REPORT

Tonkin+Taylor

Liquefaction Desktop Study

Ngahinapouri Village Concept Plan

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1 Introduction

The Waipa district has been identified as a high growth area in the National Policy Statement on Urban Development Capacity. The town of Ngahinapouri is forecast to grow by 380-650 people (+190%-225% growth) by 2050. To provide for this growth, structure plans for the growth cells located in Ngahinapouri are required, as identified in the Waipa2050 Growth Strategy (2017) and Waipa District Council (WDC) 2018 – 2028 Long Term Plan.

The extents of the 2035 growth cells shown in Figure 1 of Appendix A have since changed so that growth cell N1 covers the N1 and N2 area shown in this figure and N2 covers the area labelled as N3.

The growth cells have a total area of approximately 102 ha in size, to the west of SH39 on both the northern and southern sides of Reid Road.

Tonkin & Taylor Ltd (T+T) have been requested by Boffa Miskell Ltd to investigate and provide a Level A desktop liquefaction assessment for the growth cells. These assessments will support the Structure Plans for each cell and Plan Changes to the District Plan.

1.1 Scope of work

The scope of works comprises a desktop assessment of liquefaction vulnerability of the growth cells in general accordance with a Level A assessment as described in Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE/MfE/EQC, 2017). A Level A assessment is further described in Section 2.4 and considers basic information about geology, groundwater and seismic hazard to assess the potential for liquefaction to occur.

The scope of this report can be summarised as:

- Collation and review of available data that is relevant to this study including:
  - Geological and geomorphic maps.
  - Ground surface elevation levels for the extent of the study area.
  - Geotechnical investigations and laboratory tests that are currently available on the New Zealand Geotechnical Database (NZGD).
  - Groundwater level information for the study extent.
- Assess the liquefaction vulnerability
- Provide potential risk treatment options

It should be noted that the provision of general geotechnical advice relating to the structure plans is outside the scope of the original Request for Proposal.

2 Liquefaction vulnerability assessment

2.1 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 contact stress (known as "effective stress") to decrease. The soil softens and loses strength as this effective stress is reduced. This process is known as liquefaction.

The elevation in pore water pressure can result in the flow of water in the liquefied soil. This water can collect under a lower permeability soil layer and if this capping layer cracks, can rush to the surface bringing sediment with it. This process causes ground failure and with the removal of water and soil, a reduction in volume and hence subsidence of the ground surface.

The surface manifestation of the liquefaction process is the water, sand and silt ejecta that can be seen flowing up to two hours following an earthquake. The path for the ejecta can be a geological discontinuity or a man-made penetration, such as a fence post, which extends down to the liquefying layer to provide a preferential path for the pressurised water. The sand often forms a cone around the ejecta hole. With the dissipation of the excess pore-water pressure, the liquefied soil regains its pre-earthquake strength and stiffness.

The surface expression of liquefaction, water and sand depends on a number of characteristics of the soil and the geological profile. If there is a thick crust of non-liquefiable soil such as a clay, or sand that is too dense to liquefy during the particular level of shaking of the earthquake, then water fountains and sand ejecta may not be seen on the surface. The amount of ground surface subsidence is generally dependent on the density of the sand layers as well as how close the liquefying layers are to the surface. Ground surface subsidence increases with increasing looseness in the soil packing. The ground rarely subsides uniformly resulting in differential settlement of buildings and foundations. Figure 2.1 summarises the process of liquefaction with a schematic representation.

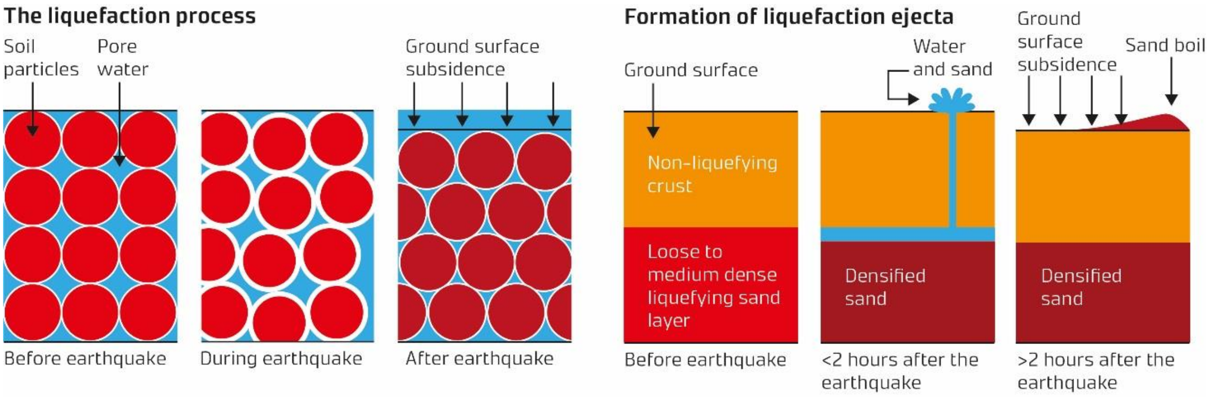

Figure 2.1: Schematic representation of the process of liquefaction and the manifestation of liquefaction ejecta.

