GEOTECHNICAL ASSESSMENT

PART 1 OF 5

CGW
CONSULTING ENGINEERS

Geotechnical Assessment Report for Subdivision

Waikanae North Subdivision

Waikanae North Developments Ltd C/-Paul Turner

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Contents

1. Executive Summary

CGW Consulting Engineers has undertaken a comprehensive geotechnical assessment of the Waikanae North subdivision to support a resource consent application under the Fast-track Approvals Act 2024. The approximately 140-hectare site is in Peka Peka, north of Waikanae, and is proposed for mixed residential development, supporting infrastructure, and open space.

Site Geology and Ground Conditions

The site has been categorised into three geomorphic environments based on landform and subsurface conditions:

- Low-lying areas: Located in the eastern and southern portions of the site, underlain by soft peat (up to 2.6 m thick), silty sand, and marine deposits. Groundwater is typically shallow (<1.5 m BGL).
- Lower Dunes: Undulating terrain comprising loose to medium-dense sand, often moist, with no observed peat. Groundwater generally ranges from 1.1 to 3.7 m BGL.
- Upper Dunes: Elevated dune ridgelines comprised of clean, well-drained sand that becomes dense below 1.5 m BGL with groundwater typically deeper than 3.0 m.

Liquefaction and Static Settlement

Liquefaction screening across 155 CPTs identified:

- SLS: Liquefaction-related settlement during SLS-level shaking is negligible across most of the site. However, at SLS2 levels of shaking, settlement increases across much of the site, with average settlements in the low-lying areas increasing to approximately 30 mm and localised values exceeding 50 mm but less than 100 mm, although these results are derived from CPTs positioned primarily within non-developed areas (proposed green space).
- ULS: Moderate to severe liquefaction-induced settlement in Low-lying areas and some Lower Dunes locations (up to 70 mm differential). The Upper Dunes are expected to perform well with negligible response to future liquefaction induced settlement.

Static settlement analysis using CPeT-IT confirms that the Low-lying areas, particularly area underlain by peat soils, are unsuitable for shallow foundations without recommended ground improvement work. Static settlements exceeding 100 mm are predicted under a typical residential load without ground improvement. In contrast, dune sandy soils exhibit minimal static settlement.

Fault Avoidance Zone (FAZ)

The southern portion of the site aligns with a regionally mapped Fault Avoidance Zone (Ōhāriu Fault – Class II, uncertain, poorly constrained) identified by Van Dissen & Heron (2003) but not shown in the Operative Kāpiti Coast District Plan (2021). No obvious subsurface fault disturbance was detected, and development within this area is considered suitable for residential structures using TC2-type foundations, subject to site-specific geotechnical input.

Slope Stability

Slope modelling across four representative profiles in the dune terrain indicated:

- Stable under static and SLS conditions.
- Localised shallow instability under extreme groundwater or ULS seismic conditions.
- Risks are confined to surficial sand layers and are mitigable through drainage control and slope-specific stabilisation.

MBIE Technical Categories and Earthworks

The site has been provisionally zoned as follows:

- TC1: Assigned to Upper Dunes — clean, dense sand with low risk of liquefaction or settlement.
- TC2: Assigned to Lower Dunes and most Low-lying areas — where minor to moderate liquefaction or compressible soils are present.
- Isolated TC3 zones: Confined to areas where no development is proposed.

Bulk earthworks will be required, including:

- Excavation and replacement of peat with compacted site-won sand or imported engineered fill.
- Drainage control, regrading, and verified fill placement under geotechnical supervision.
- Reuse of peat as non-engineered fill in areas such as landscaping or blending with sand to reduce organic content for potential reuse as engineered fill (the latter will be subject to testing and further geotechnical advice).

Foundation Design (Preliminary) and Subsoil Class

Foundations should align with the assessed technical categorisation:

- TC1 zones: SED, NZS 3604 foundations.
- TC2 zones: SED, TC2 Stiffened slabs.
- Design UBC: 200 kPa likely to be available within natural soil or within engineered fill below topsoil following earthworks.
- The site is preliminarily classified as Subsoil Class D (deep or soft soils) under NZS 1170.5.

Conclusion

The site is considered suitable for residential development from a geotechnical perspective, provided recommended earthworks, drainage measures, and appropriate foundation strategies are implemented. Lot-specific investigations and construction-phase geotechnical review will be required to confirm design assumptions for building consenting purposes.

2. Introduction

CGW Consulting Engineers (CGW) have been engaged by Waikanae North Developments Ltd C/- Paul Turner (the Client) to undertake a geotechnical investigation and assessment for the proposed Waikanae North Subdivision, located within the Kapiti Coast District Council (KCDC) jurisdiction.

The Client proposes to subdivide the site and develop it into residential allotments with associated infrastructure. This report presents a summary of geological and geotechnical conditions, identifies potential ground-related hazards, and provides preliminary recommendations to inform planning and development decisions.

The Waikanae North development is a listed project under the Fast-Track Approvals Act 2024 and is a significant development project prioritised for accelerated approval.

Where applicable, this assessment references the Ministry of Business, Innovation and Employment (MBIE) document Module 2: Geotechnical Investigations for Subdivision and Low-Rise Buildings – Earthquake Geotechnical Engineering Practice (Version 1, July 2021), with consideration given to the guidance around subdivision-scale investigation density, liquefaction hazard assessment, and geotechnical suitability reporting.

This report is intended to support a subdivision consent application made to the Environmental Protection Authority. Our geotechnical limitations are provided in Appendix A.

3. Proposed development

The Waikanae North project comprises the proposed development of approximately 140 hectares of land situated in Peka Peka, north of Waikanae on the Kāpiti Coast. The site is bounded by Peka Peka Road to the north and the Kapiti Expressway (State Highway 1) to the southeast. Paetawa Road and the Peka Peka Beach settlement lie to the west of the site.

