Appendix C: Case study examples

- **Appendix C.1: Liquefaction Assessment Summary Report Template**
- **Appendix C.2: Case Study – Scenario 1 (New Plymouth)**
- **Appendix C.3: Case Study – Scenario 2 (Inglewood)**
- **Appendix C.4: Case Study – Scenario 3 (Waitara)**

Liquefaction Assessment Summary Report Template

Version 1 – 26 June 2022

Overview

The following tables provide a template for an Assessment Summary Report for liquefaction vulnerability assessments. This report template draws on information provided in the New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021a), the Options for Liquefaction Assessment for Resource and Building Consent Report (Tonkin & Taylor, 2022) and relevant national guidance.

This template contains a summary of the following information:

- **Development information**
  - Proposed development, development scenario, location of development, existing liquefaction category, geomorphic terrain.
- **Geotechnical investigation information**
  - Number and depth of hand augers, Scala Penetrometer tests, shear vane tests, Cone Penetrometer tests, machine drilled boreholes, additional geotechnical investigations and testing.
- **Geotechnical parameters based on investigation data**
  - Description of ground conditions, groundwater conditions, seismic hazard (design parameters).
- **Assessment information**
  - Consulting practice, Chartered Engineer, Date of site assessment, Existing or previous geotechnical reports.
- **Summary of engineering assessment methodology and key parameters used**
  - Importance level of proposed development, method of liquefaction assessment, revised liquefaction vulnerability category, level of detail achieved, NZ standards or guidance documents referenced.

1. Development information	
Proposed Development	*e.g., two story residential dwelling, steel portal frame commercial development, six lot subdivision etc*
Development scenario	*As defined in NPDC liquefaction guidance flowchart e.g., Sparsely populated rural area, Rural-residential setting, Small-scale urban infill, commercial or industrial, urban residential development.*
Location of development	*Address and location of site*
Liquefaction vulnerability category assigned in NPDC Liquefaction Vulnerability Report (2021)	*Liquefaction Vulnerability Category assigned in New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021)*
Geomorphic terrain assigned in NPDC Liquefaction Vulnerability Report (2021)	*Geomorphic terrain assigned in New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021)*

2. Geotechnical investigation information	
Number and depth of hand augers (HA)	*List each hand auger undertaken and depth of test achieved*
Number and depth of Scala Penetrometer Tests or shear vane tests (SP or SV)	*List each type of test undertaken and depth of test achieved*
Number and depth of Cone Penetrometer Tests (CPT)	*List each CPT undertaken and depth of test achieved*
Number and depth of machine drilled boreholes (BH)	*List each BH undertaken and depth of test achieved*
Additional geotechnical investigations and testing	*List each CPT undertaken and depth of test achieved*

3. Geotechnical parameters based on investigation data	
Description of ground conditions	*Liquefiable soils? Granular materials or cohesive materials?*
Groundwater conditions (accounting for climate change and seasonal variations)	*Groundwater depth from geotechnical investigations*
Seismic hazard (design parameters)	*PGA and Magnitude for SLS and ULS design cases, and any other cases considered*

4. Assessment information	
Consulting practice	
Chartered Engineer: - Name - CPEngNZ Registration Number -Practice Area	
Date of site assessment	
Existing or previous geotechnical reports associated with development	

5. Summary of engineering assessment methodology and key parameters used	
Importance level of proposed development	*As per NZS 1170.0:2002*
Method of liquefaction assessment (if applicable)	*Option 1, Option 2 or Option 3 (Simplified Screening)*
Confirmed geomorphic terrain	*Confirmed geomorphic terrain applicable for the site. Explain how this was assessed, with evidence that either confirms the original mapped terrain from NPDC Liquefaction Vulnerability Report (2021) was correct, or shows that an alternative terrain is applicable.*
Groundwater level assumed for design	*Assumed depth below typical ground across the site, or specified as an RL if ground levels vary across site but groundwater RL remains similar.*
Confirmed liquefaction vulnerability category (as per MBIE/MfE 2017)	*Confirmed liquefaction vulnerability category in accordance with* MBIE/MfE (2017) *"Planning and engineering guidance for potentially liquefaction-prone land"*
Level of detail achieved	*Level of detail achieved in the assessment, in accordance with* MBIE/MfE (2017)
NZ Standards or Guidance documents referenced	

Case Study – Scenario 1 (New Plymouth)

Liquefaction Assessment Summary Report Template

Version 1 – 20 June 2022

Overview

The following tables provide a template for an Assessment Summary Report for liquefaction vulnerability assessments. This report template draws on information provided in the New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021), the Options for Liquefaction Assessment for Resource and Building Consent Report (Tonkin & Taylor, 2022) and relevant national guidance.

This template contains a summary of the following information:

- **Development information**
  - Proposed development, development scenario, location of development, existing liquefaction category, geomorphic terrain.
- **Geotechnical investigation information**
  - Number and depth of hand augers, Scala Penetrometer tests, shear vane tests, Cone Penetrometer tests, machine drilled boreholes, additional geotechnical investigations and testing.
- **Geotechnical parameters based on investigation data**
  - Description of ground conditions, groundwater conditions, seismic hazard (design parameters).
- **Assessment information**
  - Consulting practice, Chartered Engineer, Date of site assessment, Existing or previous geotechnical reports.
- **Summary of engineering assessment methodology and key parameters used**
  - Importance level of proposed development, method of liquefaction assessment, revised liquefaction vulnerability category, level of detail achieved, NZ standards or guidance documents referenced.

1. Development information	
Proposed Development	We understand that the client wishes to subdivide the site into two separate residential lots. The existing dwelling will remain on the northern lot while the southern lot will remain undeveloped at this point in time. We understand that the client intends to sell the property once subdivided
Development scenario	Development Scenario 3 from NPDC liquefaction guidance flowchart: Small-scale urban infill
Location of development	New Plymouth township *(Site specific details would be required here)*
Liquefaction vulnerability category assigned in NPDC Liquefaction Vulnerability Report (2021)	Liquefaction Category is Undetermined
Geomorphic terrain assigned in NPDC Liquefaction Vulnerability Report (2021)	Coastal terraces

2. Geotechnical investigation information	
Number and depth of hand augers (HA)	3 no. hand auger (HA01 to HA03) boreholes to a target depth of 3 m below the current ground level. 1 no. hand auger (HA04) borehole to a target depth of 5 m below the current ground level.
Number and depth of Scala Penetrometer Tests or shear vane tests (SP or SV)	Scala Penetrometer testing (SP) and/or handheld shear vane (SV) tests within the hand augered boreholes to depths of between 3 – 5 m below the current ground level.
Number and depth of Cone Penetrometer Tests (CPT)	2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 15 m below the current ground level.
Number and depth of machine drilled boreholes (BH)	*N/A*
Additional geotechnical investigations and testing	*N/A*

3. Geotechnical parameters based on investigation data	
Description of ground conditions	Topsoil and stiff SILT comprises upper 3 m of soil profile. Potentially liquefiable soils from interbedded with silt, peat and gravel sediments from 3 m to 15 m below ground level.
Groundwater conditions (accounting for climate change and seasonal variations)	Groundwater level typically 2.4 – 2.6 m below ground level. Modelled groundwater depth of 2.0 m below ground surface (accounting for seasonal fluctuations and climate change).
Seismic hazard (design parameters)	1 in 25 year (SLS): 0.07 g 1 in 100 year (Intermediate 1): 0.14 1 in 250 year (Intermediate 2): 0.21 1 in 500 year (ULS): 0.29 g

4. Assessment information	
Consulting practice	*Enter details here*
Chartered Engineer: - Name - CPEngNZ Registration Number -Practice Area	*Enter details here*
Date of site assessment	31/05/2022
Existing or previous geotechnical reports associated with development	N/A

5. Summary of engineering assessment methodology and key parameters used	
Importance level of proposed development	Importance Level 2
Method of liquefaction assessment (if applicable)	Option 2 from NPDC liquefaction guidance flowchart: Site-specific geotechnical engineering assessment and use of MBIE Canterbury Guidance (2018)
Confirmed geomorphic terrain	Coastal terraces. No change from terrain mapped in NPDC Liquefaction Vulnerability Report (2021). Ground conditions encountered on site were consistent with the mapped geology and geomorphology (i.e., predominantly silts and sands). Taranaki Brown Ash was identified in some investigations. Static groundwater levels varied from approximately 6 – 8 m RL (NZVD 2016) on sites at similar elevations.
Groundwater level assumed for design	2.0 m below the current ground level.
Confirmed liquefaction vulnerability category (as per MBIE/MfE 2017)	Medium Liquefaction Vulnerability
Level of detail achieved	Level D level of detail (Site-specific assessment)
NZ Standards or Guidance documents referenced	MBIE/MfE, 2017. *Planning and engineering guidance for potentially liquefaction-prone land,* Wellington: Ministry of Business, Innovation and Employment & Ministry for the Environment. MBIE/NZGS, 2021a. *Earthquake geotechnical engineering practice - Module 1: Overview of the guidelines,* Wellington: s.n. MBIE/NZGS, 2021c. *Earthquake geotechnical engineering practice - Module 3: Identification, assessment and mitigation of liquefaction hazards,* Wellington: s.n. MBIE, 2018. *Repairing and rebuilding houses affected by the Canterbury earthquakes. Version 3,* Wellington, New Zealand: Ministry of Business, Innovation and Employment.