2.2 Liquefaction susceptibility and triggering

The conditions often susceptible to liquefaction occur in geologically young sedimentary deposits such as those shown in Figure 2.2. In general terms, loose sands, some silts and in some cases gravels are most susceptible. While clays generally do not liquefy, they may still soften during an earthquake. Soil types which are susceptible to liquefaction include:

- Sands and low plasticity/non-plastic silts. (Bray & et al, 2014).
- Fine grained low to non-plastic soils with a high moisture content. (Bray & Sancio, 2006), (Boulanger & Idriss, 2006).
- Young, typically Holocene-aged (≤12,000 years old) deposits.
- Gravels can liquefy if they have a low permeability or are confined by less permeable layers.

The groundwater level in the soil is an important factor and soils with the groundwater at or near the surface are more susceptible.

Figure 2.2: Some landforms commonly susceptible to liquefaction, (MBIE/MfE/EQC, 2017).

2.3 Liquefaction consequence

Figure 2.3 presents the characteristics of liquefaction related land damage, and a summary of the likely consequences of liquefaction related damage for each category of land damage. This figure has been reproduced from (MBIE/MfE/EQC, 2017). Appendix A of the MBIE Guidance includes photos of liquefaction-induced land damage for each of these categories. These provide a useful reference for understanding the magnitude of land damage that can be expected within each category.

The main potential consequences of liquefaction are discussed in MBIE Planning and Engineering Guidance for Potentially Liquefaction-Prone Land. Table 2.1 from these guidelines is reproduced in Table 2.1 of this report.

DEGREE OF LIQUEFACTION-INDUCED GROUND DAMAGE (example photographs)	TYPICAL CONSEQUENCES AT THE GROUND SURFACE These are examples of the type of damage that would be expected, they are not intended to be criteria for calculation
None to Minor 	– ***None to Minor*** no signs of ejected liquefied material at the ground surface[1]. – No more than minor differential settlement of the ground surface (eg undulations less than 25 mm in height). – No apparent lateral spreading ground movement (eg only hairline ground cracks). – Liquefaction causes no or only cosmetic damage to buildings and infrastructure (but damage may still occur due to other earthquake effects).
Minor to Moderate 	– ***Minor to Moderate*** quantities of ejected liquefied material at the ground surface (eg less than 25 percent of a typical residential site covered[2]); and/or – Moderate differential settlement of the ground surface (eg undulations 25–100 mm in height). – No significant lateral spreading ground movement (eg ground cracks less than 50 mm wide may be present, but pattern of cracking suggests the cause is primarily ground oscillation or settlement rather than lateral spreading). – Liquefaction causes moderate but typically repairable damage to buildings and infrastructure. Damage may be substantially less where liquefaction was addressed during design (eg enhanced foundations).
Moderate to Severe 	– Large quantities of ejected liquefied material at the ground surface (eg more than 25 percent of a typical residential site covered[2]); and/or – ***Moderate to Severe*** differential settlement of the ground surface (eg undulations more than 100 mm in height); and/or – Significant lateral spreading ground movement (eg ground cracks greater than 50 mm wide, with pattern of cracking suggesting direction of movement downslope or towards a free-face). – Liquefaction causes substantial damage and disruption to buildings and infrastructure, and repair may be difficult or uneconomic in some cases. Damage may be substantially less, and more likely to be repairable, where liquefaction was addressed during design (eg enhanced foundations and robust infrastructure detailing).

Notes:

1 An absence of ejecta at the ground surface does not necessarily mean that liquefaction has not occurred. Liquefaction may still occur at depth, potentially causing ground settlement.

2 The coverage of the site with ejected liquefied material does not in itself represent ground damage in an engineering sense, however there is a strong correlation between the volume of ejecta and the severity of differential ground settlement and foundation/infrastructure damage.

Figure 2.3: Degrees of liquefaction-induced land damage (MBIE/MfE/EQC, 2017)

Table 2.1: Consequences of liquefaction, as published in Planning and engineering guidance for potentially liquefaction-prone land

Land	• Sand boils, where pressurised liquefied material is ejected to the surface (ejecta). • Ground settlement and undulation, due to consolidation and ejection of liquefied soil. • Ground cracking from lateral spreading, where the ground moves downslope or towards an unsupported face (e.g. a river channel or terrace edge).
Environment	• Discharge of sediment into waterways, impacting water quality and habitat. • Fine airborne dust from dried ejecta, impacting air quality. • Potential contamination issues from ejected soil. • Potential alteration of groundwater flow paths and formation of new springs.
Buildings	• Distortion of the structure due to differential settlement of the underlying ground, impacting the amenity and weathertightness of the building. • Loss of foundation-bearing capacity, resulting in settlement of the structure. In some cases this can result in tilting or overturning of multi-level buildings. • Stretch of the foundation due to lateral spreading, pulling the structure apart. In some cases this can result in collapse or near-collapse of buildings. • Damage to piles due to lateral ground movements, and settlement of piles due to down drag from ground settlement • Damage to service connections due to ground and building deformations
Infrastructure	• Damage to road, rail and port infrastructure (settlement, cracking, sinkholes, ejecta). • Damage to underground services due to ground deformation (e.g. ‘three waters’, power and gas networks). • Ongoing issues with sediment blocking pipes and chambers. • Uplift of buoyant buried structures (e.g. pipes, pump stations, manholes and tanks). • Damage to port facilities. • Sedimentation and ‘squeezing’ of waterway channels, reducing drainage capacity. • Deformation of embankments and bridge abutments (causing damage to bridge foundations and superstructure). • Settlement and cracking of flood stop banks, resulting in leakage and loss of freeboard. • Disruption of stormwater drainage and increased flooding due to ground settlement
Economic	• Lost productivity due to damage to commercial facilities, and disruption to the utilities, transport networks and other businesses that are relied upon. • Absence of staff who are displaced due to damage to their homes or unable to travel due to transport disruption. • Cost of repairing damage
Social	• Community disruption and displacement – initially due to damage to buildings and infrastructure, then the complex and lengthy process of repairing and rebuilding. • Potential ongoing health issues (e.g. respiratory and psychological health issues).

