Inferred soil profile – Site A

Layer No.	Unit	Description	Depth to top of layer (m)	Layer thickness (m)	Cone resistance (MPa)	Shear Vane or Scala Value
1	Topsoil	Black organic silt.	0.0	0.2 to 0.3	-	-
2	ASH - LATE PLEISTOCENE TEPHRA	Comprises interbedded firm to very stiff SILTs and clayey SILTs.	0.2 to 0.3	2.0 to 2.5	1 to 3	42 to 184(1)
3a	WHITIANGA GROUP	Interbedded silts and sands.	2.0 to 2.5	2.0 to 2.5	3 to 10+	100 to >500(2)
3b		Possible ignimbrite.	Below 3.0 to 5.0	-	CPT was refused at this layer.	-

Notes

Depths and thicknesses have been rounded to nearest 0.5 m due to spatial variability in depth and thickness except the first Topsoil layer.

(1) Shear strength measured using in-situ shear vane testing.

(2) Shear strength inferred from CPT results.

Site A generally consists of topsoil overlying Late Pleistocene Tephra deposits to a depth of up to 2.5m bgl. Underlying this is a 2 to 2.5 m thick layer of interbedded silts and sands interpreted as being residual soils derived from the Whitianga group. Investigations refused at depths beyond this, with refusals interpreted as being due to encountering the underlying slightly weathered ignimbrite.

Table 3.6: Inferred soil profile – Site C

Layer No.	Unit	Description	Depth to top of layer (m)	Layer thickness (m)	Cone resistance (MPa)	Shear Vane or SPT N value
1	Topsoil/Fill	Black organic silt and silt backfill material.	0.0	0.1 to 1.8	-	-
2	ASH - LATE PLEISTOCENE TEPHRA	Comprises interbedded firm to very stiff SILTs, clayey SILTs and some localised sands/silty sand layers.	0.1 to 1.8	2 to 4.5	0.3 to 2	73 to >200(1)
3a	WHITIANGA GROUP	SILTs and clayey SILTs (residual soils) from Whitianga group.	2 to 4.5	2.0 to 3.5	2 to 10+	150 to >200(2) /10 to 30 (3)
3b		Inferred dense layer (likely to be dense sand or weathered ignimbrite).	Below 4.5 to 8.5	-	CPT was refused at this layer	-

Notes

Depths and thicknesses have been rounded to nearest 0.5 m due to spatial variability in depth and thickness except the first Topsoil layer.

(3) Shear strength measured using in situ shear vane testing.

(4) Shear strength inferred from CPT results.

(5) SPT N60 value for localised sands/silty sands, inferred from CPT results.

Site C appears to have undergone periods of filling at various times, which has altered the topography. The areas of fill are shown on the geomorphic map included in Appendix C and are generally in old gully/stream systems. The fill's full extent and nature are unknown and will require additional investigation as part of any future development. These areas have variable conditions and consist of topsoil overlying Late Pleistocene Tephra deposits to a depth of up to 3m bgl. Underlying this is a 2 to 2.5 m thickness of interbedded silts and sands interpreted as being residual soils derived from the Whitianga group. Investigations refused at depths beyond this, with refusals interpreted as being due to encountering the underlying slightly weathered ignimbrite.

Table 3.7: Inferred soil profile – Site D

Layer No.	Unit	Description	Depth to top of layer (m)	Layer thickness (m)	Cone resistance (MPa)	Shear Vane or Scala Value
1	Topsoil/Fill	Black organic silt and silt backfill material.	0.0	0.2 to 0.3	-	-
2	ASH - LATE PLEISTOCENE TEPHRA	Comprises interbedded firm to very stiff SILTs and clayey SILTs with localised peat pockets and buried tree roots (Possible paleosol).	0.2 to 0.3	2.0 to 2.5	1 to 3	84 to >200(1)

Layer No.	Unit	Description	Depth to top of layer (m)	Layer thickness (m)	Cone resistance (MPa)	Shear Vane or Scala Value
3a	COROMANDEL GROUP	Interbedded CLAYS and silt CLAYS residual soils.	2.5 to 4.0	7.0 to 12.0	1 to 3	100 to >300(2)
3b		Interbedded CLAYS, SILTS, SANDS and mixtures residual soils.	7.0 to 12.0	17.5 to >20	4 to 8	200 to >500(2)
3c		Possibly weathered Waiwawa Subgroup.	Below 17.5 to >20	-	CPT was refused/terminated at this layer	-

Notes

Depths and thicknesses have been rounded to nearest 0.5 m due to spatial variability in depth and thickness except the first Topsoil layer.

(1) Shear strength measured using in situ shear vane testing.

(2) Shear strength inferred from CPT results.

Site D generally consists of Topsoil overlying Late Pleistocene Tephra deposits to a depth of up to 2.5m bgl. Underlying this is a 15 to 18+ m thickness of interbedded silts and sands layer interpreted as being residual soils derived from the Coromandel group. The soil strength increases versus the depths. Investigations refused at depths beyond this, with refusals interpreted as being due to encountering the underlying weathered volcanic rocks belonging to the Waiwawa subgroup. One investigation encountered peat/organic soil at a depth of 2.0m bgl. This is interpreted as a buried topsoil (paleosol) at the base of the recent ash deposits.

4 Preliminary site specific liquefaction assessment

4.1 Active faults

According to GNS[9] the nearest active fault, the Kerepehi Fault, is approximately 20 km west of all three sites.

4.2 Seismic site subsoil class

The determination of Subsoil Class in order to derive site seismic hazard for the purposes of liquefaction assessments is no longer necessary given the updated version of the MBIE Earthquake Engineering Guidance Module 1. For the purposes of plan change, this parameter is not considered to be required. Future developments of the three sites are likely to require an assessment of the shear wave velocity in the top 30m (Vs30) to determine the seismic hazard design parameters from the 2020 New Zealand National Seismic Hazard Model.

4.3 Seismic shaking hazard

The details of the proposed developments for Site A, C and D are currently unknown. But it will likely comprise Importance Level 2 and 3 structures only. Some future infrastructure may require to be designed equivalent to Importance Level (IL) 3 in accordance with AS/NZS 1170.0 with a 100-year design life.

A seismic assessment has been undertaken for the proposed development in accordance with recommendations in the MBIE Module 1 (2021)[10] and the New Zealand Code of Practice NZS 1170.5:2004 to represent the following design performance requirements. The design earthquake cases are summarised in Table 4.1 below.