The development is being progressed by Waikanae North Developments Ltd and is currently being processed under the Fast-track Approvals Act 2024.

The site is proposed to be developed into a mixed-use community incorporating:

- Residential allotments of varying densities,
- A commercial centre,
- Roading and infrastructure corridors,
- Significant open space and stormwater reserves, including wetland regeneration zones.

Development plans have been undertaken by Landlink Ltd and McIndoe Urban, and provided to CGW, with the current layout presented in the Illustrative Masterplan (dated 13 March 2026). The proposed subdivision layout is shown on Figure 1.

Given the scale of the site, it is expected that development will be carried out in stages, although the specific staging and sequencing are subject to further planning and design decisions.

This geotechnical investigation and assessment has been undertaken to assess the suitability of the land for subdivision, with particular focus on the extent of the underlying peat, slope stability hazard of elevated dune formations, and liquefaction hazard potential. The outcomes of the assessment will inform the planning and engineering design of the subdivision to support a Subdivision Consent application.

Figure 1. Proposed subdivision layout as shown in the Illustrative Masterplan (dated 13 March 2026). Plan prepared by McIndoe Urban.

4. Scope of Works

As outlined in Proposal 21512-PRO-001-A (dated 31 March 2025), the agreed scope of geotechnical engineering services includes:

- Desktop study of available geotechnical data, including liquefaction analysis and historical photography review.
- Site walkover assessment and mark out of testing locations.
- Undertaking 4 cross-section surveys to be used for slope stability analysis. Slope stability analysis will be confined to the dune areas.
- Install 11 additional piezometer standpipes to support the existing piezometer that were previously installed with ongoing monthly groundwater monitoring.
- Shallow geotechnical testing, including hand auger boreholes and Dynamic Cone Penetrometer (DCP) testing to a target depth of 5.0 m bgl.
- Deep geotechnical testing in line with the requirements of the MBIE module, including 157 Cone Penetration Tests (CPT).
- Slope stability analysis, liquefaction analysis and static settlement analysis.
- Provision of a geotechnical report suitable for subdivision consent. Reporting to include assessments and guidance on the following:
    - Commentary on the nearby GNS fault
    - Recommendations for batter slopes
    - Commentary on groundwater levels and piezometer monitoring
    - Recommendations regarding peat and removal
    - Slope stability analysis, liquefaction analysis and static settlement analysis

A Variation to SFA, dated 20 November 2025, involved investigating the property located at 109 Paetawa Road for inclusion in this assessment.

5. Site Information

5.1 Site Description

The site is in Peka Peka, on the northern edge of Waikanae, within the Kāpiti Coast District. It lies between Peka Peka Road to the north and the Peka Peka Link Road/State Highway 1 expressway to the southeast. The property is currently used as a mix of pastoral grazing and agriculture purposes.

The total landholding spans approximately 140 hectares and comprises the following parcels:

- Ngarara West A46A Block – 11,129 m²
- Section 1 Survey Office Plan 545508 and Section 15, 14 Survey Office Plan 505444 – 126,657 m²
- Lot 1 DP 589363 and Lot 2 DP 589363 – 229,526 m²
- Lot 2–3, 5 DP 587727 – 104,623 m²

- Part Kukutauaki 1B1 Block – 640,997 m^2
- Lot 1 DP 21514 – 274,850 m^2
- Lot 1 DP 82434 – 11,760 m^2

The land is proposed to be reconfigured into a mixed-use subdivision providing approximately 1,200 private residential dwellings across the site. This is primarily comprised of standard residential lots, with a smaller component of medium-density housing, terrace housing, apartment units, and some commercial areas.

The site comprises a combination of flat low-lying land, dune ridgelines, and undulating terrain, typical of the Waikanae coastal margin. The site is categorised into three distinct landform environments, as shown in Figure 2 – Geomorphic Zoning Plan.

- Low-lying Land Environment: Found primarily in the eastern and southern portions of the site, this area includes flat to gently undulating terrain with extensive near-surface peat and soft organic soils, interspersed with shallow historical dune remnants. These areas are poorly drained and exhibit high variability in subsurface conditions.
- Lower Dunes Environment: This zone comprises transitional terrain between the low-lying flats and the upper dune ridges. Terrain is undulating with moderate relief.
- Upper Dunes Environment: Occupying the western and central parts of the site, this zone consists of elevated sand dune ridgelines formed by windblown deposition.

Several rudimentary farm tracks and culverts traverse the site, with scattered vegetation and areas of wetland regeneration. A series of minor drains and channels are present, although no major natural watercourses were identified within the development area.

An aerial drone image (see Figure 3) illustrates the current condition of the site, including areas of pastoral use, shelterbelt planting, and visible topographic variation between low-lying flats and elevated dune ridges.

We understand that access to the proposed development will be via new internal roads and right of way accessways connecting to Peka Peka Road and other surrounding infrastructure, as shown in the Illustrative Masterplan (October 2025). High-voltage transmission lines cross the eastern portion of the site near the SH1 corridor, and easements will be required to manage associated setbacks and access restrictions.

Figure 2. Geomorphic Zoning Plan

Figure 3. Aerial drone image (view looking west) showing the approximate area of the Waikanae North development site. The photo highlights existing pastoral use, shelterbelts, low-lying peat areas, and dune ridgelines. Kāpiti Island is visible in the background.

6. Review of Previous Reporting

6.1 Previous CGW Investigations

CGW has undertaken multiple phases of geotechnical assessment across the Waikanae North site since 2021. These investigations have progressively improved the understanding of subsurface conditions, with each phase building on the findings of the last. Table 1 provides a summary of the key CGW reports and their outcomes, which have informed the scope, approach, and interpretation presented in this report.