Case Study – Scenario 1 (New Plymouth)

1 Introduction

Note: Blue italicised text throughout this report represents general notes regarding the content that should be addressed by each heading/subheading.

1.1 General

General overall text about what report is intended to be used for (e.g., resource consent, building consent etc).

This geotechnical investigation and interpretative report has been prepared to support a resource consent application for sub-division of the site.

1.2 Objectives of work

Outline objectives of work i.e. to determine if "good ground" present, to determine if land is suitable for subdivision, meets requirements of RMA etc.

1.3 Site description

The site of interest (here in known as "the site") is a residential property located within the New Plymouth Township with a land area of approximately 980 m^2. The site is generally flat and is positioned approximately 10 m above sea level and approximately 400 m away from the Taranaki coastline. The land slopes gently towards the coast before dropping steeply down to the beach at a height of 1 – 2 m. A single storey residential dwelling currently occupies the northern half of the site.

Report should provide figure to show the location of the site.

1.4 Proposed development

We understand that the client wishes to subdivide the site into two separate residential lots as shown on the attached site plan in Appendix *XX*. The existing dwelling will remain on the northern lot while the southern lot will remain undeveloped at this point in time. We understand that the client intends to sell the property once subdivided.

Report should include relevant conceptual or detailed drawings of the proposed development if available.

2 Assessment and interpretation of site conditions

2.1 Geology and faulting

A geological map published by GNS (Townsend, et al., 2008) indicates that the site is underlain by Late Pleistocene shoreline deposits comprising shallow marine conglomerate, shell beds, dune sands and peat. Geomorphology mapping undertaken by T+T (Tonkin & Taylor Ltd, 2021) shows that the site is located in the area mapped as Coastal Terraces terrain. The nearest mapped active fault to the site is the Inglewood Fault which lies approximately 20 km to the southeast of the site.

2.2 Known natural hazards

The liquefaction vulnerability category of the site was assessed by T+T as **Liquefaction Vulnerability Category is Undetermined**, this means at the time of undertaking the assessment there was *"insufficient information to characterise the expected land performance".* That liquefaction vulnerability assessment was undertaken to a Level A (Basic Desktop Assessment) – level of detail in accordance with the MBIE/MfE Guidance (2017). This report includes review of that liquefaction vulnerability assessment utilising the new site-specific geotechnical investigations undertaken.

Generally based on natural hazard information sourced from Council GIS platforms and/or site-specific assessments undertaken by other suitably qualified professionals. All sources should be referenced appropriately. Other typical hazards considered may include:

- *Coastal*
- *Flooding*
- *Wind*
- *Land stability*
- *Soil maps*

2.3 Geotechnical investigations

2.3.1 Desktop assessment

Prior to scoping geotechnical site investigations, a desktop assessment was undertaken for the site. This assessment reviewed existing geotechnical investigations on nearby sites to understand potential ground and groundwater conditions, and to inform geotechnical investigation requirements.

Geotechnical investigations on nearby sites indicated ground conditions that were consistent with the mapped geology and geomorphology (i.e., predominantly silts and sands). Taranaki Brown Ash was identified in some investigations. Static groundwater levels varied from approximately 6 – 8 m RL (NZVD 2016) on sites at similar elevations.

2.3.2 Scope of geotechnical investigations

The scope of site-specific ground investigations was developed to target key geotechnical matters identified in the desktop assessment. Geotechnical investigations on the site comprised the following:

- 3 no. hand auger (HA01 to HA03) boreholes to a target depth of 3 m below the current ground level.
- 1 no. hand auger (HA04) borehole to a target depth of 5 m below the current ground level.
- Scala Penetrometer testing (SP) and/or handheld shear vane (SV) tests within the hand augered boreholes to depths of between 3 – 5 m below the current ground level.
- 2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 15 m below the current ground level.

The location of these geotechnical investigations is shown on the attached site plan in Appendix *XX* and the investigation results are attached in Appendix *XX*.

2.3.3 Hand auger boreholes

The drilling of the 4 no. hand auger boreholes, (HA1 to HA4) was undertaken on 14 February 2022. In situ shear strength testing in cohesive materials was undertaken at 0.3 m intervals and dynamic cone penetrometer in the non-cohesive materials. These were carried out by an experienced Engineering Geologist, who logged the boreholes to the New Zealand Geotechnical Society (NZGS) logging guidelines (NZGS, 2005).

Investigation locations are presented on Figure *XX*, Appendix *XX,* hand auger borehole logs are presented in Appendix *XX*, and Table 2.1 provides a summary of the hand auger locations and depths.

Table 2.1: Hand auger borehole summary

HA ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)
HA1	*XXXXX*	10.1	3.0
HA2	*XXXXX*	10.2	3.0
HA3	*XXXXX*	9.8	3.0
HA4	*XXXXX*	9.8	5.0

2.3.4 Scala penetrometer tests/shear vane tests

The Scala Penetrometer/shear vane test locations are presented on Figure *XX*, Appendix *XX,* and Scala Penetrometer/shear vane test results are presented on the hand auger borehole logs in in Appendix *XX*. Table 2.2 provides a summary of the Scala Penetrometer locations and depths.

Table 2.2: Scala Penetrometer Test Summary

Scala Penetrometer/Shear Vane test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
SP1	*XXXXX*	10.1	3.0	Target depth
SP2	*XXXXX*	10.2	3.0	Target depth
SP3	*XXXXX*	9.8	3.0	Target depth
SP4	*XXXXX*	9.8	5.0	Target depth

2.3.5 Cone Penetrometer Tests

The Cone Penetrometer Test (CPT) locations are presented on Figure *XX*, Appendix *XX*, and Table 2.3 provides a summary of the CPT locations and depths.

Table 2.3: Cone Penetrometer Test Summary

Cone Penetrometer Test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
CPT01	*XXXXX*	10.1	15.0	Target depth
CPT02	*XXXXX*	9.8	15.0	Target depth

2.3.6 Groundwater

Groundwater levels within the investigations were recorded using an electronic dip meter on completion of drilling. The recorded groundwater levels are presented below in Table 2.3, with ground surface elevations obtained from the New Plymouth District Council GIS platform.

Table 2.3: Groundwater levels

HA ID	Ground Surface Elevation (NZVD 2016)	Static Groundwater Level RL (NZVD 2016)	Depth Below Ground Level (m)	Date of measurement
HA1	10.1	7.6	2.5	15/02/2022
HA2	10.2	7.8	2.4	15/02/2022
HA3	9.8	7.2	2.6	15/02/2022
HA4	9.8	7.3	2.5	15/02/2022

2.4 Geotechnical model

The geological profile presented in this report and in associated appendices is based upon information obtained from the recently completed hand-auger boreholes and CPT. The nature and continuity of the subsoil profile away from these locations is inferred but it must be appreciated that actual conditions may vary from the assumed model.

A summary of the generalised geological profile is presented in Table 2.4 and an interpretive geological cross section is presented in Figure *XX*, Appendix *XX*. The geological cross section of the site was developed using site survey, geological information, and measured groundwater levels.

Table 2.3: Generalised geological profile

Geological Unit	Soil Description	Typical depth to base of layer (m)	Typical layer thickness (m)	Scala Penetrometer (blows/100 mm)	Undrained shear strength (kPa)	Cone tip resistance (MPa)
Topsoil	Organic silty TOPSOIL	0.3 – 0.5	0.3 – 0.5	1 – 2	32 – 50	>2
Taranaki Brown Ash	Stiff to very stiff SILT with some clay	3.0 – 3.5	2.7 – 3.0	2 – 4	60 – 110	>2
Late Pleistocene shoreline deposits	Loose to medium dense silty SAND	5.0	2.5 – 3.0	3 – 6	-	5 - 10
	Predominantly sandy SILT and silty SAND layers with minor peat and gravel layers	15.0	10.0	-	-	

3 Geotechnical considerations

3.1 General

Recommendations and opinions contained in this report are based on our visual appraisal of the site, as well as the assessed geotechnical investigations and our experience and knowledge of the surrounding area. The nature and continuity of the soil conditions away from the test locations is inferred but it must be appreciated that actual conditions may vary from the assumed model.

Based on the field investigations and engineering analyses, the site is generally considered suitable for the proposed development, subject to the following geotechnical considerations:

1 Liquefaction;

2 Consolidation settlement;

3 Expansive soils;

4 Earthworks;

5 Retention structures; and

6 Foundation options and design parameters;

It should be noted that there may be other factors that need to be considered in foundation design that are outside of this geotechnical scope of works (i.e., flood hazard zones, minimum floor levels etc).

3.2 Liquefaction

This section of the report should present the findings of a liquefaction vulnerability assessment based on the assessed geotechnical investigations and the consultants experience and knowledge of the surrounding area.