Table 4.1: Seismic assessment inputs

Earthquake design case	Return period (years)	PGA (g)	Magnitude (M)
Serviceability Limit State (IL2 & 3 SLS case)	25	0.07	5.9
Ultimate Limit State (IL2 ULS case)	500	0.30	5.9
Ultimate Limit State (IL3 ULS case)	2,500	0.53	5.9

4.4 Liquefaction process

It can be readily observed that dry, loose sands and silts contract in volume if shaken. However, if the loose sand is saturated, the soil’s tendency to contract causes the pressure in the water between the sand grains (known as “pore water”) to increase. The increase in pore water pressure causes the soil’s effective grain-to-grain contract stress (known as “effective stress”) to decrease. The soil loses strength as this effective stress is reduced. This process is known as liquefaction.

4.5 Liquefaction vulnerability assessment methodology

The liquefaction vulnerability assessment has been undertaken in general accordance with a Level B assessment as described in MBIE Planning and Engineering Guidance[11]. In that document a Level B

[9] https://data.gns.cri.nz/af/
[10] MBIE (2021). Earthquake geotechnical engineering practice, Module 1: Overview of the guidelines, November 2021.
[11] Ministry of Business, Innovation and Employment (MBIE) guidelines on Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE/MfE/EQC 2017)

assessment is described as a calibrated desktop assessment with a small number of subsurface investigations providing a better understanding of liquefaction susceptibility for the study areas. For the purposes of this study, areas have been classified in terms of each of the five geomorphic zones. These zones are then assigned a liquefaction vulnerability classification as described below.

The methodology described In the MBIE Planning and Engineering Guidance[8] recommends categorisation of the liquefaction vulnerability of the land based on the performance criteria described in Figure 4.1 below.

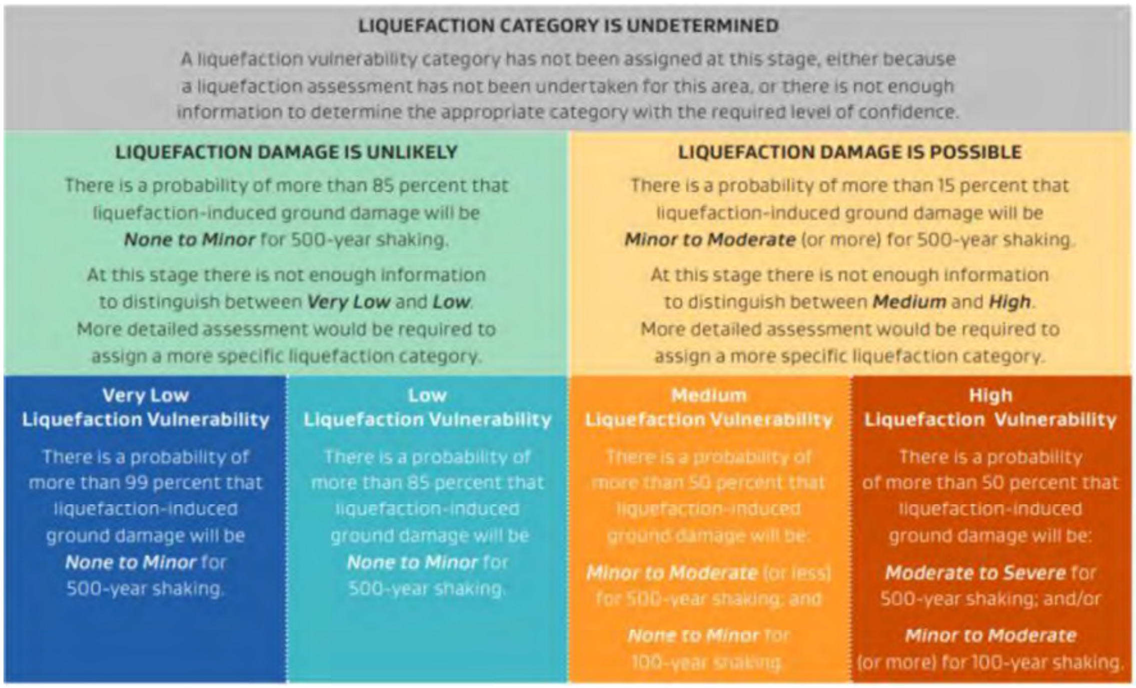


Figure 4.1: Performance criteria for determining the liquefaction vulnerability category – reproduced from Table 4.4 of MBIE/MfE/EQC (2017)

4.6 Site specific liquefaction assessment – Site A

T+T in-house software was used to evaluate the liquefaction susceptibility of the site using the trigger methods recommended by Boulanger & Idriss (2014)[12].

Based on the geomorphic study, it has been assumed that the soils within alluvial channels will either be removed, or the areas avoided for development, therefore the subsurface investigations were carried out in the undulating hills areas to confirm the liquefaction vulnerability of this terrain. The alluvial channels are expected to be susceptible to liquefaction as well as having soft or loose soils. The historic fill was identified on-site. However, we believe the historic fill is likely to be shallow, and it is expected to have alluvial channel deposits under the fill.

The assessment adopted a groundwater level based on the groundwater monitoring results. A sensitivity assessment was also undertaken for the IL2 ULS case by raising the groundwater table by 1 m[13].

[12] Boulanger, R. W., & Idriss, I. M. (2014). *CPT and SPT based liquefaction triggering procedures.* Centre for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA. Report No. UCD/CGM-14/01.

[13] For the areas where the dipped groundwater table is shallower than 1.0 m, the sensitivity assessment was completed by adopting the groundwater table at 0.1m bgl surface.

Liquefaction assessment was undertaken using data from the two CPTs from the NZGD database and the two CPTs undertaken on the previous extents of Site A, as detailed in section 3.1.2. The groundwater table adopted for the additional liquefaction assessment was based on the groundwater monitoring results from the site adjacent to Site A..

Results of our assessment of Site A by utilising nearby investigation results within the same geomorphic zone are presented in Appendix D. The liquefaction susceptibility for Site A is discussed in the following sections.

4.6.1 Liquefaction susceptibility

Negligible liquefaction is assessed to occur at SLS level of shaking for site A.

A summary of the crust thickness, LSN, and expected settlement under IL2 and IL3 ULS levels of shaking for site A is presented on Table 4.2 below.