Table 1: Summary of Previous CGW Geotechnical Investigations

Year	Report Reference	Scope Summary	Key Findings / Outcome
2021	21512-RPTGEO-001-A	Desktop review of aerials, geological maps, fault data, and borehole records.	Identified presence of peat and dune sands; proximity to Ohariu Fault; recommended detailed investigation.
2022	21512-RPT-G-002-A (Draft)	9 CPTs, 9 piezometers, 7 window samples (peat), 9 test pits (dunes), DCPs throughout.	Peat in lowlands; dense sands on dunes; variable groundwater; preliminary liquefaction assessment.
2024	21512-RPT-G-003-A	GPR survey, 7 hand augers, 7 DCPs, organic content lab testing.	Peat up to 3.0 m thick confirmed; recommended avoidance or engineering treatment in eastern lowlands.
2024	Data Provided to client for review	48 hand augers and 48 DCPs across peat zones to map lateral variability.	Confirmed extent and depth variability of peat.

7. Geological Model

7.1 Published Geology

According to GNS Science geological mapping (QMap – Wellington), the entire Waikanae North site is mapped as OIS1 (Holocene) stable dune deposits. These comprise inactive, wind-blown sand dunes formed during the Holocene epoch. Early Pleistocene swamp deposits have been mapped along the eastern boundary of the site and may extend westward into the project area, particularly where surface topography indicates paleo-wetland environments. These deposits are likely associated with ancient meander channels or backswamp areas within the broader fluvial system. This material is predominately made up of interbedded silt, sand and peat deposits, with the peat potentially occurring as discontinuous lenses within the deposit.

The dunes generally comprise fine to medium-grained sand and form a gently undulating landscape of subdued ridges and shallow swales. No older alluvial, marine, or swamp deposits are mapped at the surface. CGW investigations have confirmed the presence of compressible peat and organic soils within low-lying areas between dune ridges.

The site is likely to be underlain at significant depth by basement rock of the Torlesse Composite Terrane (Greywacke) of Triassic age. This basement material is likely to be at significant depth and not encountered within the subdivision investigations.

7.2 Active Faults

The Ōhāriu Fault is mapped through the southern portion of the Waikanae North site and is included in the GNS Science Active Faults Database, however this FAZ is not included in the Operative Kāpiti Coast District Plan (2021). According to the GNS report by Van Dissen & Heron (*GNS Science Report 2003/77*), the fault is classified as:

- Fault Name: Ōhāriu Fault
- Fault Complexity: Uncertain – poorly constrained
- Recurrence Interval Class: II (>2,000 to ≤3,500 years)
- Fault Type: Dextral (right-lateral) strike-slip

The fault location is considered uncertain due to a lack of obvious surface rupture features across parts of the coastal plain and is mapped conservatively based on geomorphic indicators and aerial interpretations.

A Fault Avoidance Zone (FAZ) has been defined across the southern portion of the site, as shown in Figure 4, to reflect the mapped position and uncertainty of the fault trace. The FAZ intersects a significant area of the proposed development and is based on regional mapping by GNS Science due to the lack of direct surface rupture evidence in this part of the coastal plain. This FAZ is not included in the Operative Kāpiti Coast District Plan (2021).

The fault trace within the Waikanae North site is interpreted to represent a secondary or branching segment of the main Ōhāriu Fault system, which runs northward through the Kāpiti foothills and the Transmission Gully corridor. This segment has not been trenched or directly confirmed on site. As such, its classification remains "uncertain – poorly constrained", and the FAZ has been conservatively applied based on limited geomorphic and regional alignment.

Any development within or adjacent to the FAZ must comply with the Ministry for the Environment's Active Fault Guidelines (2003), which include specific planning controls based on fault activity class, fault complexity, and the building importance category. A review of subsurface data is provided later in this report to assess whether the mapped fault trace is supported by geological evidence within the site.

Figure 4. Overlay of the mapped Ōhāriu Fault and Fault Avoidance Zone (FAZ) on the Waikanae North Illustrative Masterplan (Landlink Ltd October 2025).

7.3 NZGD Review

A review of the New Zealand Geotechnical Database (NZGD) found several relevant studies have been completed within proximity of the site. A summary of these investigations is presented below in Table 2 and locations on Figure 5. Selected NZGD borehole logs and summary data have been included in Appendix B for reference.

Table 2: Nearby Ground Investigation Data (NZGD)			
Hole ID	**Location relative to the site**	**Depth of Hole (m bgl)**	**Comments**
GWRC BP32_0034 (2013)	Within site, north of farm sheds (central area)	32.45	Water bore. Sand to 23.6 m bgl; interbedded sand, silt, and gravel below.
CPT-159595 (2021)	153 Peka Peka Rd, ~50 m NE of site	6.26	GW at 5.0 m bgl. Edge of wetland. Fine sand to 1.5 m bgl in HA logs.
GWRC R26/7080 (1999)	151 Peka Peka Rd, ~250 m NE of site	7.00	Peat at 4.5–5.0 m bgl within fine sand profile.
M2PP BH308 (2011)	Eastern boundary of site	10.63	2.5 m silty PEAT over fine sand. Dense sand from 7.5 m bgl.
M2PP BH347 (2013)	Eastern boundary, M2PP alignment	19.95	Sand to 12.3 m, gravel 12.3–16.5 m. Dense sand from 10.5 m.
M2PP BH217 (2010)	Southeastern boundary, M2PP alignment	10.75	PEAT to 2.0 m, over fine sand. Dense below 4.5 m.
M2PP BH13 (2010)	Southeastern boundary, M2PP alignment	15.45	Like BH217: PEAT to 2.0 m, dense sand below 4.5 m.
GWRC R26/6207 (2003)	362 Ngarara Rd, ~350 m SW of site boundary	32.20	Interbedded PEAT/SAND to 20.5 m. SAND/GRAVEL to 25.5 m. Clay at depth.

Figure 5. NZGD data points within proximity of the site.

These nearby investigations confirm the geological variability described in Section 7.1, including widespread Holocene dune sand underlain by peat layers in low-lying areas and dense sand or gravel at depth. Peat was observed in multiple boreholes on the eastern and southern boundaries of the site, generally ranging from 1.5 to 2.5 m thick.