It should generally outline the methodology and software used and present the results in terms of a range of liquefaction vulnerability parameters (e.g., post-liquefaction one-dimensional volumetric consolidation (S_{V1D}), lateral spread displacements and Liquefaction Severity Number (LSN)). At a minimum, these parameters should be calculated for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) earthquake events in accordance with the requirements of the Building Act.

Module 3 of MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c) *also recommends the consideration of intermediate earthquake events to identify the point of "step change" in liquefaction performance. This approach is known as the "holistic evaluation of performance." Seismic hazard design parameters for New Plymouth recommended for use in geotechnical design are provided in Appendix A of Module 1 MBIE/NZGS Earthquake geotechnical engineering practice series* (MBIE/NZGS, 2021a).

The results of these assessments are typically displayed in a table and can be used to inform both the assessment of liquefaction vulnerability at the site in accordance with the MBIE/MfE Guidance (2017) and the adoption of a suitable foundation design.

3.2.1 Method of liquefaction assessment

Liquefaction assessment of the CPT investigations has been carried out in accordance with Module 3 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c). Liquefaction potential and the calculation of post liquefaction induced settlements have been assessed using the methodology developed by Idriss & Boulanger (2014) and Zhang et al (2002) respectively. The method of Robertson and Wride (1998) has been applied to account for the effect of grain characteristics or fine content.

3.2.2 Earthquake return periods

The New Zealand design loads code NZS 1170 defines to design conditions which need to be assessed for the purpose of liquefaction assessments.

- **ULS – Ultimate Limit State** is concerned with ground damage associated with a 500-year earthquake event, for which buildings should be designed to avoid collapse and potential loss of life.
- **SLS – Serviceability Limit State** is concerned with ground damage associated with smaller earthquakes with a return period of 25 years. Buildings and their non-structural components should be designed to withstand permanent damage for a 25-year event i.e. readily repairable.

Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) recommends that intermediate seismic events also be considered alongside the SLS and ULS requirements to determine if a significant step change in ground performance is present between the SLS and ULS design events. Following review of the seismic hazard information for New Plymouth we have selected the 1 in 100-year and 1 in 250-year return periods as intermediate events.

The 1 in 100-year and 1 in 500-year events are also recommended in the MBIE/MfE Guidance (2017) as the minimum return periods for assessment of liquefaction vulnerability.

3.2.3 Ground motion inputs

The ground motion inputs for analysis were obtained from Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) for SLS, ULS and the selected intermediate events. Table 3.1 outlines the ground motion inputs.

Table 3.1: Ground motion inputs obtained from Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series.

Earthquake event	PGA Value (g)	Earthquake Magnitude
1 in 25 year (SLS)	0.07	6
1 in 100 year (Intermediate 1)	0.14	6
1 in 250 year (Intermediate 2)	0.21	6
1 in 500 year (ULS)	0.29	6

3.2.4 Ground water assessment

The deepest hand auger undertaken during the investigation (HA4) extended to a depth of 5.0 m below the current ground level. The sediments logged throughout this depth were described as being dry to wet with the water table being encountered at between 2.4 – 2.6 m. A shallower groundwater depth of 2.0 m below the current ground level was used for the liquefaction calculations presented below. This shallower design groundwater depth is based on the potential for elevated groundwater levels associated with seasonal variation.

Given the relatively close proximity of the site to the Taranaki coastline there is also the potential for sea level rise associated with climate change to further elevate long term groundwater levels. Therefore, sensitivity analysis to understand the effect of groundwater assumptions on liquefaction vulnerability has also been undertaken.

3.2.5 Liquefaction vulnerability assessment

Using the liquefaction assessment method discussed in Section 3.2.1 above, combined with a design ground water level of 2.0 m, there is the potential for liquefaction to occur within the upper 10 m of the soil profile. Liquefaction is predicted to occur in the loose to medium dense silty sand deposits that are encountered at a depth below 3 – 3.5 m. A non-liquefiable crust comprising predominantly stiff to very stiff silt overlies the potentially liquefiable material. Table 3.2 summarises the liquefaction vulnerability parameters (S_{V1D} and LSN) for the assumed ground motion parameters.

Table 3.2: S_{V1D} and LSN values at CPT01 and CPT02 for selected ground motions

Earthquake event	S_{V1D}		LSN	
	CPT01	CPT02	CPT01	CPT02
1 in 25 year (SLS)	0	0	0	0
1 in 100 year (Intermediate 1)	10	5	2	3
1 in 250 year (Intermediate 2)	25	30	8	10
1 in 500 year (ULS)	60	90	12	14

*Based on groundwater level at 2.0 m below ground level.

Groundwater sensitivity analysis indicated that there was no significant change to the calculated liquefaction vulnerability parameters. This is because liquefiable layers in the soil profiles are located at depths greater than 3 m.

The nearest free-face is the Taranaki Coastline. It is less than 2m high and is located more than 400 m from the site and therefore lateral spreading is unlikely to cause consequential liquefaction related land damage at the site.

Interpretation of the liquefaction vulnerability parameters provided is as follows:

- At 1 in 25 year (SLS) and 1 in 100 year levels of earthquake shaking liquefaction related damage is likely to be no more than None to Minor
- At 1 in 250 year and 1 in 500 year (ULS) levels of earthquake shaking liquefaction related damage is likely to be no more than Minor to Moderate

Based on the values provided in Table 3.2 and Table 2.2 of the MBIE/MfE Guidance (2017) "*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*" and as a result, we have categorised the site as having a Medium Liquefaction Vulnerability. This liquefaction vulnerability assessment has been undertaken to a Level D – Level of detail (Site-specific assessment) in accordance with the MBIE/MfE Guidance (2017).

3.3 Consolidation settlement

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.4 Expansive soils

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.5 Slope stability

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.6 Earthworks

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.7 Retention structures

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.8 Contamination

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.9 Stormwater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.10 Wastewater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.11 Foundation recommendations

Comment: Foundation recommendations should then be given based on the information in Section 3.2 to Section 3.7 above. If this geotechnical report is being prepared for a subdivision resource consent, site specific recommendations may be given for further testing at individual sites.

Due to the site being categorised as having a Medium Liquefaction vulnerability (refer to Section 3.2.5), is it considered to be "prone to liquefaction or lateral spreading" and therefore **does not meet the definition of "Good Ground"** as outlined in the Building Code amendments.

Geotechnical investigations undertaken on the site show that a geotechnical ultimate bearing capacity of 200 kPa is generally available across the site at a depth of 0.5 m below ground level.

Based on the above results, we have categorised the site as having a Medium liquefaction vulnerability. Recent information uploaded to the MBIE Building Performance website (MBIE, 2022a) recommends TC2 type foundation options for a Medium liquefaction vulnerability. Furthermore, Table 3.1 of the MBIE Canterbury Guidance (MBIE, 2018) also indicates that the calculated S_{V1D} values for both the SLS and ULS design events are consistent with those specified for a TC2 foundation solution. Figure 5.2 of the MBIE Canterbury Guidance (2018) shows that for a stiffened raft slab foundation type would be suitable for a TC2 site with a geotechnical ultimate bearing capacity greater than 200 kPa.

There are three different types of TC2 foundation options which are dependent on the geotechnical ultimate bearing capacity available on the site. Foundations could comprise shallow pile/foundation wall system (Types A & B), Stiffened raft slab (Type C) or foundations that are specifically designed by a chartered professional geotechnical engineer. Please see Figure 5.2 of the MBIE Canterbury Guidance "Repairing and rebuilding houses affected by the Canterbury earthquake" for Futher information.

4 Resource Management Act – Section 106

Comment: This section would only be included for geotechnical reports to support subdivision consent applications.

Section 106 of the RMA (1991) includes subdivision consent provisions relating to risk from natural hazards. This includes a combined assessment of likelihood, material damage and subsequent use, and the option of specifying consent conditions for the purpose of avoiding, remedying or mitigating the effects of natural hazards. This geotechnical report is intended to help inform a Section 106 assessment by providing information about geotechnical-related natural hazards:

- The proposed development at the site is considered feasible from a geotechnical perspective.
- The key geotechnical-related natural hazard for the site is considered to be earthquake-induced liquefaction. Given the topography and geographical location of the site, other geotechnical hazards are considered to either have a low likelihood of occurring or are unlikely to result in significant material damage to land or structures.
- If the recommendations detailed in this report are followed, we consider that:
  - The likely subsequent use of the land is unlikely to accelerate, worsen or result in geotechnical-related hazards.
  - Liquefaction-related natural hazard risk can be appropriately mitigated by adopting the foundation recommendations provided in this report.

5 Statement of professional opinion on suitability of land

Refer to Schedule 2A of NPDC "Land Development and Subdivision Infrastructure Standard"

6 References

MBIE/MfE, 2017. *Planning and engineering guidance for potentially liquefaction-prone land,* Wellington: Ministry of Business, Innovation and Employment & Ministry for the Environment.

MBIE/NZGS, 2021a. *Earthquake geotechnical engineering practice - Module 1: Overview of the guidelines,* Wellington: s.n.