Table 4.2: ULS liquefaction assessment results summary – Site A

CPT ID	IL2 ULS – 1 in 500			IL2 ULS – 1 in 500 (increase groundwater table by 1m)			IL3 ULS – 1 in 2500		
	Crust thickness, CT (m)	Liquefaction Severity Number, LSN[2]	Free-field Surface Settlement[1], Sv1D (mm)	Crust thickness, CT (m)	Liquefaction Severity Number, LSN[2]	Free-field Surface Settlement[1], Sv1D (mm)	Crust thickness, CT (m)	Liquefaction Severity Number, LSN[2]	Free-field Surface Settlement[1], Sv1D (mm)
CPTA04	2.7	7	25	2.1	15	40	2.7	9	30
CPTA05[3]	3.0	0	0	1.9	12	30	2.9	1	5
CPT 153850	5.2	0	5	5.2	0	5	5.2	0	5
CPT 153851	8.2	0	10	8.2	1	10	7.5	2	20

Notes:

1 Rounded to the nearest 5 mm due to inherent uncertainties associated with the calculation methods.

2 LSN values typically represent the following consequences (From MBIE Module 3[14] Table 5-1):

- L0, Insignificant (LSN<10, LPI=0): No significant excess pore water pressures (no liquefaction).
- L1, Mild (LSN 5-15, LPI=0): Limited excess pore water pressures; negligible deformation of the ground and small settlements.
- L2, Moderate (LSN 10-25, LPI<5): Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.

3 CPTA05 terminated at 3.0 m bgl. Liquefiable materials may be present below this depth.

Key findings of the Site A liquefaction assessment:

- Liquefaction assessment indicates under design ULS case LSN values are typically less than ten, and free field settlement estimates are typically between 0 and 30 mm.
- A significant part of the soils is plastic and not considered susceptible to liquefaction.
- Shallow refusals were encountered across the site. Based on the geomorphic study, these refusals are likely taken to be the surface of the underlying volcanics.
- The additional liquefaction assessment utilising the CPTs from the vicinity site showed similar results as the site-specific CPTs.

[14] MBIE (2021). Earthquake geotechnical engineering practice, Module 3: Identification, assessment and mitigation of liquefaction hazards-Version 1, November 2021.

4.6.2 Lateral spreading

Given that the upper soils are not considered to the susceptible to liquefaction, lateral spreading to the free face of the stream is not considered likely. See section 4.6.4 for uncertainties to be confirmed at the time of subdivision.

4.6.3 Cyclic softening

A cyclic softening assessment has been undertaken using Idriss and Boulanger (2007)[15]. The results indicate that the firm to stiff layer encountered in CPTA04 between 1.7 m and 2.0 m depth would need to be less than 20 kPa to trigger cyclic softening at the design shaking levels. The shear strength measurement indicates greater than 25 kPa, inferred from CPT results and over 100 kPa from in situ shear vane tests.

Therefore, the cyclic softening risk is considered low for this site.

4.6.4 Site A liquefaction vulnerability

The preliminary liquefaction assessment indicated that site A falls in to a Low to Medium liquefaction vulnerability category. It is likely that the alluvial channel will be medium, whereas the undulating hills are more likely to be low. The uncertainties in these assessments are explained below and should be confirmed at the time of subdivision consent:

- **Investigation density:** Following the MBIE Planning and Engineering Guidance document, there is a sufficient density of investigations (two site-specific deep investigations on adjacent land plus two deep investigations from the vicinity site on the same geology) to determine the liquefaction vulnerability of the site; however, following MBIE module 2; further investigations would be required at the subdivision stage.
- **Susceptibility of upper soils:** The soils in the upper 2-3 m have been judged to be non-susceptible on account of the predicted plasticity from CPT data. Soil description of this material is low to medium plasticity. In order to remove this uncertainty, further testing such as Plasticity Index testing is recommended to confirm the CPT findings. Once Site A access was obtained, CPT testing within in Site A boundary is also recommended to confirm the ground behaviour.
- **Ground water depths**: Confirmation of groundwater depths including consideration of seasonal variation, should be provided with subdivision consent applications.
- **Geological Unit:** No investigations have been undertaken with the mapped geological unit of the Matua Subgroup. It is expected that this unit will have similar performance to the Ohinemuri subgroup, however, due the Matua Subgroup's younger age, it may have increased liquefaction vulnerability.
- **Age of soils**: It has been shown that liquefaction resistance increased in older soils. In order to assess this specialist in situ testing and subsequent analysis will be required. A cost benefit analysis of undertaking this work should be completed at the subdivision stage to determine if aging effects are likely to reduce the liquefaction vulnerability of these soils.

Table 4.3 has been provided below to comply with the data requirements of the MBIE planning and engineering guidance for potentially liquefaction-prone land.

Table 4.3: Liquefaction summary information table

LIQUEFACTION ASSESSMENT SUMMARY

[15] Boulanger and Idriss (2007), Evaluation of Cyclic Softening in Silts and Clays

This liquefaction assessment has been undertaken in general accordance with the guidance document 'Assessment of Liquefaction-Induced Ground Damage to Inform Planning Processes' published by the Ministry of Business, Innovation and Employment in 2017. https://www.building.govt.nz/building-code-compliance/geotechnical-education	
Client	Hauraki District Council
Assessment undertaken by	Tonkin + Taylor Ltd
Report date	10/10/23
Extent of the study area	See Figure 1.1: Site A Location Plan
Intended RMA planning and consenting purposes	Plan Change
Other intended purposes	Subdivision, subject to further assessment regarding investigation density, plasticity of upper soils and seasonal variation in groundwater
Level of detail	Level B
Notes regarding base information	-
Other notes	-

4.7 Site specific liquefaction assessment – Site C

T+T in-house software was used to evaluate the liquefaction susceptibility of the site using the trigger methods recommended by Boulanger & Idriss (2014)[16].

Based on the geomorphic study, alluvium lowlands and alluvium channels are likely to be susceptible to liquefaction. Therefore, the subsurface investigations were mainly carried out in the undulating hills area (CPTC01 to CPTC03 and CPTC06) to confirm the liquefaction vulnerability of this terrain, and 1 CPT (CPTC04A) was carried out in the alluvial lowlands to understand the liquefaction susceptibility in this terrain better.

The assessment adopted a groundwater level based on the groundwater monitoring results. A sensitivity assessment was also undertaken for the IL2 ULS case by raising the groundwater table 1m[17].

Results of our assessment of site C are presented in Appendix A. The liquefaction susceptibility for all sites is discussed in the following sections.

4.7.1 Liquefaction susceptibility

Negligible liquefaction is assessed to occur at SLS level of shaking for Site C.

A summary of the crust thickness, LSN, and expected settlement under IL2 and IL3 ULS levels of shaking for Site C is presented on Table 4.4 below.

Table 4.4: ULS liquefaction assessment results summary – Site C

CPT ID	IL2 ULS – 1 in 500	IL2 ULS – 1 in 500 (increase groundwater table by 1m)	IL3 ULS – 1 in 2500

[16] Boulanger, R. W., & Idriss, I. M. (2014). *CPT and SPT based liquefaction triggering procedures.* Centre for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA. Report No. UCD/CGM-14/01.