7.4 Historic Aerial Photography Review

Historic aerial photographs were reviewed using Retrolens, Kāpiti Coast District Council (KCDC) mapping, and Google Earth covering the period from 1948 to 2017. The following changes to the landscape and land use were observed over time:

- 1948–1957: The site was largely undeveloped and used for pastoral farming. The original State Highway 1 (SH1) was already in place along the western boundary, with scattered rural dwellings. Wetlands were present along the northern boundary and northwest corner of the site.
- 1966–1977: Peka Peka Road appeared on the northeastern edge by 1966. The northwestern wetland remained prominent, although other wetland areas appear to have reduced. By 1977, farm tracks and drainage features became evident throughout the property, indicating increased agricultural activity.
- 1983–1987: A significant pine forest plantation had been established along the northern and western boundaries by 1987, especially near the internal access track. Smaller pine clusters were also established on the western dune ridges.
- 1998–2010: By 1998, residential development was expanding east of the site. The pine forests on the northern boundary were progressively removed by 2010. Farm infrastructure remained visible across the site.
- 2015–2017: The new SH1 expressway (Kāpiti Expressway) was under construction by 2015 along the southeastern site boundary. By 2017, the expressway was operational and residential expansion continued around the periphery of the site.

These observations indicate that while parts of the site have been modified through forestry and drainage for farming, much of the central and eastern land remains in a pastoral or undeveloped state. Historic wetland areas have been progressively reduced or drained over time. Representative aerial photographs and annotated imagery are provided in Appendix C.

7.5 Council Information

A review of hazard mapping provided by both Kāpiti Coast District Council (KCDC) and Greater Wellington Regional Council (GWRC) identified the following key natural hazard risks relevant to the site:

- Flood Hazard - Ponding (blue shading on Figure 6) is indicated along the northwestern boundary and within low-lying areas in the eastern part of the site. These align with natural drainage depressions and areas previously identified as containing peat.

- Flood Likelihood - GWRC flood hazard data classifies the site as being in a medium-likelihood flood zone, with an estimated 1 in 100-year return period. This corresponds to a flood event with a 1% annual exceedance probability (AEP).
- Slope Stability - Most of the site is mapped as low risk for slope failure during an earthquake event. Two small areas in the centre of the site, corresponding to the highest elevations on dune ridgelines, are mapped as moderate risk.
- Seismic Shaking - The entire site is mapped as having a moderate risk for ground shaking during an earthquake, consistent with regional seismic hazard models.
- Liquefaction Susceptibility - GWRC mapping indicates the entire site is located within an area classified as high liquefaction risk
- Tsunami Hazard - GWRC mapping indicates that the western portion of the site falls within the yellow tsunami evacuation zone. This zone represents areas exposed to tsunami inundation from the maximum credible event, defined using probabilistic wave heights with a 0.04% annual exceedance probability (AEP), equivalent to a 2,500-year return period.

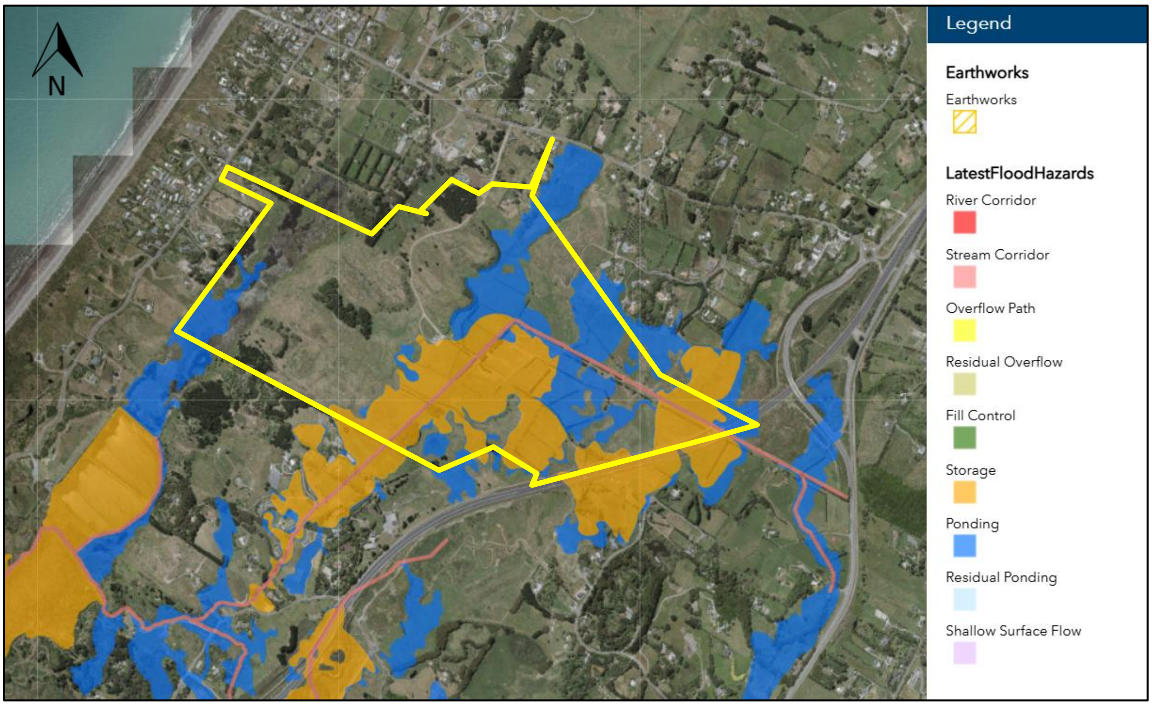

Figure 6. Kāpiti Coast District Council (KCDC) flood hazard mapping for the Waikanae North site.

8. Geotechnical Investigation

In this section we will present both our site-specific investigation information as well as nearby information.