While the immediate effects of liquefaction relate primarily to land, building and infrastructure damage, liquefaction can also have a significant social, economic and environmental impact, refer to Section 2.4 of Planning and Engineering Guidance for Potentially Liquefaction-prone Land (MBIE/MfE/EQC, 2017).

2.4 Assessment methodology

This liquefaction vulnerability assessment has been undertaken general accordance with a Level A assessment as described in Planning and engineering guidance for potentially liquefaction-prone land (MBIE/MfE/EQC, 2017). In that document a Level A assessment is described as a *Basic Desktop*

Assessment which equates to an assessment of regional-scale information supported by a site walkover. For the purposes of this study, each growth cell has been classified in terms of its geomorphic zone. These zones are then assigned a liquefaction vulnerability classification as described below.

The methodology described in the Planning and engineering guidance for potentially liquefaction-prone land (MBIE/MfE/EQC, 2017) recommends categorisation of the liquefaction vulnerability of the land based on the performance criteria described in Figure 2.4 below.

<table>
<tr><td colspan="4">LIQUEFACTION CATEGORY IS UNDETERMINED
A liquefaction vulnerability category has not been assigned at this stage, either because a liquefaction assessment has not been undertaken for this area, or there is not enough information to determine the appropriate category with the required level of confidence.</td></tr>
<tr><td colspan="2">LIQUEFACTION DAMAGE IS UNLIKELY
There is a probability of more than 85 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking.
At this stage there is not enough information to distinguish between Very Low and Low. More detailed assessment would be required to assign a more specific liquefaction category.</td><td colspan="2">LIQUEFACTION DAMAGE IS POSSIBLE
There is a probability of more than 15 percent that liquefaction-induced ground damage will be Minor to Moderate (or more) for 500-year shaking.
At this stage there is not enough information to distinguish between Medium and High. More detailed assessment would be required to assign a more specific liquefaction category.</td></tr>
<tr><td>Very Low Liquefaction Vulnerability
There is a probability of more than 99 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking.</td><td>Low Liquefaction Vulnerability
There is a probability of more than 85 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking.</td><td>Medium Liquefaction Vulnerability
There is a probability of more than 50 percent that liquefaction-induced ground damage will be:
Minor to Moderate (or less) for 500-year shaking; and
None to Minor for 100-year shaking.</td><td>High Liquefaction Vulnerability
There is a probability of more than 50 percent that liquefaction-induced ground damage will be:
Moderate to Severe for 500-year shaking; and/or
Minor to Moderate (or more) for 100-year shaking.</td></tr>
</table>

Figure 2.4: Performance criteria for determining the liquefaction vulnerability category - reproduced from Table 4.4 of MBIE/MfE/EQC (2017)

The performance criteria listed in Figure 2.4 relate the liquefaction vulnerability category to the expected liquefaction-induced land damage at a given ARI level of earthquake shaking. The assessment requires the assessor to consider the probability that a particular level of liquefaction-induced land damage will occur for a given level of shaking. In undertaking this assessment it is important to understand the following note attached to the table in the guidance document:

"The probabilities listed in this table are intended to provide a general indication of the level of confidence required to assign a particular category, rather than to be a specific numerical criteria for calculation. Conceptually, these probabilities relate to the total effect of all uncertainties in the assessment…"

That is, the guidance recommends the assessor consider the combined effect of all the uncertainties associated with the available information in the determination of the land damage category.

The general methodology applied to determine the liquefaction vulnerability category for the study area is as follows:

1 Evaluate the uncertainties associated with the mapping. This includes consideration of the resolution of mapping and the variability of soil conditions.

2 Evaluate the uncertainties associated with the groundwater level. Due to the limited amount of information about groundwater within the study area this is primarily dependent on field

experience and engineering judgement and is one of the most significant sources of uncertainty in this assessment.

3 Evaluate the uncertainties associated with the determination of the seismic hazard for the study extent. Whilst current scientific understanding suggests that the Hamilton and Waipa Basin area is expected to have a relatively low level of seismic hazard compared to other regions across New Zealand, there remains considerable uncertainty regarding the likelihood and intensity of earthquake shaking that could occur. This uncertainty is especially relevant where liquefaction-susceptible soils are present but estimated design shaking intensities (e.g. PGA for 100 year ARI design event) are unlikely to be strong enough to trigger liquefaction. This means that if earthquake shaking intensity is slightly greater than assumed for design (or if design PGA values increase in future due to improved understanding of the hazard), then a step-change worsening in performance could occur. For this reason, where liquefaction-susceptible soils are present it is generally not preferable to rely exclusively on low design PGA values to assign a liquefaction vulnerability category of *Liquefaction Damage Is Unlikely, Very Low* or *Low*.