[17] For the areas where the dipped groundwater table is shallower than 1.0 m, the sensitivity assessment was completed by adopting the groundwater table at 0.1m bgl surface.

	Crust thickness, CT (m)	Liquefaction Severity Number, LSN²	Free-field Surface Settlement¹, Sv1D (mm)	Crust thickness, CT (m)	Liquefaction Severity Number, LSN²	Free-field Surface Settlement¹, Sv1D (mm)	Crust thickness, CT (m)	Liquefaction Severity Number, LSN²	Free-field Surface Settlement¹, Sv1D (mm)
CPTC01	3.5	14	70	3.5	14	70	3.5	14	75
CPTC02	2.4	6	30	0.8	15	45	2.4	11	50
CPTC03	3.0	6	25	2.1	12	40	2.6	13	50
CPTC04A	1.8	16	70	1.8	20	80	1.8	20	95
CPTC06	4.4	0	< 5	4.4	0	< 5	2.7	0	5

1 Rounded to the nearest 5 mm due to inherent uncertainties associated with the calculation methods.

2 LSN values typically represent the following consequences (From MBIE Module 3[18] Table 5-1):

- L0, Insignificant (LSN<10, LPI=0): No significant excess pore water pressures (no liquefaction).
- L1, Mild (LSN 5-15, LPI=0): Limited excess pore water pressures; negligible deformation of the ground and small settlements.
- L2, Moderate (LSN 10-25, LPI<5): Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.

Key findings of the Site C liquefaction assessment:

- Liquefaction assessment indicates under design ULS case LSN values are typically between 0 and 20, and free field settlement estimates generally are between 0 and 95 mm.
- A significant part of the soils (the upper 2.5-3 m soils) within the undulating hills terrain are plastic and not considered susceptible to liquefaction. The soils below this depth are considered mildly susceptible to liquefaction under the designed ULS events.
- The soils are moderately susceptible to liquefaction under the design ULS events in the alluvial lowlands and alluvial channel terrains.
- The infilled channels are likely to be variable in nature, for the purposes of this assessment it is assumed that they will be susceptible to liquefaction.
- Shallow refusals were encountered across the site. Based on the geomorphic study, these refusals are likely taken to be the surface of the underlying volcanics.

4.7.2 Lateral spreading

Given that the upper soils within the undulating hills are considered slightly to mildly susceptible to liquefaction, lateral spreading to the free face of the stream is considered possible in this terrain with minor impacts. The alluvium lowlands and channel and infilled channel terrains will likely experience more significant lateral spreading during liquefaction. Our initial assessment indicated these areas may not be suitable for future development. See section 4.7.4 for uncertainties to be confirmed at the time of subdivision.

4.7.3 Cyclic softening

A cyclic softening assessment has been undertaken using Idriss and Boulanger (2007)[19]. The results indicate that the firm to stiff layer encountered in CPTC01 between 2.0 m and 2.4 m depth would need to be less than 15 kPa to trigger cyclic softening at the design shaking levels. The shear strength measurement indicates greater than 20 kPa, inferred from CPT results and over 50 kPa from in situ

[18] MBIE (2021). Earthquake geotechnical engineering practice, Module 3: Identification, assessment and mitigation of liquefaction hazards-Version 1, November 2021.
[19] Boulanger and Idriss (2007), Evaluation of Cyclic Softening in Silts and Clays

shear vane tests. Therefore, the cyclic softening risk is considered low for this site in the terrain of the undulating hills.

The results from CPTC04A indicate that the firm layer encountered between 3.4 m and 4.6 m bgl may be susceptible to a short-term strength reduction. Therefore, the cyclic softening risk is considered possible for the alluvial lowlands and channels and the infilled channels.

4.7.4 Site C liquefaction vulnerability

The preliminary liquefaction assessment indicated that the site falls into the Liquefaction Damage is Possible category. Whilst CPTC06 undertaken in the undulating hills terrain does suggest a low vulnerability, there are insufficient investigations to rule it out. The quantitative assessment did not highlight any discernible differences in expected performance between the geomorphic units and the information collected isn't of sufficient detail to determine a more precise category. The uncertainties in these assessments are explained below and should be confirmed at the time of subdivision consent:

- **Investigation density:** Following the MBIE Planning and Engineering Guidance document, there is sufficient density of investigations to determine the liquefaction vulnerability of the site, however, following MBIE module 2, further investigations would be required at the subdivision stage.
- **Susceptibility of upper soils:** The soils in the upper 2-3 m have been judged to be non-susceptible on account of the predicted plasticity from CPT data. Soils descriptions of this material are low to medium plasticity. In order to remove this uncertainty, further testing such as Plasticity Index testing is recommended to confirm the CPT findings.
- **Ground water depths**: Confirmation of groundwater depths including consideration of seasonal variation, should be provided with subdivision consent applications.
- **Delineation of historical filling**. The extent of fill should be investigated at subdivision stage.
- **Age of soils**: It has been shown that liquefaction resistance increased in older soils. In order to assess this specialist in-situ testing and subsequent analysis will be required. A cost benefit analysis of undertaking this work should be completed at the subdivision stage to determine if aging effects are likely to reduce the liquefaction vulnerability of these soils.

Table 4.5 has been provided below to comply with the data requirements of the MBIE planning and engineering guidance for potentially liquefaction-prone land.

Table 4.5: Liquefaction summary information table

LIQUEFACTION ASSESSMENT SUMMARY	
This liquefaction assessment has been undertaken in general accordance with the guidance document 'Assessment of Liquefaction-Induced Ground Damage to Inform Planning Processes' published by the Ministry of Business, Innovation and Employment in 2017. https://www.building.govt.nz/building-code-compliance/geotechnical-education	
Client	Hauraki District Council
Assessment undertaken by	Tonkin + Taylor Ltd
Report date	10/10/23
Extent of the study area	See Figure 1.2 Site C Location Plan
Intended RMA planning and consenting purposes	Plan Change
Other intended purposes	Subdivision, subject to further assessment regarding investigation density, plasticity of upper soils and seasonal variation in groundwater
Level of detail	Level B
Notes regarding base information	-
Other notes	-

4.8 Site specific liquefaction assessment – Site D

T+T in-house software was used to evaluate the liquefaction susceptibility of site D using the trigger methods recommended by Boulanger & Idriss (2014)[20].

No sensitivity assessment was undertaken for Site D because the groundwater table adopted in the assessment was considered to be conservative.

Results of our assessment of Site D are presented in Appendix D. The liquefaction susceptibility for all sites is discussed in the following sections.