MBIE/NZGS, 2021c. *Earthquake geotechnical engineering practice - Module 3: Identification, assessment and mitigation of liquefaction hazards,* Wellington: s.n.

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NZGS, 2005. *Field description of soil and rock, guideline for the field classification and description of soil and rock for engineering purposes,* New Zealand: New Zealand Geotechnical Society.

Tonkin & Taylor Ltd, 2021. *New Plymouth District Liquefaction Vulnerability Assessment. Report reference 1016765.v1,* Tauranga, New Zealand: Tonkin & Taylor Ltd.

Townsend, D., Vonk, A. & Kamp, P. J. J., 2008. *Geology of the Taranaki area. 1:250 000 geological map 7,* Lower Hutt, New Zealand: Institute of Geological & Nuclear Sciences.

Case Study 2 – Scenario 2 (Inglewood)

Liquefaction Assessment Summary Report Template

Version 1 – 20 June 2022

Overview

The following tables provide a template for an Assessment Summary Report for liquefaction vulnerability assessments. This report template draws on information provided in the New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021), the Options for Liquefaction Assessment for Resource and Building Consent Report (Tonkin & Taylor, 2022) and relevant national guidance.

This template contains a summary of the following information:

Development information

Proposed development, development scenario, location of development, existing liquefaction category, geomorphic terrain.

Geotechnical investigation information

Number and depth of hand augers, Scala Penetrometer tests, shear vane tests, Cone Penetrometer tests, machine drilled boreholes, additional geotechnical investigations and testing.

Geotechnical parameters based on investigation data

Description of ground conditions, groundwater conditions, seismic hazard (design parameters).

Assessment information

Consulting practice, Chartered Engineer, Date of site assessment, Existing or previous geotechnical reports.

Summary of engineering assessment methodology and key parameters used

Importance level of proposed development, method of liquefaction assessment, revised liquefaction vulnerability category, level of detail achieved, NZ standards or guidance documents referenced.

1. Development information	
Proposed Development	We understand that the client wishes to construct a 1000 m^2 commercial building. Conceptual drawings provided by the client show the proposed commercial building comprising a portal frame structure founded on shallow concrete pads with a concrete floor slab.
Development scenario	Development Scenario 4 from NPDC liquefaction guidance flowchart: Commercial or Industrial Development
Location of development	Inglewood *(Site specific details would be required here)*
Liquefaction vulnerability category assigned in NPDC Liquefaction Vulnerability Report (2021)	Liquefaction Category is Undetermined
Geomorphic terrain assigned in NPDC Liquefaction Vulnerability Report (2021)	Lahars geomorphic terrain

2. Geotechnical investigation information	
Number and depth of hand augers (HA)	- 3 no. hand auger (HA01 to HA03) boreholes to a target depth of 3 m below the current ground level.
Number and depth of Scala Penetrometer Tests or shear vane tests (SP or SV)	- Scala Penetrometer testing (SP) and/or handheld shear vane (SV) tests within the hand augered boreholes to a target depth of 3 m below the current ground level.
Number and depth of Cone Penetrometer Tests (CPT)	- 2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 20 m below the current ground level.
Number and depth of machine drilled boreholes (BH)	- 1 no. machine-drilled borehole (BH01)
Additional geotechnical investigations and testing	*N/A*

3. Geotechnical parameters based on investigation data	
Description of ground conditions	- 0.5 m of topsoil - 0.4 m of sandy SILT - 5.5 m of stiff to very stiff SILT - 5 m of medium dense to dense SAND - 8 m alternating dense gravel, cobbles and silt layers
Groundwater conditions (accounting for climate change and seasonal variations)	Groundwater depth from geotechnical investigations
Seismic hazard (design parameters)	1 in 25 year (SLS): 0.7 g 1 in 100 year (Intermediate 1): 0.14 g 1 in 200 year (Intermediate 2): 0.21 g 1 in 500 year (ULS): 0.29 g

4. Assessment information	
Consulting practice	*Enter details here*
Chartered Engineer: - Name - CPEngNZ Registration Number -Practice Area	*Enter details here*
Date of site assessment	*31/05/2022*
Existing or previous geotechnical reports associated with development	*N/A*

5. Summary of engineering assessment methodology and key parameters used	
Importance level of proposed development	Importance Level 2
Method of liquefaction assessment (if applicable)	Option 1 from NPDC liquefaction guidance flowchart: Site-specific geotechnical engineering assessment
Confirmed geomorphic terrain	Lahars. No change from terrain mapped in NPDC Liquefaction Vulnerability Report (2021). Ground conditions encountered on site were consistent with the mapped geology and geomorphology (i.e., highly variable including silts, sands, conglomerates, and peat and other organic material). Taranaki Brown Ash was identified in some investigations.
Groundwater level assumed for design	7.5 m below the current ground level.
Confirmed liquefaction vulnerability category (as per MBIE/MfE 2017)	Low Liquefaction Vulnerability
Level of detail achieved	Level D level of detail (Site-specific assessment)
NZ Standards or Guidance documents referenced	MBIE. (2017). *Planning and engineering guidance for potentially liquefaction-prone land.* Wellington: Ministry of Business, Innovation and Employment & Ministry for the Environment. MBIE/NZGS. (2021a). *Earthquake geotechnical engineering practice - Module 1: Overview of the guidelines.* Wellington. MBIE/NZGS. (2021b). *Earthquake geotechnical engineering practice - Module 2: Geotechnical investigations for earthquake engineering.* Wellington. MBIE/NZGS. (2021c). *Earthquake geotechnical engineering practice - Module 3: Identification, assessment and mitigation of liquefaction hazards.* Wellington. MBIE/NZGS. (2021d). *Earthquake geotechnical engineering practice - Module 4: Earthquake resistant foundation design.* Wellington, New Zealand: NZGS/MBIE.

Case Study – Scenario 2 (Inglewood)

1 Introduction

Note: Blue italicised text throughout this report represents general notes regarding the content that should be addressed by each heading/subheading.

1.1 General

General overall text about what report is intended to be used for (Resource consent, plan change, building consent etc).

This geotechnical investigation and interpretation report has been prepared to support a building consent application for a commercial building.

1.2 Objectives of work

Outline objectives of work i.e., to determine if “good ground” present, to determine if land is suitable for subdivision, meets requirements of RMA etc.

1.3 Site description

The site of interest (here in known as “the site”) is an industrial property located in the Inglewood Township with an area of approximately 2000 m^2. The site is generally flat and is currently undeveloped. The site is surrounded by other commercial and industrial buildings.

Report should provide a figure to show the location of the site.

1.4 Proposed development

We understand that the client wishes to construct a 1000 m^2 commercial building as shown on the attached site plan in Appendix *XX*. Conceptual drawings provided by the client show the proposed commercial building comprising a portal frame structure founded on shallow concrete pads with a concrete floor slab.

Report should include relevant conceptual or detailed drawings of the proposed development if available.

2 Assessment and interpretation of site conditions

2.1 Geology and faulting

A geological map published by GNS (Townsend, et al., 2008) indicates that the site is underlain by Holocene lahar flow deposits of the Egmont Volcanic Centre. These sediments are likely to comprise "*multiple beds of andesitic conglomerate and sand, some with broken tree trunks and branches, and pyroclastic flow deposits*". Geomorphology mapping undertaken by T+T (Tonkin & Taylor Ltd, 2021) shows that the site is located in the area mapped as Lahars. This terrain comprises highly variable sediments both in terms of age and composition. The nearest mapped active fault to the site is the Inglewood fault which lies approximately 500 m to the North of the site.

2.2 Known natural hazards

The liquefaction vulnerability category of the site was assessed by T+T as **Liquefaction Vulnerability Category is Undetermined** this means at the time of undertaking the assessment there was *"insufficient information to characterise the expected land performance"* (Tonkin & Taylor Ltd, 2021). This existing liquefaction vulnerability assessment was undertaken to a Level A – level of detail in accordance with the MBIE/MfE Guidance (2017) document. This report includes review of that liquefaction vulnerability assessment utilising the new site-specific geotechnical investigations undertaken.

Generally based on natural hazard information sourced from Council GIS platforms and/or site-specific assessments undertaken by other suitably qualified professionals. All sources should be referenced appropriately. Other typical hazards considered may include:

- *Coastal*
- *Flooding*
- *Wind*
- *Land stability*
- *Soil maps*

2.3 Geotechnical investigations

2.3.1 Desktop assessment

Prior to scoping geotechnical site investigations, a desktop assessment was undertaken for the site. This assessment reviewed existing geotechnical investigations on nearby sites to understand potential ground and groundwater conditions, and to inform geotechnical investigation requirements.

Geotechnical investigations on nearby sites indicated ground conditions that were consistent with the mapped geology and geomorphology (i.e., highly variable including silts, sands, conglomerates, and peat and other organic material). Taranaki Brown Ash was identified in some investigations. Static groundwater levels varied from approximately 190 - 195 m RL (NZVD 2016) on sites at similar elevations.