4 Based on the consideration of all of these uncertainties, assign one of the liquefaction vulnerability categories defined in Figure 2.4 to the land within the project extent.

3 Ground conditions

3.1 Geology

Ngahinapouri is situated within the Hamilton basin (Figure 3.1), a graben that has been progressively infilled with a complex sequence of volcanogenic alluvium and various ignimbrites and tephra since c. 2 million years ago (McCraw, 2011). Geological mapping indicates that the study area is underlain by the Hinuera Formation.

The Hinuera Formation forms the extensive plains observed in Hamilton lowlands. This formation comprises interbedded coarse alluvium, pumice gravels, peat and silts deposited by braided river systems of the ancestral Waikato and Waipa Rivers. These rivers continued to deposit vast amounts of sediment into the Hamilton lowlands until climatic conditions changed c. 17,000 and the river systems entrenched into present day positions (Molloy, 1998).

Due to the nature of the depositional environment, the Hinuera Formation is highly variable both laterally and vertically. Loose sands and gravels are found in the higher energy environments and levees and finer grained sediments such as silt represent the low energy environments such as embayed channels and on the inside of river bends.

It has been noted that sand mining has occurred in the northwest of the site within N2. In the absence of compaction records, it is assumed that this area has been backfilled with non-engineered fill, this will need to be further investigated before the development of this site.

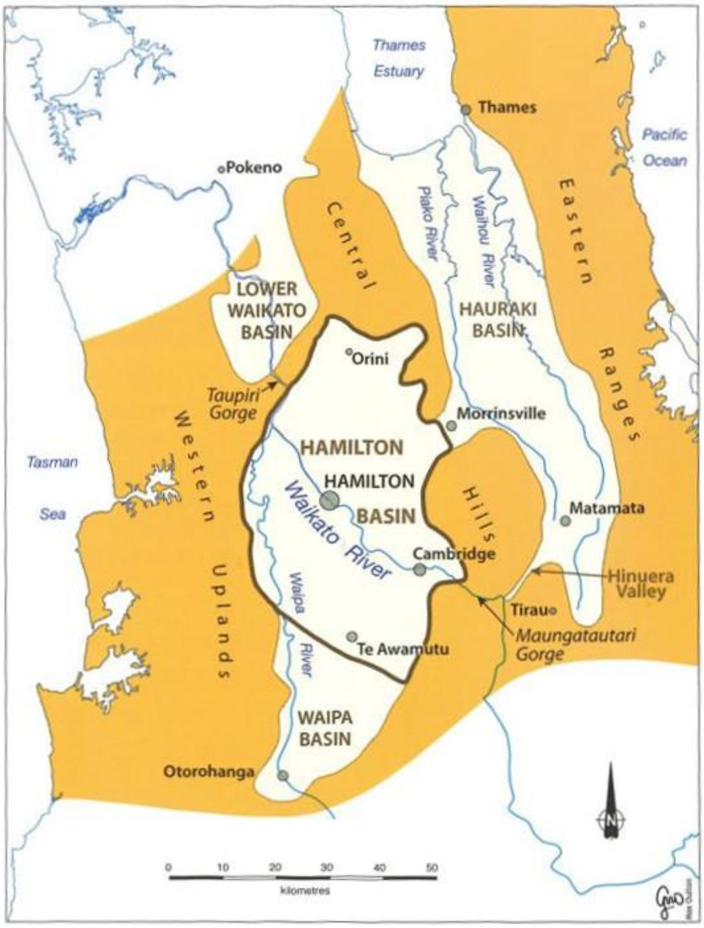

Figure 3.1: The Hamilton lowlands or Basin in the upper central North Island is bounded to the west and east by ranges and bisected by the Waikato River (McCraw, 2011).

3.2 Faulting

The GNS New Zealand Active Fault Database identifies the Kerepehi fault as the closest active fault to the site at approximately 45km to the east. Other faults affecting the Hamilton and Waipa Basin include the inferred non-active Waipa fault and the Taupiri fault to the north proposed by (Kirk, 1991).

3.3 Site geomorphology

Ngahinapouri is in a low-lying level area of the Hamilton Basin with no significant changes in geomorphology save for the Mangahia stream running along the northern boundary of the site. An area which marks the site of previous sand mining is present in the northwest corner of the site, due to the lack of records regarding the filling of this area, this has been marked accordingly on the Geomorphic Plan, and the remainder of the site is classed as Alluvial Plains. Figure 1 in Appendix A shows the layout of the geomorphic zones and they are further described in Table 3.1 below. The basis of the zones are the geological mapping (Edbrooke, 2005), a Digital Elevation Model (DEM) derived from LiDAR data, aerial photography and a site walkover undertaken in January 2019.