4.8.1 Liquefaction susceptibility

Negligible liquefaction is assessed to occur at SLS level of shaking for site D.

A summary of the crust thickness, LSN, and expected settlement under IL2 and IL3 ULS levels of shaking for site D is presented on Table 4.6 below.

[20] Boulanger, R. W., & Idriss, I. M. (2014). *CPT and SPT based liquefaction triggering procedures.* Centre for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA. Report No. UCD/CGM-14/01.

Table 4.6: ULS liquefaction assessment results summary – Site D

CPT ID	IL2 ULS – 1 in 500			IL3 ULS – 1 in 2500		
	Crust thickness, CT (m)	Liquefaction Severity Number, LSN[2]	Free-field Surface Settlement[1], Sv1D (mm)	Crust thickness, CT (m)	Liquefaction Severity Number, LSN[2]	Free-field Surface Settlement[1], Sv1D (mm)
CPTD 01	5.2	2	20	4.8	7	60
CPTD 02	4.1	8	60	4.1	10	90

Notes:

1 Rounded to the nearest 5 mm due to inherent uncertainties associated with the calculation methods.

2 LSN values typically represent the following consequences (From MBIE Module 3[21] Table 5-1):

- L0, Insignificant (LSN<10, LPI=0): No significant excess pore water pressures (no liquefaction).
- L1, Mild (LSN 5-15, LPI=0): Limited excess pore water pressures; negligible deformation of the ground and small settlements.
- L2, Moderate (LSN 10-25, LPI<5): Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.

Key findings of the liquefaction assessment:

- Liquefaction assessment indicates under design ULS case LSN values are typically less than ten, and free field settlement estimates generally are between 0 and 90 mm.
- The soils encountered during the investigation are considered mildly to moderately susceptible to liquefaction under the design ULS events.

It is noted that the liquefaction assessment is based on simple Boulanger & Idriss (2014) triggering assessment and is not entirely consistent with the expectations of the geomorphic assessment and age of the soils which may suggest a low vulnerability. There are several potential reasons for this which are discussed as uncertainties in section 4.8.4 below.

4.8.2 Lateral spreading

Given that the soils on site are considered slightly to mildly susceptible to liquefaction, lateral spreading to the free faces is considered possible in this terrain. Whilst not part of the scope of this assessment, it is likely that the existing slopes will be subject to a setback to maintain slope stability in the static case. A detailed assessment of slope stability should be undertaken where appropriate setbacks can be determined, including consideration of lateral spreading. See section 4.8.4 uncertainties to be confirmed at the time of subdivision.

4.8.3 Cyclic softening

A cyclic softening assessment has been undertaken using Idriss and Boulanger (2007)[22]. The results indicate that materials encountered in CPTD02 are reasonably stiff and have higher shear strength values to trigger cyclic softening at the design shaking levels.

Therefore, the cyclic softening risk is considered low for this site.

[21] MBIE (2021). Earthquake geotechnical engineering practice, Module 3: Identification, assessment and mitigation of liquefaction hazards-Version 1, November 2021.
[22] Boulanger and Idriss (2007), Evaluation of Cyclic Softening in Silts and Clays

4.8.4 Site D liquefaction Vulnerability

The preliminary liquefaction assessment indicated that the site falls into a Liquefaction Damage is Possible liquefaction vulnerability category. Typically, it would be reasonable to expect hills and ranges geomorphic terrains to not be susceptible to liquefaction. There are a few uncertainties that may cause this difference in the expected and the calculated assessment. These are listed below and should be confirmed at the time of subdivision consent:

- **Investigation density:** Following the MBIE Planning and Engineering Guidance document, there is a sufficient density of investigations to determine the liquefaction vulnerability of the site; however, following MBIE module 2, further investigations would be required at the subdivision stage.
- **Ground water depths**: In our assessment, a conservative groundwater table was adopted based on the available data. Confirmation of groundwater depths, including consideration of seasonal variation, should be provided with subdivision consent applications.
- **Susceptibility of soils**: Confirmation of the soil plasticity by additional testing, such as plasticity index testing, should be conducted at the subdivision consent stage to understand the susceptibility of the soils better.
- **Age of soils**: It has been shown that liquefaction resistance increased in older soils. In order to assess this specialist in-situ testing and subsequent analysis will be required. A cost benefit analysis of undertaking this work should be completed at the subdivision stage to determine if aging effects are likely to reduce the liquefaction vulnerability of these soils.
- **Lateral spreading impacts:** Slope stability assessment is out of the scope of the current assessment. However, a slope stability assessment is required at the subdivision stage to confirm the setback from the edge of the slopes. This may also mitigate the potential lateral spreading impacts.

Table 4.7 has been provided below to comply with the data requirements of the MBIE planning and engineering guidance for potentially liquefaction-prone land.

Table 4.7: Liquefaction summary information table

LIQUEFACTION ASSESSMENT SUMMARY	
This liquefaction assessment has been undertaken in general accordance with the guidance document 'Assessment of Liquefaction-Induced Ground Damage to Inform Planning Processes' published by the Ministry of Business, Innovation and Employment in 2017. https://www.building.govt.nz/building-code-compliance/geotechnical-education	
Client	Hauraki District Council
Assessment undertaken by	Tonkin + Taylor Ltd
Report date	14/08/23
Extent of the study area	See Figure 1.8 Site D Location Plan
Intended RMA planning and consenting purposes	Plan Change
Other intended purposes	Subdivision, subject to further assessment regarding investigation density, plasticity of upper soils and seasonal variation in groundwater
Level of detail	Level B
Notes regarding base information	-
Other notes	-

4.9 2022 National Seismic Hazard Model (NSHM) update

In October 2022, GNS Science released the revised National Seismic Hazard Model (NSHM)[23]. This represents the latest scientific knowledge of earthquake hazard in New Zealand and is an important factor for understanding and managing earthquake risk in the built environment.

While the NSHM will inform future design standards, it does not provide information that can be directly applied in design applications. Consequently, the current minimum compliance pathway within the Building Code has not changed[24]. However, important updates to Building Code compliance documents that will be informed by the NSHM are expected to be released between 2023 and 2025.

We have undertaken an initial appraisal of the implications of the 2022 NSHM for geotechnical design for this project. It is indicated that the PGA calculated using the new NSHM method will likely be less than the current design guidelines (MBIE Module 1). We suggest evaluating the potential effects if the 2022 NSHM method is used in the detailed design stage.

The assessment included in this report outlines the seismic shaking hazard for the site as determined by the current minimum compliance pathway within the Building Code, and it is considered appropriate at the time of writing this report. However, through the plan change process, the new NSHM model may be available for direct use. We recommend that HDC liaise closely with their geotechnical and structural professionals to understand the potential impacts of uncertainty and how this can be managed for the site.