2.3.2 Scope of geotechnical investigations

The scope of site-specific ground investigations was developed to target key geotechnical matters identified in the desktop assessment. Geotechnical investigations on the site comprised the following:

- 3 no. hand auger (HA01 to HA03) boreholes to a target depth of 3 m below the current ground level.
- Scala Penetrometer testing (SP) and/or handheld shear vane (SV) tests within the hand augered boreholes to a target depth of 3 m below the current ground level.
- 1 no. machine-drilled borehole (BH01) with associated standard penetration tests to a target depth of 20 m below the current ground level
- 2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 20 m below the current ground level.

The location of these geotechnical investigations is shown on the attached site plan in Appendix *XX* and the investigation results are attached in Appendix *XX*.

Note: Table 2.3 of Module 2 of MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021b) *outlines testing frequency required for detailed design/building consent applications (note the following "Not to be substituted for higher levels of investigation if required by other standards or local authority requirements").*

2.3.3 Hand auger boreholes

The drilling of the 3 no. hand auger boreholes, (HA1 to HA3) was undertaken on 14 February 2022. In situ shear strength testing in cohesive materials was undertaken at 0.3 m intervals and dynamic cone penetrometer in the non-cohesive materials. These were carried out by an experienced Engineering Geologist, who logged the boreholes to the New Zealand Geotechnical Society logging guidelines (NZGS, 2005).

Investigation locations are presented on Figure *XX*, Appendix *XX,* hand auger borehole logs are presented in Appendix *XX*, and Table 2.1 provides a summary of the hand auger locations and depths.

Table 2.1: Hand auger borehole summary

HA ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)
HA1	*XXXXX*	203.5	3.0
HA2	*XXXXX*	203.2	3.0
HA3	*XXXXX*	202.9	3.0

2.3.4 Scala penetrometer tests/shear vane tests

The Scala Penetrometer/shear vane test locations are presented on Figure *XX*, Appendix *XX,* and Scala Penetrometer/shear vane test results are presented on the hand auger borehole logs in in Appendix *XX*. Table 2.2 provides a summary of the Scala Penetrometer locations and depths.

Table 2.2: Scala Penetrometer Test Summary

Scala Penetrometer/Shear Vane test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
SP1	*XXXXX*	203.5	3.0	Target depth
SP2	*XXXXX*	203.2	3.0	Target depth
SP3	*XXXXX*	202.9	3.0	Target depth

2.3.5 Machine-drilled boreholes

The machine-drilled borehole test location is presented on Figure *XX*, Appendix *XX*. A summary of the test is presented in Table 2.3.

Table 2.3: Scala Penetrometer Test Summary

Cone Penetrometer Test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
BH01	*XXXXX*	203.5	20.0	Target depth

2.3.6 Cone Penetrometer Tests

The Cone Penetrometer Test (CPT) locations are presented on Figure *XX*, Appendix *XX*, and Table 2.3 provides a summary of the CPT locations and depths.

Table 2.4: Scala Penetrometer Test Summary

Cone Penetrometer Test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
CPT01	*XXXXX*	203.5	20.0	Target depth
CPT02	*XXXXX*	202.9	20.0	Target depth

2.3.7 Groundwater

Groundwater levels within the investigations were recorded using an electronic dip meter on completion of drilling. The recorded groundwater levels are presented below in Table 2.3, with ground surface elevations obtained from the New Plymouth District Council GIS platform.

Table 2.3: Groundwater levels

HA ID	Ground Surface Elevation (NZVD 2016)	Static Groundwater Level RL (NZVD 2016)	Depth Below Ground Level (m)	Date of measurement
HA1	203.5	Not encountered	-	15/02/2022
HA2	203.2	Not encountered	-	15/02/2022
HA3	202.9	Not encountered	-	15/02/2022
BH1	203.5	195.5	8	15/02/2022
CPT01	203.5	195.2	8.3	15/02/2022
CPT02	202.9	194.8	8.1	15/02/2022

2.4 Geotechnical model

The geological profile presented in this report and in associated appendices is based upon information obtained from the recently completed hand-auger boreholes, machine-drilled boreholes and Cone Penetrometer Tests. The nature and continuity of the subsoil profile away from these locations is inferred but it must be appreciated that actual conditions may vary from the assumed model.

A summary of the generalised geological profile is presented in Table 2.4 and an interpretive geological cross section is presented in Figure *XX*, Appendix *XX*. The geological cross section of the site was developed using site survey, geological information, and measured groundwater levels.

Table 2.3: Generalised geological profile

<table>
<tr><th>Geological Unit</th><th>Soil Description</th><th>Typical depth to base of layer (m)</th><th>Typical layer thickness (m)</th><th>Scala Penetrometer (blows/100 mm)</th><th>Undrained shear strength (kPa)</th><th>Cone tip resistance (MPa)</th></tr>
<tr><td>Topsoil</td><td>Organic silty TOPSOIL</td><td>0.3 – 0.5</td><td>0.3 – 0.5</td><td>0 – 1</td><td>32 – 50</td><td>>2</td></tr>
<tr><td>Undifferentiated Fill material</td><td>Sandy SILT with some organics</td><td>0.5 – 0.7</td><td>0.2 – 0.4</td><td>1 – 2</td><td>28 – 45</td><td>>2</td></tr>
<tr><td>Taranaki Brown Ash</td><td>Stiff to very stiff SILT with some clay</td><td>6 – 7</td><td>5.5 – 6.3</td><td>2 – 4 (to 3.0 m depth)</td><td>75 – 140+</td><td>>5</td></tr>
<tr><td rowspan="2">Holocene Lahar Deposits</td><td>Medium dense to dense gravelly SAND</td><td>10 - 12</td><td>4.0 – 5.0</td><td>-</td><td>-</td><td rowspan="2">10 - 35</td></tr>
<tr><td>Alternating dense gravel, cobble and SILT layers with minor peat/organic layers</td><td>20</td><td>>8</td><td>-</td><td>-</td></tr>
</table>

3 Geotechnical considerations

3.1 General

Recommendations and opinions contained in this report are based on our visual appraisal of the site, as well as the assessed geotechnical investigations and our experience and knowledge of the surrounding area. The nature and continuity of the soil conditions away from the test locations is inferred but it must be appreciated tat actual conditions may vary from the assumed model.

Based on the field investigations and engineering analyses, the site is generally considered suitable for the proposed development, subject to consideration of the following geotechnical design considerations:

1. Liquefaction;
2. Consolidation settlement;
3. Expansive soils;
4. Earthworks;
5. Retention structures; and
6. Foundation options and design parameters;

It should be noted that there may be other factors that need to be considered in foundation design that are outside of this geotechnical scope of works (i.e., flood hazard zones, minimum floor levels etc).

3.2 Liquefaction

This section of the report should present the findings of a liquefaction vulnerability assessment based on the assessed geotechnical investigations and the consultants experience and knowledge of the surrounding area.

It should generally outline the methodology and software used and present the results in terms of a range of liquefaction vulnerability parameters (e.g., post-liquefaction one-dimensional volumetric consolidation (S_{V1D}), lateral spread displacements and Liquefaction Severity Number (LSN)). At a minimum, these parameters should be calculated for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) earthquake events in accordance with the requirements of the Building Act.

Module 3 of MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c) *also recommends the consideration of intermediate earthquake events to identify the point of "step change" in liquefaction performance. This approach is known as the "holistic evaluation of performance." Seismic hazard design parameters for New Plymouth recommended for use in geotechnical design are provided in Appendix A of Module 1 MBIE/NZGS Earthquake geotechnical engineering practice series* (MBIE/NZGS, 2021a).

The results of these assessments are typically displayed in a table and can be used to inform both the assessment of liquefaction vulnerability at the site in accordance with the MBIE/MfE Guidance (2017) and the adoption of a suitable foundation design.

3.2.1 Method of liquefaction assessment

Liquefaction assessment of the CPT investigations has been carried out in accordance with Module 3 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c). Liquefaction potential and the calculation of post liquefaction induced settlements have been assessed using the methodology developed by Idriss & Boulanger (2014) and Zhang et al (2002) respectively. The method of Robertson and Wride (1998) has been applied to account for the effect of grain characteristics or fine content.

3.2.2 Earthquake return periods

The New Zealand design loads code NZS 1170 defines to design conditions which need to be assessed for the purpose of liquefaction assessments.

- **ULS – Ultimate Limit State** is concerned with ground damage associated with a 500-year earthquake event, for which buildings should be designed to avoid collapse and potential loss of life.
- **SLS – Serviceability Limit State** is concerned with ground damage associated with smaller earthquakes with a return period of 25 years. Buildings and their non-structural components should be designed to withstand permanent damage for a 25-year event i.e. readily repairable.

Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) recommends that intermediate seismic events also be considered alongside the SLS and ULS requirements to determine if a significant step change in ground performance is present between the SLS and ULS design events. Following review of the seismic hazard information for New Plymouth we have selected the 1 in 100-year and 1 in 250-year return periods as intermediate events.

The 1 in 100-year and 1 in 500-year events are also recommended in the MBIE/MfE Guidelines[6] as the minimum return periods for assessment of liquefaction vulnerability.