Table 3.1: Description of geomorphic zones adopted for the study areas

Geomorphic zone	Typical geology	Description
Alluvial plains	Piako Subgroup	Highly variable both vertically and laterally as the ancestral Waipa and Waikato Rivers deposited material eroded from the volcanic catchments of the central North Island. The deposits filled the low lying ground and channels and depressions within the eroded surface of the Walton Subgroup. The "Hinuera Surface" today consists of a series of low ridges, swales and flat plains sloping gently to the north. Soils comprise cross-bedded silts, sands, gravels with peat lenses also common. Sequences may exhibit a general fining upwards sequence, (McKay, Lowe, & Moon, 2017).
Previously mined area	Non-engineered fill	Likely to be variable in consistency and have inconsistent compaction. No records available for the backfill materials at this stage

3.4 Groundwater

In the absence of long term groundwater monitoring data we have undertaken a review of groundwater information for the Ngahinapouri. The Mangahia stream running along the northern boundary of the site will control the local groundwater levels, however, groundwater within the Alluvial Plains can be shallow groundwater when not controlled by localised drainage associated with river terraces, gullies and deep swales.

The New Zealand Geotechnical Database shows a number of investigations undertaken within the N1 and N2 growth cells. These investigations indicate that groundwater was encountered at 2.4 to 3.5 m bgl, with slightly shallow levels recorded further south of the Mangahia stream. This should be taken as an indication only and is subject to confirmation by monitoring. Seasonal fluctuations in the water table should also be considered when determining the groundwater level to be used in design.

4 Liquefaction assessment

4.1 Seismic site subsoil class

The seismic subsoil class in accordance with NZS 1170.5:2004 (Section 3.1.3) for the site is considered to be 'Class D – Deep Soil Sites'. This assumption is based on recent research by The University of Waikato (Jeong, 2019) which suggests that the majority of the Hamilton and Waipa Basin should be categorised as site Class D except on the basin margins.

4.2 Ground shaking hazard

The seismic hazard in terms of Peak Ground Acceleration (PGA) for the area has been assessed based on the NZTA Bridge Manual in accordance with the approach recommended in NZGS Module 1 (NZGS/MBIE, 2016).

For the purposes of this study, we have assumed that the growth cells will be used for residential development consisting of Importance Level 2 buildings with a 50 year design life. Consequently, the 25 and 500 year return periods correspond to Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design events in this case. It should be noted that if the liquefaction assessment is extended to the school then a larger return period may need to be considered if it is classified as an IL3 building.

Table 4.1 presents the return periods for earthquakes with various 'unweighted' PGAs with corresponding earthquake magnitudes.

Table 4.1: Ground seismic hazard

Event	Return period (years)	PGA (g)	Magnitude (M_{eff})
SLS	25	0.056	5.9
ULS	500	0.223	5.9

4.3 Results

A liquefaction category of "*Liquefaction Damage is Possible*" is assigned to the areas of Alluvial Plains. The previously mined area is assigned a "*High Liquefaction Vulnerability*" Classification. A plan showing these areas is presented in Appendix B, Figure 2. Liquefaction vulnerability for the site has been assessed by geological screening with qualitative calibration and using semi-quantitative screening criteria based on age, peak ground acceleration expected, depth to groundwater and experience in undertaking quantitative assessments in these geological materials.

The alluvial plains are highly variable in geology both laterally and vertically. Land damage of "None to Minor" through to "Moderate to Severe" are all possible within the alluvial plains, therefore it is important to have a good understanding of the underlying geology. The site may be underlain by a great thickness of liquefiable soils or may only have thin, intermittent layers of liquefiable soils interbedded with medium dense to dense gravels.

The current categorisation of *Liquefaction Damage Is Possible* for the Alluvial Plains does not preclude the later categorisation of these areas into *the Liquefaction Damage Is Unlikely* category (or *Low* or *Very Low* categories) if appropriate based on additional local investigation and analysis. If a sufficiently thick non-liquefiable crust is shown to be present on site, the effect of liquefaction can be minimised at the surface.

The infilled mine area is likely to contain non-engineered fill. These materials are highly susceptible to liquefaction, however, it is likely that this material will require removal due to the potential presence of ground contamination and/or other geotechnical considerations. The removal of this material will not remove the liquefaction hazard completely as the level of vulnerability will be determined by the underlying soil. The extent shown on the plan is indicative only, the full extent of mine full is unknown at this stage and should be determined through site investigation.

4.4 Lateral spreading vulnerability

Observations from previous earthquakes demonstrate that liquefaction-induced lateral spreading can cause significant damage to buildings, infrastructure and the environment. Therefore consideration of the potential for lateral spreading should be applied when undertaking a liquefaction vulnerability assessment.

When considering the potential for lateral spreading adjacent to a free-face, the Planning and engineering guidance for potentially liquefaction-prone land (MBIE/MfE/EQC, 2017) notes that *"It is less likely (but not impossible) for lateral spreading to occur if there is no liquefied soil within a depth of 2H of the ground surface (where H is the height of the free-face)."* Zhang, Robertson, & Brachman (2004) define H as the difference in height from the toe of the embankment (frequently the invert of a river or other water surface body) to the top of the embankment for which lateral spreading is being assessed.

However, with the information available for this study it is difficult to accurately define the free face height (H). This is primarily because it is difficult to confirm whether or not Digital Elevation Models (DEM) derived from LiDAR data are accurately estimating the elevation of the invert due to it frequently being obscured by water or vegetation.

The Planning and engineering guidance for potentially liquefaction-prone land (MBIE/MfE/EQC, 2017) recommends that particular attention should be given to land that is susceptible to liquefaction within 100 m of a free face less than 2 m high; or within 200 m of a free face greater than 2 m high. That is not to say that lateral spreading is likely to extend this far, however, the effects need to be considered to these extents.