---

23 https://nshm.gns.cri.nz/

24 Current relevant compliance documents to meet *Clause B1: Structure* of the Building Code are as shown in Verification Method B1/VM1. For structural seismic design this is *NZS 1170.5:2004 – Structural Design Actions Part 5: Earthquake Actions – New Zealand.* For geotechnical design, although not directly referenced in B1/VM1, the Section 175 MBIE/NZGS guidance document *Earthquake Geotechnical Engineering Practice: Module 1 (November 2021)* is to be continued to be used for seismic design loadings.

5 Other geotechnical hazards

Assessment of slope stability, bearing capacity, erosion, expansive soils is outside the scope of this assessment. Aside from liquefaction, we have also considered the potential for static settlement risks related to compressible soils for each site and also high-level earthworks considerations. These are explained in the following sections and summarised in Table 5.1 to Table 5.3 at each potential site in relation to the proposed development.

5.1 Static Settlement

Based on the geomorphic study, there are five geomorphic terrains across the three sites (results were presented in Table 2.1). These have been qualitatively assessed on their potential to pose issues with static settlement due to the presence of soft and/or compressible soils. The assessment indicates the following:

- The undulating terrain and the hills and ranges terrain are not susceptible to static settlement for lightweight IL2 & IL3 structures except the area near-surface peat is present. Heavier structures or structures with longer design life will likely increase the risk of differential settlement. If this kind of structure were proposed with these two terrains, the risk would need to be carefully assessed at the design stage.
- The alluvium lowlands terrain and alluvium channel terrain are likely susceptible to static settlement risk, especially in the areas where peat was encountered during the investigation. Any development located within these two terrains needs to be assessed specifically at the project's planning stage.
- The infilled channel terrain is likely susceptible to static settlement risk. The extent and depth of the non-engineered fill will need to be confirmed. Any development located within these areas need to be assessed specifically at the project's planning stage.

5.2 Earthworks and retaining structures

Bulk earthworks are likely required to form building platforms. At this early stage, no bulk earthworks plans are available. However, some general considerations for the earthworks are provided below.

- Topsoil and backfill material should be stripped from working areas and stockpiled for later use. A progressive earthworks operation is expected to limit open working areas for erosion control (to be designed by others). At sites C and D, fill material was encountered during the investigation to at least 2.5 m below the surface.
- Cut surfaces should be stabilised following trimming to design levels to avoid rutting of the subgrade and scouring in heavy rainfall.
- Any cut proposed on site may reduce the thickness of the surficial non-liquefiable crust, increasing the liquefaction risks.
- We suggest the unretained cut face batter should be no steeper than 1V:2H. The unretained batter gradient can be refined as part of the detailed design phase.
- The site won silts and clays are likely to be sensitive. If site won material will be used as backfill material, lab tests are required to confirm the suitability of using the site won materials.

Table 5.1: Geotechnical issue comparison for different terrain - Site A

Issue	Undulating hills	Alluvium channel
Liquefaction vulnerability	**Low to Medium liquefaction vulnerability** This terrain is more likely to fall within the low category, this should be confirmed at the Subdivision stage.	**Low to Medium liquefaction vulnerability** Any loose, saturated sands that are susceptible to liquefaction are likely to be removed during the development of the site, alternatively, this feature may be retained as part of the stormwater disposal and may not be developed.
Compressible clay like soils	The static settlement risk is considered to be low for this terrain. The site-specific static settlement assessment indicated at the areas near the current testing locations, the anticipated settlement and different settlements for the lightweight residential dwelling are likely to be within the building code requirement. **Low risk.**	The geomorphic study indicates that this terrain is likely to comprise soils that are susceptible to settlement. **Moderate risk.**
Earthworks and retaining structures	No bulk earthworks plans are available; and the site won material may be suitable for future earthworks subject to further lab testing. **Low risk.**	

Table 5.2: Geotechnical issue comparison for different terrain – Site C

Issue	Undulating hills	Alluvium lowlands	Alluvium channel	Infilled Channels
Liquefaction vulnerability	**Liquefaction Damage is Possible** This terrain is more likely to fall within the low category, this should be confirmed at the Subdivision stage (see key uncertainties in section 4.8.4).	**Liquefaction Damage is Possible** This terrain is more likely to fall within the medium category, this should be confirmed at the Subdivision stage. Alternatively, this area may be retained as part of the stormwater treatment and may not be developed.	**Liquefaction Damage is Possible** This terrain is more likely to fall within the medium category. Any loose, saturated sands that are susceptible to liquefaction are likely to require removal under the proposed development and replaced with shallow ground improvements during the development of the site, alternatively, this feature may be retained as part of the stormwater disposal and may not be developed.	**Liquefaction Damage is Possible** The extent and nature of this terrain and the soils underlying it are unknown. It is likely that they will fall within the medium vulnerability category. Any loose, saturated sands that are susceptible to liquefaction are likely to require removal and be replaced with shallow ground improvements during the development of the site.
Compressible clay like soils	The static settlement risk is considered to be low for this terrain. The site-specific static settlement assessment indicated at the areas near the current testing locations, the anticipated settlement and different settlements for the lightweight residential dwelling are likely to be within the building code requirement. **Low risk.**	Compressible materials were encountered within these two terrains during the investigation (CHA02 and CHA03). And the geomorphic study indicates that these two terrains are likely to comprise soils that are susceptible to settlement. **Moderate risk.**		The extent and nature of this terrain and the soils underlying it are unknown. It is likely that they fill has been placed in an uncontrolled fashion and may contain unsuitable materials. **High Risk.**
Earthworks and retaining structures	No bulk earthworks plans are available; however, it is considered this risk is low for this terrain. The site won material may be suitable for future earthworks subject to further lab testing. **Low risk.**	Our investigation indicated earthworks had been carried out on-site in the past; fill material was encountered during the investigation to at least 1.8 m below the surface. Additional investigation is required for future developments to confirm the extent and depth of the fill material. No bulk earthworks plans are available; the site-won material may be suitable for future earthworks subject to further lab testing. **Low to moderate risk.**		

Table 5.3: Geotechnical issue comparison for different terrain – Site D

Issue	Hills and ranges
Liquefaction vulnerability	**Liquefaction Damage is Possible** The assessments indicate these soils are susceptible to liquefaction, however, there are significant uncertainties in this assessment that should be assessed at Subdivision stage. See section 4.8.4 for list of uncertainties. Deep machine drill holes and installing dual piezometers to confirm the actual the groundwater table are recommended.
Compressible clay like soils	The static settlement risk is considered to be low for majority of the site except the area where compressible materials were encountered during the investigation (DHA01). **Low to moderate risk.**
Earthworks and retaining structures	Our investigation indicated earthworks may had been carried out on-site in the past; fill material may encounter below the surface. Additional investigation is required for future developments to confirm the extent and depth of the fill material. No bulk earthworks plans are available; the site-won material may be suitable for future earthworks subject to further lab testing. **Low to moderate risk.**

6 Further work

We understand that this report is prepared to assist a plan change application. This study is intended to provide a high-level liquefaction and static settlement assessment to support plan change with rezoning purposes only.