3.2.3 Ground motion inputs

The ground motion inputs for analysis were obtained from the Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) for SLS, ULS and the selected intermediate events. Table 3.1 outlines the ground motion inputs.

Table 3.1: Ground motion inputs obtained from Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series.

Earthquake event	PGA Value (g)	Earthquake Magnitude
1 in 25 year (SLS)	0.07	6
1 in 100 year (Intermediate 1)	0.14	6
1 in 250 year (Intermediate 2)	0.21	6
1 in 500 year (ULS)	0.29	6

3.2.4 Ground water assessment

Geotechnical investigations encountered the water table between 8.0 m – 8.3 m below the current ground level. A shallower groundwater depth of 7.5 m below the current ground level was used for the liquefaction calculations presented below. This shallower design groundwater depth was chosen based on review of the potential for elevated groundwater levels associated with seasonal variation. This level was chosen based on review of the variability in groundwater conditions in geotechnical investigations undertaken on nearby sites.

Increases or decreases in rainfall as a result of climate change may also affect groundwater levels in future, however the magnitude of any change is difficult to predict. To explore this further sensitivity analysis to understand the effect of groundwater assumptions on liquefaction vulnerability has been undertaken.

3.2.5 Liquefaction vulnerability assessment

Using the liquefaction assessment method discussed in Section 3.3.1 above, combined with a design ground water level of 7.5 m, only thin layers (<50 mm) of liquefaction are predicted. These thin liquefiable layers predominantly occur in the medium dense to dense gravelly sands that are encountered from approximately 6 – 12 m depth across the site. Table 3.2 summarises the liquefaction vulnerability parameters (S_{V1D} and LSN) for the assumed ground motion parameters.

Table 3.2: S_{V1D} and LSN values at CPT01 and CPT02 for selected ground motions

Earthquake event	**S_{V1D}**		**LSN**	
	CPT01	**CPT02**	**CPT01**	**CPT02**
1 in 25 year (SLS)	0	0	0	0
1 in 100 year (Intermediate 1)	0	5	0	1
1 in 250 year (Intermediate 2)	11	19	0	3
1 in 500 year (ULS)	24	28	1	4

*Based on groundwater level at 7.5 m below ground level.

Groundwater sensitivity analysis indicated that there was only minor change to the calculated liquefaction vulnerability parameters. This is because liquefiable layers in the soil profiles are located at depths greater than 6 m.

There are no free-faces greater than 1 m in height within 500 m of the site and therefore lateral spreading is unlikely to cause consequential liquefaction related land damage at the site.

Based on the values provided in Table 3.2 and Table 2.2 of the MBIE/MfE Guidance (2017) "*There is a probability of more than 85 percent that liquefaction induced ground damage will be none to minor (or less) for 500-year shaking*" and as a result, we have categorised the site as having a Low liquefaction vulnerability.

This liquefaction vulnerability assessment has been undertaken to a Level D – Level of detail (Site-specific assessment) in accordance with the MBIE/MfE Guidance (2017).

3.3 Consolidation settlement

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.4 Expansive soils

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.5 Slope stability

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.6 Earthworks

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.7 Retention structures

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.8 Contamination

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.9 Stormwater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.10 Wastewater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.11 Foundation recommendations

Comment: Foundation recommendations should then be given based on the information in Section 3.2 to Section 3.7 above. If this geotechnical report is being prepared for a subdivision resource consent, site specific recommendations may be given for further testing at individual sites.

Based on the liquefaction vulnerability analysis undertaken in this report and the assessed liquefaction vulnerability category of Low, the proposed shallow concrete pads and concrete floor slab do not need to be designed to tolerate ground deformations associated with liquefaction induced land damage.

4 Statement of professional opinion on suitability of land

Refer to Schedule 2A of NPDC "Land Development and Subdivision Infrastructure Standard"

5 References

MBIE/MfE, 2017. *Planning and engineering guidance for potentially liquefaction-prone land,* Wellington: Ministry of Business, Innovation and Employment & Ministry for the Environment.

MBIE/NZGS, 2021a. *Earthquake geotechnical engineering practice - Module 1: Overview of the guidelines,* Wellington: s.n.

MBIE/NZGS, 2021b. *Earthquake geotechnical engineering practice - Module 2: Geotechnical investigations for earthquake engineering,* Wellington: s.n.

MBIE/NZGS, 2021c. *Earthquake geotechnical engineering practice - Module 3: Identification, assessment and mitigation of liquefaction hazards,* Wellington: s.n.

NZGS, 2005. *Field description of soil and rock, guideline for the field classification and description of soil and rock for engineering purposes,* New Zealand: New Zealand Geotechnical Society.

Tonkin & Taylor Ltd, 2021. *New Plymouth District Liquefaction Vulnerability Assessment. Report reference 1016765.v1,* Tauranga, New Zealand: Tonkin & Taylor Ltd.

Townsend, D., Vonk, A. & Kamp, P. J. J., 2008. *Geology of the Taranaki area. 1:250 000 geological map 7,* Lower Hutt, New Zealand: Institute of Geological & Nuclear Sciences.

Case Study 3 – Scenario 3 (Waitara)

Liquefaction Assessment Summary Report Template

Version 1 – 20 June 2022

Overview

The following tables provide a template for an Assessment Summary Report for liquefaction vulnerability assessments. This report template draws on information provided in the New Plymouth District Liquefaction Vulnerability Report (Tonkin & Taylor, 2021), the Options for Liquefaction Assessment for Resource and Building Consent Report (Tonkin & Taylor, 2022) and relevant national guidance.

This template contains a summary of the following information:

- **Development information**
  - Proposed development, development scenario, location of development, existing liquefaction category, geomorphic terrain.
- **Geotechnical investigation information**
  - Number and depth of hand augers, Scala Penetrometer tests, shear vane tests, Cone Penetrometer tests, machine drilled boreholes, additional geotechnical investigations and testing.
- **Geotechnical parameters based on investigation data**
  - Description of ground conditions, groundwater conditions, seismic hazard (design parameters).
- **Assessment information**
  - Consulting practice, Chartered Engineer, Date of site assessment, Existing or previous geotechnical reports.
- **Summary of engineering assessment methodology and key parameters used**
  - Importance level of proposed development, method of liquefaction assessment, revised liquefaction vulnerability category, level of detail achieved, NZ standards or guidance documents referenced.

1. Development information	
Proposed Development	We understand that the client wishes to construct a three bedroom, 180 m^2 dwelling. Concept drawings provided by the client show the dwelling being founded on a concrete slab foundation system and having brick cladding and a tile roof. The dwelling is considered to be a heavy weight structure designed to NZS 3604:2011.
Development scenario	Development Scenario 3 from NPDC liquefaction guidance flowchart: Small-scale urban infill
Location of development	Waitara *(Site specific details would be required here)*
Liquefaction vulnerability category assigned in NPDC Liquefaction Vulnerability Report (2021)	Liquefaction Damage is Possible
Geomorphic terrain assigned in NPDC Liquefaction Vulnerability Report (2021)	Alluvial plains and river flats

2. Geotechnical investigation information	
Number and depth of hand augers (HA)	- 4 no. hand auger (HA01 to HA04) boreholes to a target depth of 3 m below the current ground level.
Number and depth of Scala Penetrometer Tests or shear vane tests (SP or SV)	- Scala Penetrometer testing (SP) within the hand augered boreholes to a target depth of 3 m below the current ground level.
Number and depth of Cone Penetrometer Tests (CPT)	- 2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 15 m below the current ground level.
Number and depth of machine drilled boreholes (BH)	*N/A*
Additional geotechnical investigations and testing	*N/A*

3. Geotechnical parameters based on investigation data	
Description of ground conditions	Liquefiable sand and silt sediments present to at least 10 m below ground level
Groundwater conditions (accounting for climate change and seasonal variations)	The deepest hand auger undertaken during the investigation (HA4) extended to a depth of 3.0 m below the current ground level. The sediments logged throughout this depth were described as being dry to wet with the water table being encountered at between 1.9 – 2.0 m. A shallower groundwater depth of 1.5 m below the current ground level was used for the liquefaction calculations presented below. This shallower design groundwater depth is based on the potential for elevated groundwater levels associated with seasonal variation.
Seismic hazard (design parameters)	1 in 25 year (SLS): 0.07 g 1 in 100 year (Intermediate 1): 0.14 g 1 in 250 year (Intermediate 2): 0.21 g 1 in 500 year (ULS): 0.29 g

4. Assessment information	
Consulting practice	*Enter details here*
Chartered Engineer: - Name - CPEngNZ Registration Number -Practice Area	*Enter details here*
Date of site assessment	31/05/2022
Existing or previous geotechnical reports associated with development	*N/A*

5. Summary of engineering assessment methodology and key parameters used	
Importance level of proposed development	*Importance Level 2*
Method of liquefaction assessment (if applicable)	*Option 2* from NPDC liquefaction guidance flowchart: Site-specific geotechnical engineering assessment and use of MBIE Canterbury Guidance (2018)
Confirmed geomorphic terrain	Alluvial plains and river flats. No change from terrain mapped in NPDC Liquefaction Vulnerability Report (2021). Ground conditions encountered on site were consistent with the mapped geology and geomorphology (i.e., predominantly silts and sands).
Groundwater level assumed for design	1.5 m below the current ground level
Confirmed liquefaction vulnerability category (as per MBIE/MfE 2017)	High Liquefaction Vulnerability
Level of detail achieved	Level D level of detail (Site-specific assessment)
NZ Standards or Guidance documents referenced	MBIE/MfE, 2017. *Planning and engineering guidance for potentially liquefaction-prone land,* Wellington: Ministry of Business, Innovation and Employment & Ministry for the Environment. MBIE/NZGS, 2021a. *Earthquake geotechnical engineering practice - Module 1: Overview of the guidelines,* Wellington: s.n. MBIE/NZGS, 2021c. *Earthquake geotechnical engineering practice - Module 3: Identification, assessment and mitigation of liquefaction hazards,* Wellington: s.n. MBIE, 2018. *Repairing and rebuilding houses affected by the Canterbury earthquakes. Version 3,* Wellington, New Zealand: Ministry of Business, Innovation and Employment.