Also, particular attention should be given to the potential for lateral spreading to occur on land within the Stream Gullies geomorphic zone. This is because of a combination of the land being categorised as *Liquefaction Damage Is Possible*, the potential for relatively shallow groundwater and there being a significant number of free faces associated with rivers and streams in this zone.

4.5 Potential risk treatment options

There are various potential options available to manage liquefaction-related risk, as summarised in Section 6 of MBIE (2017).

One potential solution is to avoid exposure to the hazard by not constructing within liquefaction-prone land. Further investigation will allow WDC to refine the liquefaction vulnerability areas and may allow uncategorised areas to be reclassified as low vulnerability.

Another potential solution is to reduce or mitigate liquefaction-related risk by reducing the likelihood of liquefaction occurring and/or reducing the consequences if liquefaction occurs. Potential foundation design and ground improvement options to mitigate the damaging effects of liquefaction are discussed in the series of guidance documents produced by MBIE for repairing and rebuilding houses affected by the Canterbury earthquakes (MBIE, 2012). Generally, the type of damage experienced may result in differential settlements, global settlements and ingress of liquefaction ejecta that could damage infrastructure and buildings. The risk of damage such as this is normally treated in one or a combination of the following ways:

- Undertake **ground improvement** so that a higher level of earthquake shaking is required to trigger liquefaction. In some cases it may be possible to change the fundamental behaviour of the ground (e.g. by physically removing or cementing susceptible soil) so that liquefaction will not occur even under the highest levels of earthquake shaking expected.
- Specify **robust foundation systems** that are able to tolerate liquefaction related land damage, such as thick reinforced foundations or stiff platforms. The importance level of the structure and the specific ground conditions at the site would inform the performance standard required for these foundation systems.
- Specify **readily repairable foundation systems** that are able to be reinstated relatively easily following liquefaction induced land damage.
- Specify the use of **lightweight building materials** for construction of buildings. Adopting lightweight cladding and roofing materials reduces the required bearing strength of the underlying soils and the severity of structural shaking imposed on the foundations. As such, lightweight building materials reduce the potential for liquefaction-induced foundation and building damage to occur.

There are various potential opportunities for Territorial Authorities to take an active role in managing liquefaction-related risk, while also facilitating development by simplifying site-specific ground investigation and foundation design requirements where appropriate. Possible examples include:

- Defining succinct geotechnical information requirements for resource and building consent applications, which focus on resolving the key uncertainties in the liquefaction assessment relevant for each geomorphic zone.

- Identifying standard foundation solutions which can be applied “off the shelf” once the liquefaction vulnerability category has been confirmed with sufficient certainty.
- Undertaking a widely-spaced grid of ground investigations and/or groundwater monitoring across the growth cells. This would provide greater certainty in the assessment of liquefaction vulnerability, and could allow some types of development to proceed relying only on the existing information without the need for site-specific investigations (where appropriate, and subject to a requirement for robust foundations).

4.6 Recommendations for further assessment

Further assessment of the liquefaction risk should be undertaken at subdivision stage in order to satisfy the requirements of s106 of the RMA. In terms of the MBIE (2017) guidance, this should consist of a Level C assessment that will provide a quantitative assessment of the liquefaction vulnerability of the village growth cells that is specific to the proposed land use. The level of uncertainty will be reduced so that a more precise liquefaction vulnerability category can be assigned and appropriate risk treatment options can be determined. The Level C assessment will provide a quantitative site wide assessment that should be based on Cone Penetrometer Testing, borehole log information and groundwater monitoring. The extent of the mine fill should be confirmed in order to delineate the area of *High Liquefaction Vulnerability.*