It is recommended that a site-specific geotechnical assessment for new developments is undertaken as part of the design of the natural hazard risks should be undertaken at the subdivision stage in order to satisfy the requirements of s106 of the RMA. The key uncertainties listed in the Liquefaction Vulnerability Assessment should be addressed at subdivision stage.

7 Applicability

This report has been prepared for the exclusive use of our client Hauraki District Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Reviewed by:
....	....
Jia Hong Senior Geotechnical Engineer	John Brzeski Senior Engineering Geologist

Authorised for Tonkin & Taylor Ltd by:

....

Craig Davanna
Project Director

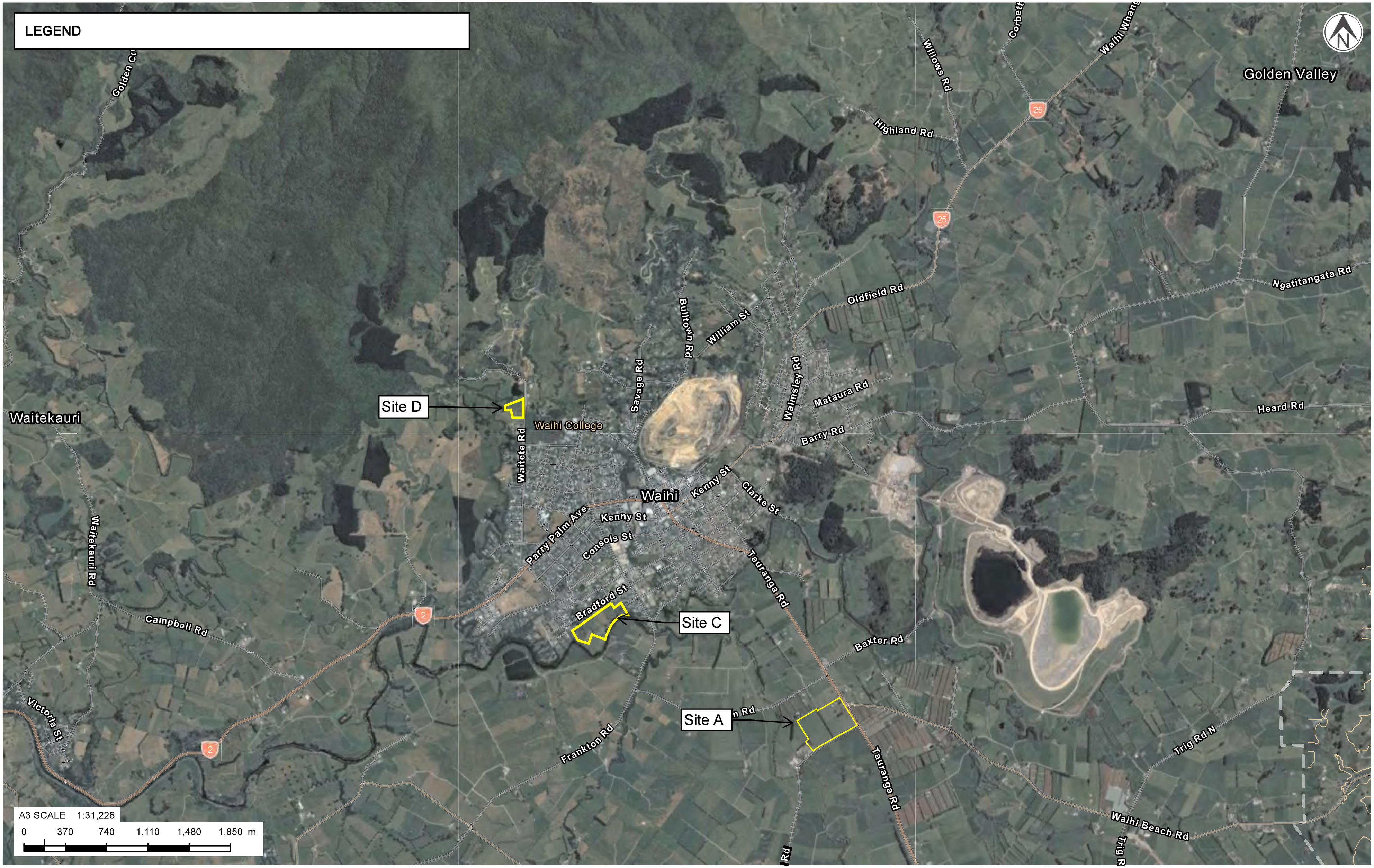
17-Oct-23
\\ttgroup.local\corporate\hamilton\projects\1090294\issueddocuments\231017.geotechnical assessment report site a, c & d_rev 1.1.docx

Appendix A Figures

- **Figure 1 – Overall Site Plan**
- **Figure A-1 – Site A Plan**
- **Figure C-1 – Site C Plan**
- **Figure D-1 – Site D Plan**

COPYRIGHT ON THIS FIGURE IS RESERVED   DO NOT SCALE FROM THIS FIGURE.