Case Study – Scenario 3 (Waitara)

1 Introduction

Note: Blue italicised text throughout this report represents general notes regarding the content that should be addressed by each heading/subheading.

1.1 General

General overall text about what report is intended to be used for (Resource consent, plan change, building consent etc).

This geotechnical investigation and interpretative report has been prepared to support a building consent application for a proposed dwelling at the site.

1.2 Objectives of work

Outline objectives of work i.e. to determine if "good ground" present, to determine if land is suitable for subdivision, meets requirements of RMA etc.

1.3 Site description

The site of interest (here in known as "the site") is a residential property located within the Waitara Township with an area of approximately 880 m^2. The site is generally flat and is positioned approximately 4 m above sea level and approximately 200 m away from the Waitara River. At the edge of the river there is a steep bank that is a 1 – 2 m high free-face.

Report should provide figure to show the location of the site.

1.4 Proposed development

We understand that the client wishes to construct a three bedroom, 180 m^2 dwelling on the site as shown on the attached site plan in Appendix *XX*. Concept drawings provided by the client show the dwelling being founded on a concrete slab foundation system and having brick cladding and a tile roof. The dwelling is considered to be a heavy weight structure designed to NZS 3604:2011.

Report should include relevant conceptual or detailed drawings of the proposed development if available.

2 Assessment and interpretation of site conditions

2.1 Geology and faulting

A geological map published by GNS (Townsend, et al., 2008) indicates that the site is underlain by Holocene river deposits comprising alluvial gravel, sand, silt, mud and clay with local peat. Geomorphology mapping undertaken by T+T (Tonkin & Taylor Ltd, 2021) shows that the site is located in the area mapped as alluvial plains and river flats. This terrain typically comprises sediments deposited by active and historic river systems. The nearest mapped active fault to the site is the Inglewood Fault which lies approximately 20 km to the southeast of the site.

2.2 Known natural hazards

The liquefaction vulnerability category of the site was assessed by T+T as "***Liquefaction damage is possible***" (Tonkin & Taylor Ltd, 2021). According to the MBIE/MfE Guidance (2017), this means that *"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". This previous liquefaction vulnerability assessment was undertaken to a Level A – level of detail in accordance with the MBIE/MfE Guidance (2017).* This report includes review of that liquefaction vulnerability assessment utilising the new site-specific geotechnical investigations undertaken.

Generally based on natural hazard information sourced from Council GIS platforms and/or site-specific assessments undertaken by other suitably qualified professionals. All sources should be referenced appropriately. Other typical hazards considered may include:

- *Coastal*
- *Flooding*
- *Wind*
- *Land stability*
- *Soil maps*

2.3 Geotechnical investigations

2.3.1 Desktop assessment

Prior to scoping geotechnical site investigations, a desktop assessment was undertaken for the site. This assessment reviewed existing geotechnical investigations on nearby sites to understand potential ground and groundwater conditions, and to inform geotechnical investigation requirements.

Geotechnical investigations on nearby sites indicated ground conditions that were consistent with the mapped geology and geomorphology (i.e., predominantly silts and sands). Static groundwater levels varied from approximately 2 – 4 m RL (NZVD 2016) on sites at similar elevations.

2.3.2 Scope of geotechnical investigations

The scope of site-specific ground investigations was developed to target key geotechnical matters identified in the desktop assessment. Geotechnical investigations on the site comprised the following:

- 4 no. hand auger (HA01 to HA04) boreholes to a target depth of 3 m below the current ground level.
- Scala Penetrometer testing (SP) within the hand augered boreholes to a target depth of 3 m below the current ground level.
- 2 no. Cone Penetrometer Tests (CPT01 and CPT02) to a target depth of 15 m below the current ground level.

The location of these geotechnical investigations is shown on the attached site plan in Appendix *XX* and the investigation results are attached in Appendix *XX*.

2.3.3 Hand auger boreholes

The drilling of the 4 no. hand auger boreholes, (HA1 to HA4) was undertaken on 14 February 2022. These were carried out by an experienced Engineering Geologist, who logged the boreholes to the New Zealand Geotechnical Society logging guidelines (NZGS, 2005).

Investigation locations are presented on Figure *XX*, Appendix *XX,* hand auger borehole logs are presented in Appendix *XX*, and Table 2.1 provides a summary of the hand auger locations and depths.

Table 2.1: Hand auger borehole summary

HA ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)
HA1	*XXXXX*	4.2	3.0
HA2	*XXXXX*	4.2	3.0
HA3	*XXXXX*	4.1	3.0
HA4	*XXXXX*	4.0	3.0

2.3.4 Scala Penetrometer tests

The Scala Penetrometer/shear vane test locations are presented on Figure *XX*, Appendix *XX,* and Scala Penetrometer/shear vane test results are presented on the hand auger borehole logs in in Appendix *XX*. Table 2.2 provides a summary of the Scala Penetrometer locations and depths.

Table 2.2: Scala Penetrometer Test Summary

Scala Penetrometer/Shear Vane test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
SP1	*XXXXX*	4.2	3.0	Target depth
SP2	*XXXXX*	4.2	3.0	Target depth
SP3	*XXXXX*	4.1	3.0	Target depth
SP4	*XXXXX*	4.0	3.0	Target depth

2.3.5 Cone Penetrometer Tests

The Cone Penetrometer Test (CPT) locations are presented on Figure *XX*, Appendix *XX*, and Table 2.3 provides a summary of the CPT locations and depths.

Table 2.3: Cone Penetrometer Test Summary

Cone Penetrometer Test ID	Coordinates	Ground Surface Elevation (NZVD 2016)	Depth (m)	Reason for termination
CPT01	*XXXXX*	4.2	15.0	Target depth
CPT02	*XXXXX*	4.0	15.0	Target depth

2.3.6 Groundwater

Groundwater levels within the investigations were recorded using an electronic dip meter on completion of drilling. The recorded groundwater levels are presented below in Table 2.3, with ground surface elevations obtained from the New Plymouth District Council GIS platform.

Table 2.3: Groundwater levels

HA ID	Ground Surface Elevation (NZVD 2016)	Static Groundwater Level RL (NZVD 2016)	Depth Below Ground Level (m)	Date of measurement
HA1	4.2	2.0	2.2	15/02/2022
HA2	4.2	1.9	2.3	15/02/2022
HA3	4.1	1.9	2.2	15/02/2022
HA4	4.0	2.0	2.0	15/02/2022

2.4 Geotechnical model

The geological profile presented in this report and in associated appendices is based upon information obtained from the recently completed hand-auger boreholes and Cone Penetrometer Tests. The nature and continuity of the subsoil profile away from these locations is inferred but it must be appreciated that actual conditions may vary from the assumed model.

A summary of the generalised geotechnical profile is presented in Table 2.4 and an interpretive geological cross section is presented in Figure *XX*, Appendix *XX*. The geological cross section of the site was developed using site survey, geological information, and measured groundwater levels.

Table 2.3: Generalised geological profile

Geological Unit	Soil Description	Typical depth to base of layer (m)	Typical layer thickness (m)	Scala Penetrometer (blows/100 mm)	Cone tip resistance (MPa)
Topsoil	Organic silty TOPSOIL	0.1 – 0.2	0.1 – 0.2	0 – 2	>2
Holocene river deposits	Loose to medium dense silty SAND	3.0	2.8 – 2.9	2 – 4	>2
	Loose to medium dense silty SAND	15.0	15.0	-	5 - 10

3 Geotechnical considerations

3.1 General

Recommendations and opinions contained in this report are based on our visual appraisal of the site, as well as the assessed geotechnical investigations and our experience and knowledge of the surrounding area. The nature and continuity of the soil conditions away from the test locations is inferred but it must be appreciated tat actual conditions may vary from the assumed model.