8.1 Site Walkover and Drone Survey

A site walkover and drone assessment was undertaken by a Senior Engineering Geologist in May 2025. The general site conditions are illustrated in Figure 7 and Figure 8. The following key observations were made:

- The site comprises a series of undulating Holocene dune ridgelines, low-lying areas, and swale features. These landforms are consistent with earlier CGW assessments (CGW, 2021; CGW, 2022).
- Distinct geomorphic zones were observed:
    - Upper dune areas are present in the central and north-western sectors of the site. These elevated ridgelines are vegetated with pasture grasses or scrubby bush. Surface soils in these areas appear sandy and well drained.
    - Lower dune areas are less elevated and display more variation in local slope. These ridges transition into shallow depressions and are often used for pastoral farming.
    - The low-lying areas are characterised by swampy, low-lying ground, with visible signs of seasonal wetness, patchy vegetation, and historic ponding. These zones coincide with mapped peat extents and flood-prone areas. Within this area, there are isolated dunes up to 4.0 m higher than the surrounding land. The dunes are generally free of organic material.
- Several minor stream channels and artificial drains traverse the site, generally following these natural low points. These features connect to lower-elevation outflow zones and align with flood hazard mapping (refer to Figure 6).
- The land is currently used for pastoral farming, with grazing paddocks, shelterbelts, and informal vehicle access tracks. A farm shed and multiple transmission towers are located centrally within the site.
- No active slope instability was observed during the walkover. Localised unevenness, soil pugging, and minor ponded areas were present in low-lying areas but are attributed to drainage limitations and stock rather than ground movement.
- High-voltage transmission lines and supporting towers traverse the eastern portion of the site in a north–south direction

These observations support earlier CGW assessments and confirm the site's strong geomorphic contrast between elevated, well-drained dune ridges and lower, compressible and flood-prone terrain.

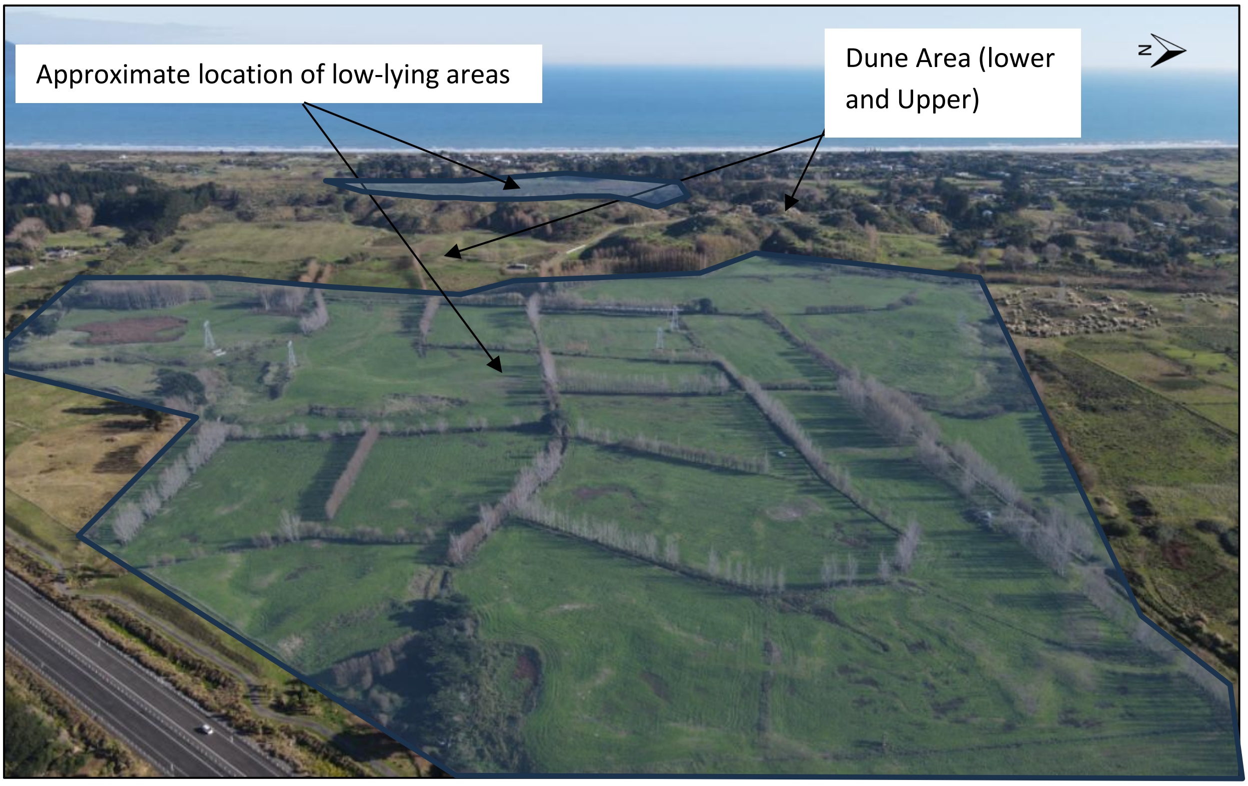

Figure 7. Aerial view looking northwest across the approximate area of the Waikanae North site. The image shows undulating dune terrain, low-lying areas, and central-eastern transmission lines. Kāpiti Island is visible in the background, with State Highway 1 in the foreground. Photo captured May 2025 during CGW site walkover.