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6 Applicability

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

Tonkin & Taylor Ltd

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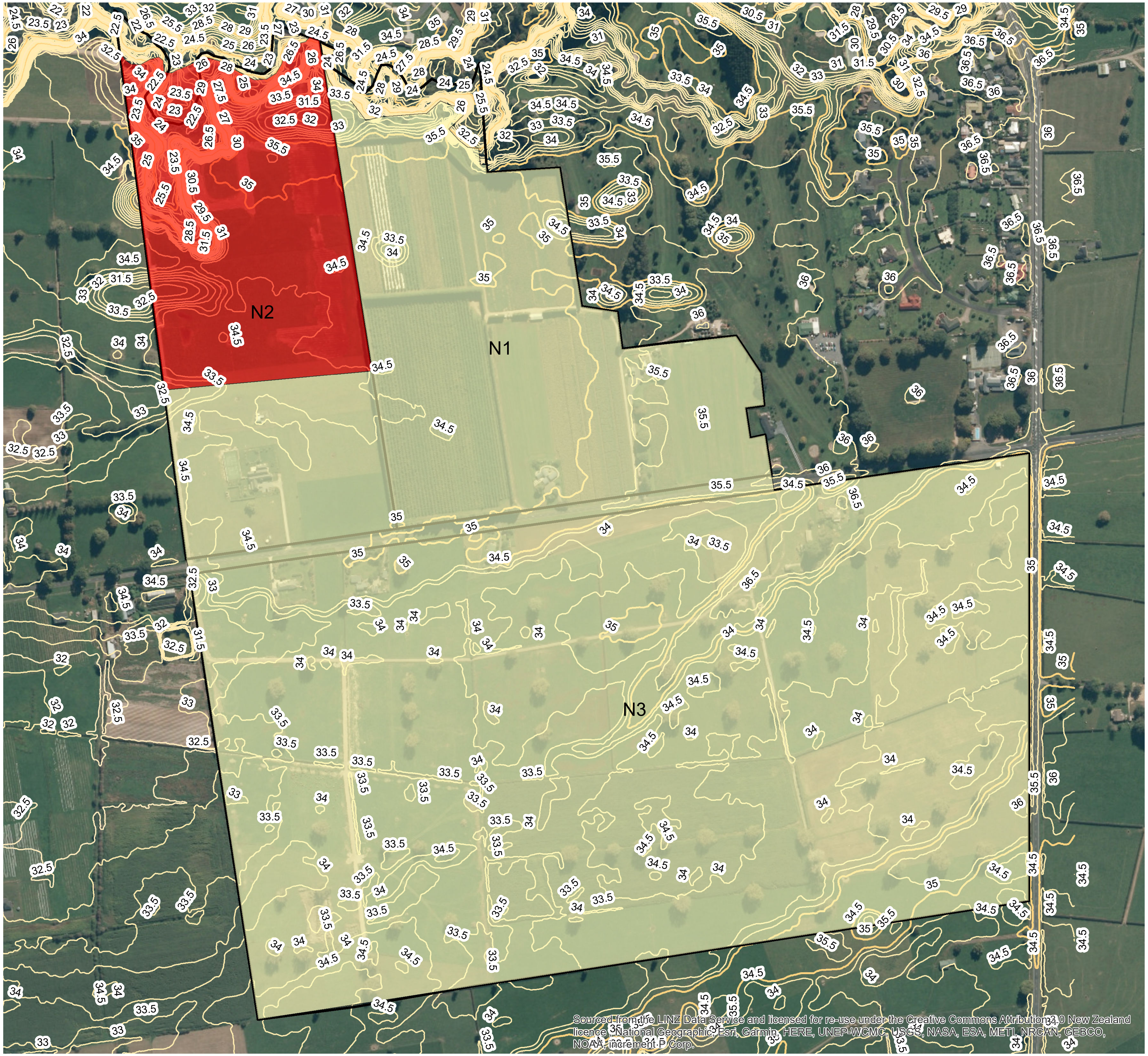
Appendix A: Geomorphic Zones



LEGEND

Ngahinapouri Growth Cells

Ngahinapouri Geomorphic Zones

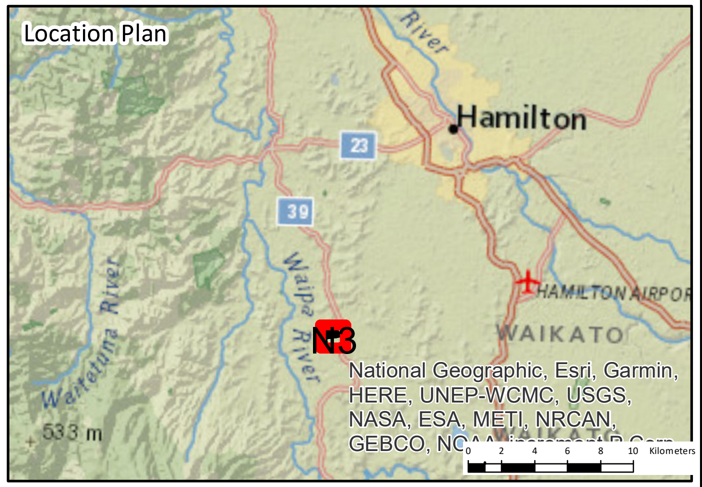
Alluviual Plains

Mined Area

A3 SCALE:1:5,000

0 40 80 120 160 200 Meters

N



Notes:

Topo map and aerial sourced from LINZ Data (Crown Copyright Reserved).

Applicability:

This figure has been prepared with respect to the particular brief given to Tonkin + Taylor.

Tonkin + Taylor do not accept responsibility for any loss or damage resulting from the use of the information and any person relying on the information does so at their own risk.

The basis of geomorphic zones are the geological mapping (Edbrooke, 2005), a Digital Elevation Model (DEM) derived from LiDAR data and a site walkover undertaken in January 2019.

This plan should be read in conjunction with the Ngahinapouri village plan liquefaction assessment report