Source: Tonkin + Taylor Map Viewer (NZ)   Date: 25/07/2023 12:23 PM

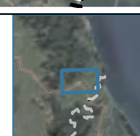


Tonkin+Taylor

Exceptional thinking together   www.tonkintaylor.co.nz

NOTES:

CRS: NZGD 2000 New Zealand Transverse Mercator   Credits: Earthstar Geographics, LINZ, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS, Esri, HERE, Garmin, FAO, NOAA, USGS



LOCATION PLAN

PROJECT No.	1090294	
DESIGNED	JIHO	Oct.23
DRAWN	JJBR	Oct.23
CHECKED	Not for construction	
APPROVED		DATE

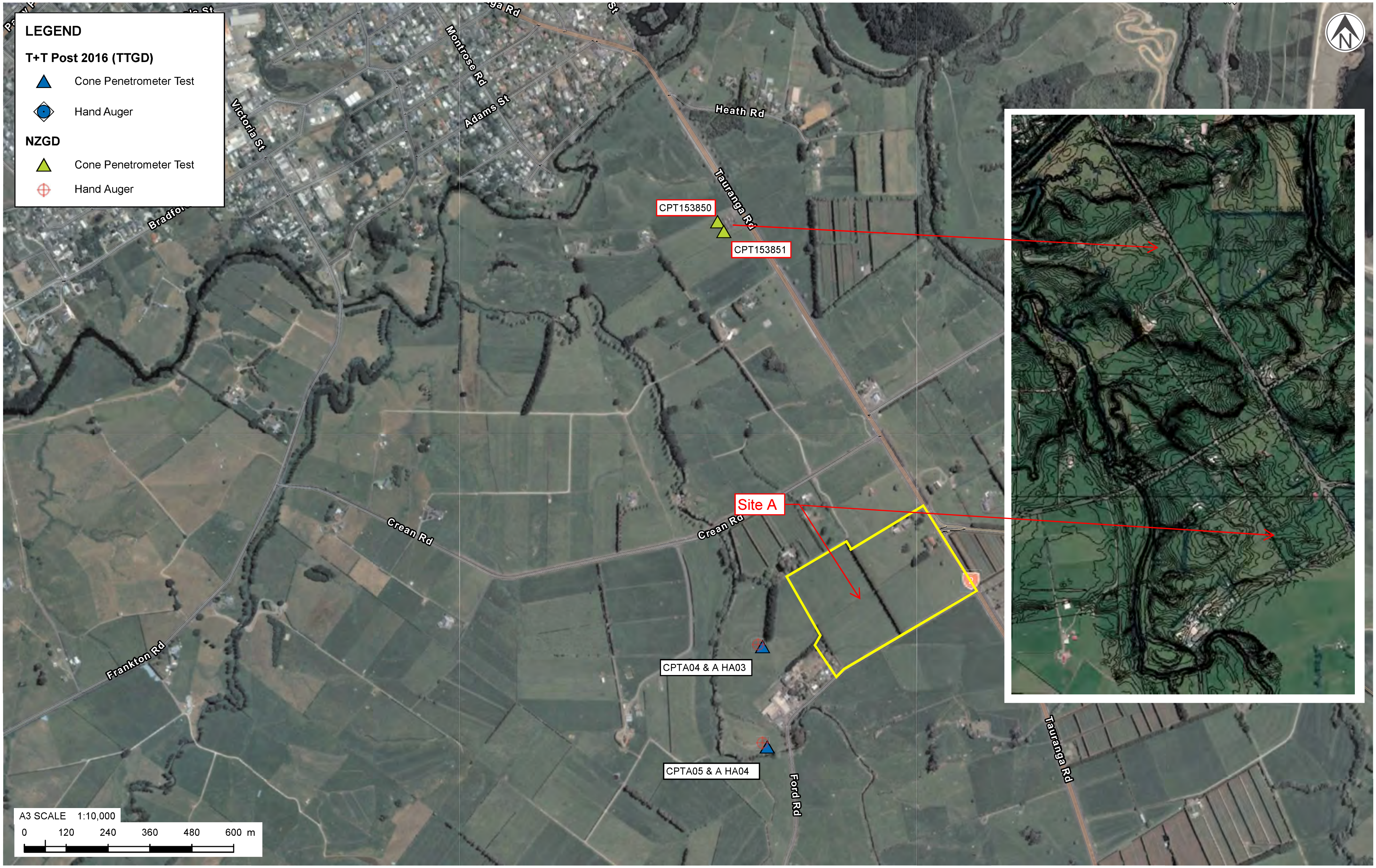
CLIENT HAURAKI DISTRICT COUNCIL

PROJECT WAIHI LIQUEFACTION STUDY

TITLE SITE PLAN - OVERVIEW

SCALE (A3) 1:31,226   FIG No. FIGURE 1.   REV 0

Source: Tonkin + Taylor Map Viewer (NZ) Date: 6/08/2023 11:16 PM



Tonkin+Taylor
Exceptional thinking together www.tonkintaylor.co.nz

NOTES:

CRS: NZGD 2000 New Zealand Transverse Mercator Credits: Earthstar Geographics, Esri Community Maps Contributors, LINZ, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS, Stats NZ, Esri, HERE, Garmin, Foursquare, FAO, METI/NASA, USGS, Maxar

LOCATION PLAN

PROJECT No.	1090294	
DESIGNED	JIHO	Oct.23
DRAWN	-WEB-	AUG.23
CHECKED	Not for construction	
APPROVED		DATE

CLIENT HAURAKI DISTRICT COUNCIL
PROJECT WRC LIQUEFACTION WORK - REZONING LAND
TITLE Site A - nearby CPTs - NZGD
SCALE (A3) 1:10,000 FIG No. FIGURE A-1 REV 0

Source: Tonkin + Taylor Map Viewer (NZ)   Date: 1/06/2023 3:30 PM

Tonkin+Taylor

Exceptional thinking together   www.tonkintaylor.co.nz

NOTES:
Site C

CRS: NZGD 2000 New Zealand Transverse Mercator   Credits: Earthstar Geographics, Esri Community Maps Contributors, LINZ, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS, Maxar, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS

Waihi
LOCATION PLAN

PROJECT No.	1090294.000	
DESIGNED	JIHO	Oct.23
DRAWN	-WEB-	JUN.23
CHECKED	JJBR	Oct.23
APPROVED		DATE

CLIENT	HAURAKI DISTRICT COUNCIL
PROJECT	WAIHI REZONING
TITLE	Waihi Liquefaction Study 1090294: Locations of GI - Site C
SCALE (A3)	1:2,500
FIG No.	C -1
REV	0

Source: Tonkin + Taylor Map Viewer (NZ) Date: 1/06/2023 3:36 PM

Tonkin+Taylor

Exceptional thinking together www.tonkintaylor.co.nz

NOTES:
Site D

CRS: NZGD 2000 New Zealand Transverse Mercator Credits: LINZ, Waikato Regional, Earthstar Geographics, Esri Community Maps Contributors, LINZ, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS, LINZ, Stats NZ, Esri, HERE, Garmin, Foursquare, METI/NASA, USGS

LOCATION PLAN

PROJECT No.	1090294.000	
DESIGNED	SCRO	JUN.23
DRAWN	-WEB-	JUN.23
CHECKED	JIHO	JUN.23
APPROVED	DATE	

CLIENT HAURAKI DISTRICT COUNCIL

PROJECT WAIHI REZONING

TITLE Waihi Liquefaction Study 1090294: Locations of GI - Site D

SCALE (A3) 1:500 FIG No. D - 1 REV 0