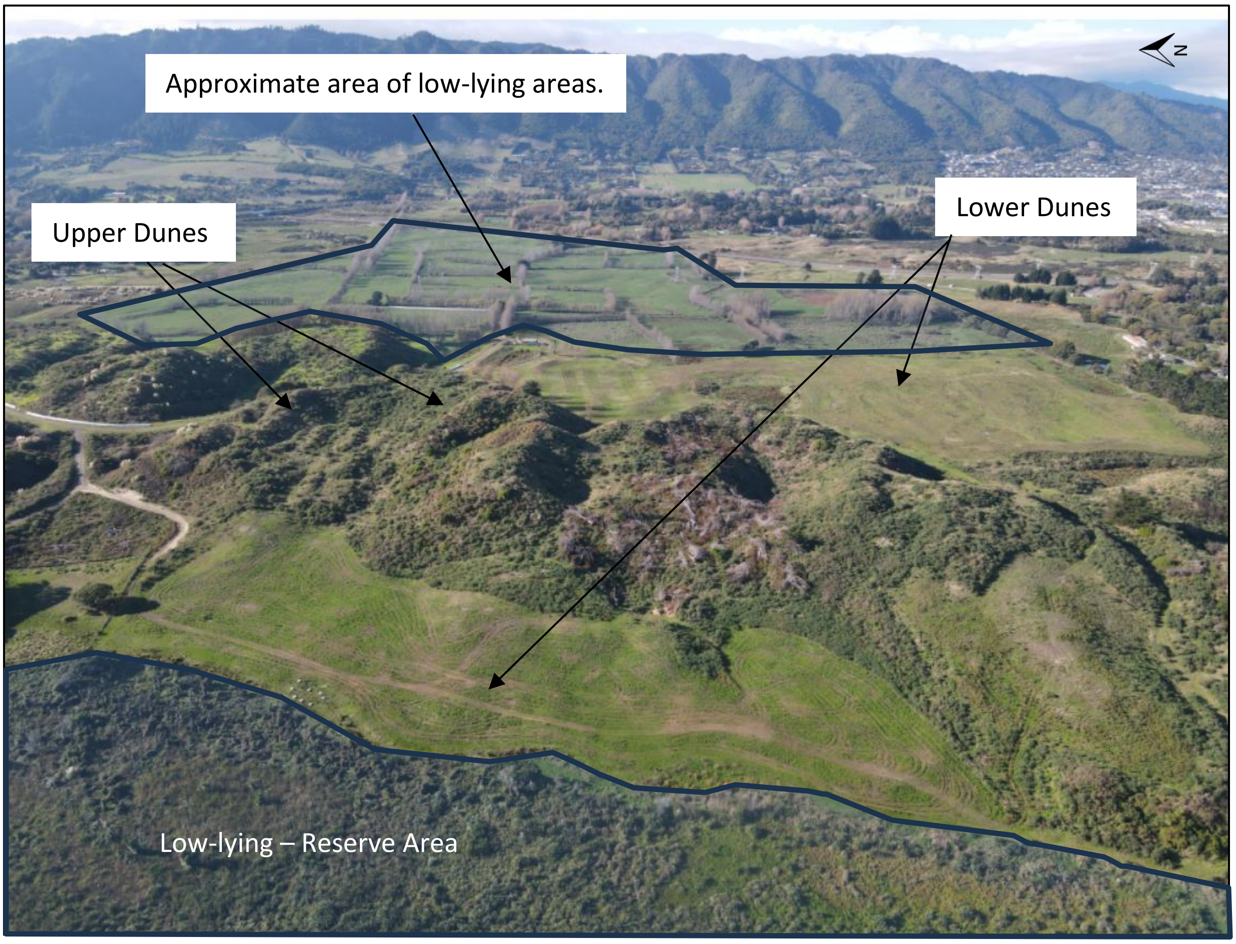

Figure 8. Aerial view looking southeast across the higher dune ridges at the encompassing the Waikanae North site. The image shows steep vegetated dune landforms in the foreground, transitioning to central paddocks and low-lying areas in the distance. The Tararua Ranges and Waikanae township are visible in the background. Photo captured during the May 2025 CGW site walkover.

9. Site Specific Investigation

The initial 2025 field investigation was undertaken by CGW Consulting Engineers in May 2025 and comprised the following exploratory work:

- Sixty-five (65) hand auger boreholes, undertaken across the subdivision area to assess near-surface soil conditions and identify organic soils or peat;
- Thirty-five (35) Dynamic Cone Penetrometer (DCP) tests, completed at selected hand auger locations to assess shallow soil strength and resistance;
- One hundred and forty-five (145) Cone Penetrometer Tests (CPTs) to characterise subsurface stratigraphy, screen for liquefaction risk, and inform ground zoning;
- Five (5) soil soakage tests carried out in representative soil zones to inform stormwater disposal design;
- Twelve (12) piezometers installed across a range of terrain types to monitor groundwater levels.

In December 2025, additional testing was undertaken on the property at 107 Paetawa Road comprising two hand auger boreholes and DCP tests to a target depth of 5.0 m and one CPT test to 10.0 m bgl.

A visual-tactile field classification of the subsoils encountered during hand auger hole drilling was carried out in accordance with 'Guidelines for the Field Classification and Description of Soil and Rock for Engineering Purposes' (NZGS, 2005) and Dynamic Cone Penetrometer testing was carried out in accordance with NZS 4402.1988, Test 6.5.2, 'Dynamic Cone Penetrometer'. Handheld shear vane testing was undertaken as per BS 1377-7:1990.

The CPTs were carried out by a specialist CPT rig operated by CPT Elite in accordance with ASTM Standard D5778-12 'Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils'. The CPT test data was provided to us in both graphical form and as a full electronic record for use in later data interpretation and analyses.

9.1 Ground Model

Subsurface conditions at the Waikanae North site were assessed using hand augers, DCPs, CPTs, and piezometer installations during 2025. Testing was distributed to provide coverage across the full subdivision area, with targeted investigations in low-lying, transitional, and elevated zones. In addition to the 2025 fieldwork, data from previous CGW investigations and nearby borehole logs from the New Zealand Geotechnical Database (NZGD) have been incorporated to inform the ground model.

Based on surface topography and depositional context, the site has been categorised into three geomorphic zones:

- Low-lying Areas,
- Lower Dune Slopes (undulating terrain),
- Upper Dune Ridges (elevated sand crests).