Based on the field investigations and engineering analyses, the site is generally considered suitable for the proposed development, subject to consideration of the following geotechnical design considerations:

1 Liquefaction;

2 Consolidation settlement;

3 Expansive soils;

4 Earthworks;

5 Retention structures; and

6 Foundation options and design parameters;

It should be noted that there may be other factors that need to be considered in foundation design that are outside of this geotechnical scope of works (i.e., flood hazard zones, minimum floor levels etc).

3.2 Liquefaction

This section of the report should present the findings of a liquefaction vulnerability assessment based on the assessed geotechnical investigations and the consultants experience and knowledge of the surrounding area.

It should generally outline the methodology and software used and present the results in terms of a range of liquefaction vulnerability parameters (e.g., post-liquefaction one-dimensional volumetric consolidation (S_{V1D}), lateral spread displacements and Liquefaction Severity Number (LSN)). At a minimum, these parameters should be calculated for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) earthquake events in accordance with the requirements of the Building Act.

Module 3 of MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c) *also recommends the consideration of intermediate earthquake events to identify the point of "step change" in liquefaction performance. This approach is known as the "holistic evaluation of performance." Seismic hazard design parameters for New Plymouth recommended for use in geotechnical design are provided in Appendix A of Module 1 MBIE/NZGS Earthquake geotechnical engineering practice series* (MBIE/NZGS, 2021a).

The results of these assessments are typically displayed in a table and can be used to inform both the assessment of liquefaction vulnerability at the site in accordance with the MBIE/MfE Guidance (2017) and the adoption of a suitable foundation design.

3.2.1 Method of liquefaction assessment

Liquefaction assessment of the CPT investigations has been carried out in accordance with Module 3 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021c). Liquefaction potential and the calculation of post liquefaction induced settlements and lateral spread have been assessed using the methodology developed by Idriss & Boulanger (2014) and Zhang et al (2002) respectively. The method of Robertson and Wride (1998) has been applied to account for the effect of grain characteristics or fine content.

3.2.2 Earthquake return periods

The New Zealand design loads code NZS 1170 defines to design conditions which need to be assessed for the purpose of liquefaction assessments.

- **ULS – Ultimate Limit State** is concerned with ground damage associated with a 500-year earthquake event, for which buildings should be designed to avoid collapse and potential loss of life.
- **SLS – Serviceability Limit State** is concerned with ground damage associated with smaller earthquakes with a return period of 25 years. Buildings and their non-structural components should be designed to withstand permanent damage for a 25-year event i.e. readily repairable.

Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) recommends that intermediate seismic events also be considered alongside the SLS and ULS requirements to determine if a significant step change in ground performance is present between the SLS and ULS design events. Following review of the seismic hazard information for New Plymouth we have selected the 1 in 100-year and 1 in 250-year return periods as intermediate events.

The 1 in 100-year and 1 in 500-year events are also recommended in the MBIE/MfE Guidance (2017) as the minimum return periods for assessment of liquefaction vulnerability.

3.2.3 Ground motion inputs

The ground motion inputs for analysis were obtained from Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series (MBIE/NZGS, 2021a) for SLS, ULS and the selected intermediate events. Table 3.1 outlines the ground motion inputs.

Table 3.1: Ground motion inputs obtained from Module 1 of the MBIE/NZGS Earthquake geotechnical engineering practice series.

Earthquake event	PGA Value (g)	Earthquake Magnitude
1 in 25 year (SLS)	0.07	6
1 in 100 year (Intermediate 1)	0.14	6
1 in 250 year (Intermediate 2)	0.21	6
1 in 500 year (ULS)	0.29	6

3.2.4 Ground water assessment

The deepest hand auger undertaken during the investigation (HA4) extended to a depth of 3.0 m below the current ground level. The sediments logged throughout this depth were described as being dry to wet with the water table being encountered at between 1.9 – 2.0 m. A shallower groundwater depth of 1.5 m below the current ground level was used for the liquefaction calculations presented below. This shallower design groundwater depth is based on the potential for elevated groundwater levels associated with seasonal variation.

Given the relatively close proximity of the site to the Waitara River which is tidally influenced in this location, there is also the potential for sea level rise associated with climate change to further elevate long term groundwater levels. Therefore sensitivity analysis to understand the effect of groundwater assumptions on liquefaction vulnerability has also been undertaken.

3.2.5 Liquefaction vulnerability assessment

Using the liquefaction assessment method discussed in Section 3.2.1 above, combined with a design ground water level of 1.5 m, there is the potential for liquefaction to occur within the upper 10 m of the soil profile. Table 3.2 summarises liquefaction vulnerability parameters (S_{V1D} and LSN) for the assumed ground motion parameters.

Table 3.2: S_{V1D} and LSN values at CPT01 and CPT02 for selected ground motions

Earthquake event	S_{V1D}		LSN	
	CPT01	CPT02	CPT01	CPT02
1 in 25 year (SLS)	0	0	0	0
1 in 100 year (Intermediate 1)	24	21	14	11
1 in 250 year (Intermediate 2)	55	46	19	14
1 in 500 year (ULS)	110	85	25	21

*Based on groundwater level at 1.5 m below ground level.

Due to a free-face with a height approximately 2 m (river banks associated with Waitara River) being located approximately 200 m from the site, there is the potential for lateral spreading to occur as a result of an earthquake event. Table 3.3 summarises Lateral Displacement Index (LDI) values calculated using the empirical method described by Zhang et al (2004).

Table 3.3: LDI values at CPT01 and CPT02 for selected ground motions

Earthquake event	LDI (mm)	
	CPT01	CPT02
1 in 25 year (SLS)	0	0
1 in 100 year (Intermediate 1)	5	7
1 in 250 year (Intermediate 2)	25	22
1 in 500 year (ULS)	85	60

Groundwater sensitivity analysis undertaken at the site indicated that the calculated liquefaction vulnerability parameters were sensitive to changes in groundwater level. This sensitivity has been allowed for in the specification of the foundation options adopted.

Interpretation of the liquefaction vulnerability parameters provided is as follows:

- At 1 in 25 year (SLS) levels of earthquake shaking liquefaction related damage is likely to be no more than None to Minor
- At 1 in 100 year levels of earthquake shaking liquefaction related damage is likely to be no Minor to Moderate
- At 1 in 250 year levels of earthquake shaking liquefaction related damage could range from Minor to Moderate to Moderate to Severe
- At 1 in 500 year levels of earthquake shaking liquefaction related damage is likely to be Moderate to Severe

Based on the values provided in Table 3.2, Table 3.3 and Table 2.2 of the MBIE/MfE Guidance (2017) *"There is a probability of more than 50 percent that liquefaction induced ground damage will be Moderate to Severe for 500-year shaking; and Minor to Moderate for 100-year shaking"* and as a result, we have categorised the site as having a High Liquefaction Vulnerability.

This liquefaction vulnerability assessment has been undertaken to a Level D – Level of detail (Site-specific assessment) in accordance with the MBIE/MfE Guidance (2017).

3.3 Consolidation settlement

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.4 Expansive soils

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.5 Slope stability

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.6 Earthworks

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.7 Retention structures

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.8 Contamination

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.9 Stormwater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.10 Wastewater Management

Comment: Topic should be addressed in geotechnical reports accompanying resource consent and building consent applications.

3.11 Foundation recommendations

Comment: Foundation recommendations should then be given based on the information in Section 3.2 to Section 3.7 above. If this geotechnical report is being prepared for a subdivision resource consent, site specific recommendations may be given for further testing at individual sites.

Due to the site being categorised as having a High Liquefaction vulnerability (refer to Section 3.2.5), is it considered to be "prone to liquefaction or lateral spreading" and therefore **does not meet the definition of "Good Ground"** as outlined in the Building Code amendments.

Geotechnical investigations undertaken on the site show that a geotechnical ultimate bearing capacity of 200 kPa is generally available across the site at a depth of 0.2 m below ground level.

Based on the above results, we have categorised the site as having a High Liquefaction Vulnerability. Recent information uploaded to the MBIE Building Performance website (MBIE, 2022a) recommends TC3 type foundation options for a High Liquefaction vulnerability. Furthermore, Table 3.1 of the MBIE Canterbury Guidance (2018) "Repairing and rebuilding houses affected by the Canterbury earthquakes" also indicates that the calculated S_{V1D} values for both the SLS and ULS seismic event fall within the TC3 Foundation Technical Category.

Table 15.1 and Table 15.4 of the MBIE Canterbury Guidance (2018) shows that a stiffened raft slab foundation type (TC2) founded on a 1.2 m thick reinforced crushed gravel raft would be suitable for.

There are several different types of TC3 foundation options which are dependent on the geotechnical ultimate bearing capacity available on the site and the calculated liquefaction vulnerability parameters. Please see the MBIE "Repairing and rebuilding houses affected by the Canterbury earthquake" for further information.

Guidance on these foundations can be found in the MBIE guidance document "Repairing and rebuilding houses effected by the Canterbury earthquakes" (2018).

4 Statement of professional opinion on suitability of land

Refer to Schedule 2A of NPDC "Land Development and Subdivision Infrastructure Standard"

5 References

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