DRAWN	JJBR	Jun.19
CHECKED	MJTT	Jun. 19
APPROVED	GGN	Aug.19

ARCFILE
Ngahinapouri_geomorph_map_20190116.mxd

SCALE (AT A3 SIZE)
1:5,000

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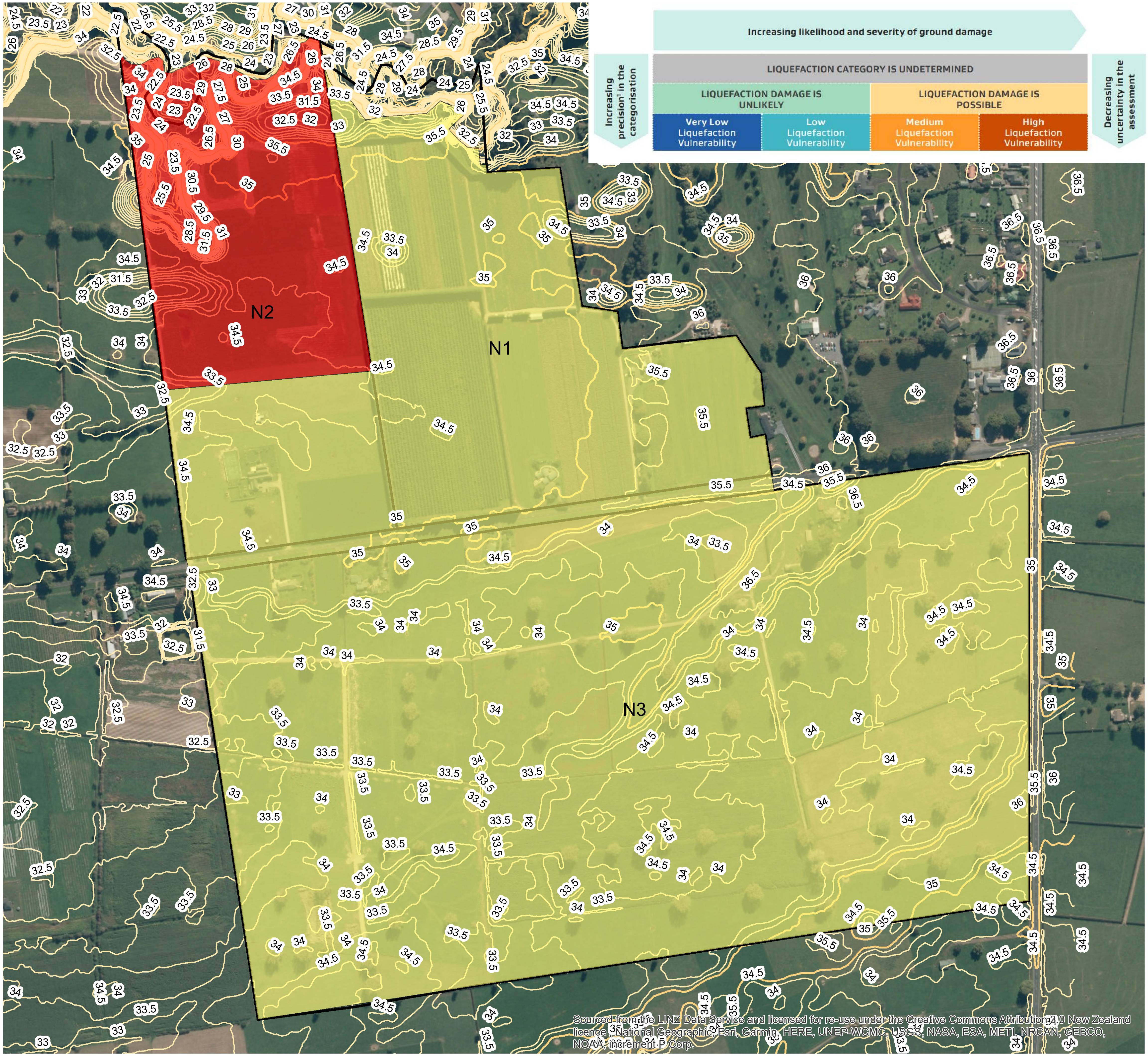
BOFFA MISKELL
WAIPA STRUCTURE PLAN
Ngahinapouri Township
Geomorphic Zones

Figure 1: Ngahinapouri Geomorphic Zones

# Appendix B:	Liquefaction Vulnerability Map



LEGEND

Ngahinapouri Growth Cells

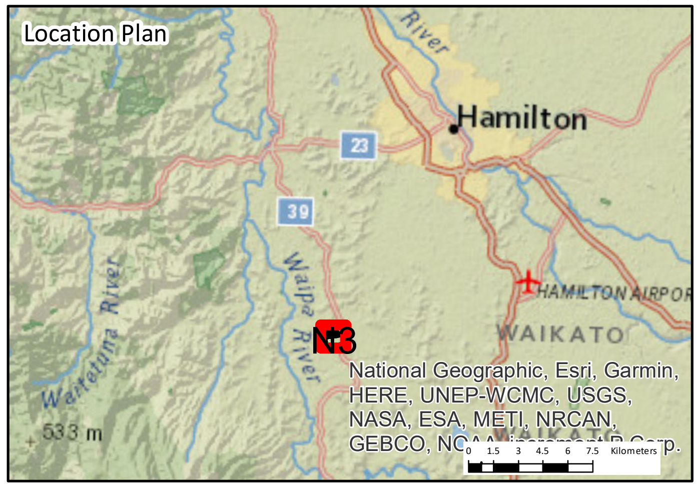
Liquefaction Vulnerability Categories

Liquefaction Damage is Possible

High Liquefaction Vulnerability

A3 SCALE:1:5,000

0 40 80 120 160 200 Meters



Notes:
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Liquefaction vulnerability categories determined by undertaken a Level A liquefaction assessment in line with MBIE/MfE/EQC (2017) guidelines.
This plan should be read in conjunction with the Ngahinapouri village plan liquefaction assessment report

DRAWN	JJBR	Aug.19
CHECKED	MJTT	Aug.19
APPROVED	GGN	Aug.19

ARCFILE
Ngahinapouri_liquefaction.mxd

SCALE (AT A3 SIZE)
1:5,000

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Liquefaction Vulnerability Map

Figure 2: Ngahinapouri Liquefaction Vulnerability Map

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