Test locations are shown in Appendix D. Detailed logs are presented in Appendix E and CPT data summary and CPT plots are presented in Appendix F and G respectively. Subsurface conditions for each zone are summarised below.

9.1.1 Low-lying areas

These areas are underlain by variable sequences of dune sand, swamp deposits (peat and organic silt), and deeper marine sands. Many CPTs show a loose surficial layer underlain by organic soils up to 2.5 m deep. However, the presence of peat is spatially variable — absent in some shallow ridges or historic shallow dunes.

Below the peat, loose to medium dense sands are commonly present, often interbedded with silts. In deeper CPTs, these sands extend beyond 10 m and are consistent with marine or estuarine origins.

Table 3: Low-lying Areas			
Soil Type	**Depth to bottom of layer (m)**	**Typical Thickness (m)**	**Notes**
Silty SAND/ SAND (TOPSOIL)	Avg ~0.18	0.0 – 0.7	Moist, dark brown, rootlets common
Fine SAND[1] (DUNE DEPOSITS)	Avg ~0.40	0.2 – 0.6	Very Loose
Peat / Organic Silt (SWAMP DEPOSITS)	~0.2 – 3.5	Avg: ~1.21	Black to dark brown, fibrous, soft to firm
Fine to Medium SAND (MARINE SANDS)	>1.5 – >10	Variable	Grey, loose to medium dense
Notes: • Note[1]: 6 of the hand augers encountered loose sand overlying the PEAT. In most cases the PEAT was lying directly under the topsoil. • Peat zones underlie much of this area. • Low bearing strength and high settlement potential. • Nearby NZGD data indicates that gravel deposits may be encountered below 20 m bgl.			

Peat was not encountered in all hand auger locations within the Low-lying Areas. Localised sandier profiles were observed, particularly along shallow ridges or historic shallow dunes. The extent and depth of peat across the site is shown in Figure 9, derived from CGW test data and summarised in the Peat Depth Overview Plan prepared by Landlink Ltd (2023).

9.1.2 Lower Dune Slopes

The Lower Dunes are characterised by uniform dune sands. CPTs typically show low cone resistance values in the top 1.5 m, transitioning to dense to very dense profiles at greater depth. No widespread peat or organic-rich deposits were identified in this zone. Some local variability in fines content is evident in CPT behaviour type plots, likely reflecting historic dune reworking or aeolian processes.

Table 4: Lower Dune Slopes			
Soil Type	**Depth to bottom of layer (m)**	**Typical Thickness (m)**	**Notes**
SAND, minor to some silt (TOPSOIL)	Avg ~0.26	0.05 – 0.50	Dark brown, rootlets, Loose
Fine to Medium SAND, minor silt (DUNE DEPSOITS)	>10	Not Proven	Grey, Loose grading to dense and very dense below 1.5 m bgl.
Notes: • Testing locations at variable RL levels across lower dunes. • Peat: Not observed. • Nearby NZGD data indicates that gravel, clay and isolated peat deposits may be encountered below 23 m bgl.			

9.1.3 Upper Dune Ridges

The Upper Dune Ridges are comprised almost entirely of well-drained, uniform sand. CPT profiles indicate consistently loose to medium dense sand in the upper 1.0–1.5 m, transitioning to dense to very dense conditions below. These zones are located at higher elevation and are unsaturated in most cases.

CPTs show very low fines content and increasing resistance with depth, consistent with mature dune formation and minimal recent deposition. No peat or soft compressible layers were observed.

Table 5: Upper Dune Ridges			
Soil Type	**Depth to bottom of layer (m)**	**Typical Thickness (m)**	**Notes**
SAND, minor to some silt (TOPSOIL)	Avg ~0.23	0.00 – 0.70	Dark brown, rootlets, Loose
Fine to Medium SAND, minor silt (DUNE DEPSOITS)	>10	Variable	Dry to moist, light grey/orange, Loose grading to dense and very dense below 1.5 m bgl.
Notes: • Testing locations at variable RL levels across Upper dunes. • Peat: Not observed. • Uniform dense sand below 1.5 m.			

9.1.4 Ground Model Summary

Subsurface conditions vary significantly across the three geomorphic zones and are controlled by topographic position, groundwater elevation, and historic depositional processes.

Peat was identified in the low-lying areas of the site, typically underlying a thin mantle of topsoil or loose sand and extending to depths of up to 2.6 m. However, it was not present in all locations within this zone, highlighting localised variability likely associated with former drainage paths or elevated dunes.

In contrast, the dune environments are underlain by uniform sand with increasing density with depth. No peat was encountered in these areas.

9.2 Peat Assessment

9.2.1 Overview

Significant peat deposits have been identified within the eastern and southern portions of the Waikanae North site, associated with low-lying swampy terrain and historic wetland features. The extent and thickness of peat were assessed based on 65 hand auger investigations and DCPs completed during 2025, with additional reference to previous CGW work and regional geological mapping. Peat was typically encountered at shallow depth below topsoil and varied considerably in lateral extent and thickness.

9.2.2 Peat Thickness and Distribution

Peat thickness ranged from 0.2 to 2.6 m, with an average thickness of approximately 1.2 m across locations where it was present. The thickest deposits were encountered near the central and eastern low-lying basins and taper off toward the transitional dune margins. The deepest base of the peat layer encountered was in Hand Auger HA094 at a depth of 3.6 m bgl.

Not all low-lying areas contain peat. In some locations, elevated swales and shallow ridges contained sandy soil profiles with minimal organic content. The mapped distribution and thickness of peat across the site is shown in Figure 9, derived from CGW's 2025 data and compiled into a contour-based depth map by Landlink Ltd.

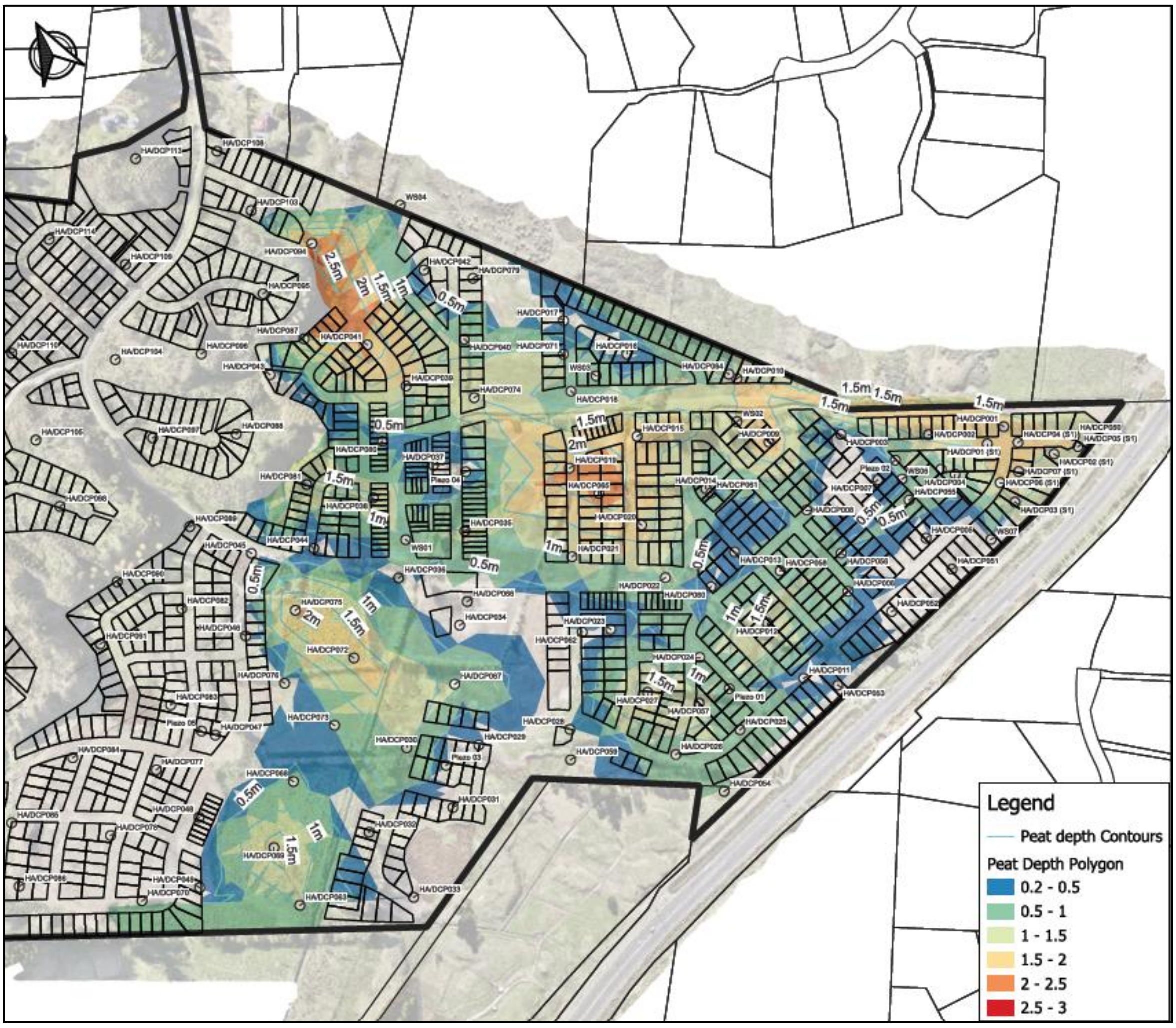

Figure 9. Peat Depth Overview Plan - Mapped extent and estimated thickness of near-surface peat across the site, based on 2024 CGW shallow ground investigations. Prepared by Landlink Ltd.

9.2.3 Peat laboratory testing

In the 2024 Investigation four soil samples were collected during the hand auger borehole fieldwork and dispatched to Fulton Hogan Laboratory in Christchurch. These samples were subjected to ignition testing to determine the percentage organic content as per NZS 4402:1986 Test 3.1.2. (*Organic Content – subsidiary method by ignition*).

Three of the samples were taken from peat intervals and the fourth sample (HA/DCP02) was taken from a dark black silty sand layer directly beneath a peat layer for comparative purposes.

The results of the testing are shown in Table 6 and Appendix H.

Table 6: Peat Laboratory Results

Sample	Depth (meters)	Organic Content (%)	Moisture Content (%)
HA/DCP01	1.1 – 1.9	58	713
HA/DCP02	0.2 – 1.3	78	342
HA/DCP02	1.6 – 2.6	0	47
HA/DCP03	0.3 – 1.5	64	357

From Table 6, the organic content of three of the four intervals sampled are consistent with peat. The moisture content indicates that the peat will be subject to significant volumetric changes when dried.

9.3 Groundwater

Groundwater conditions were assessed through a combination of long-term piezometer monitoring and shallow hand auger observations completed in May–October 2025. Depths to groundwater across the site are shallow and spatially variable, generally correlating with surface elevation and geomorphic setting.

Groundwater was encountered at depths of less than 1.0 m bgl in low-lying areas (and locally deeper on dunes within the low-lying areas), and greater than 2.5 m bgl in elevated dune terrain. Several hand auger probes advanced to depths of up to 5.0 m did not intercept groundwater, particularly in the elevated parts of the site, indicating that the groundwater table in these areas are likely below the depth of investigation. This highlights the significant influence of the site topography on groundwater depth, with the shallowest groundwater table observed in swampy depressions and the deepest in dune ridges.

For reporting, the site has been grouped into three groundwater zones:

Zone A – Low-lying Flats (Eastern and Southern)

- Found near piezometers: P01 to P07, and hand augers in HA/DCP cluster near low elevation areas.
- Average depth to groundwater: ~1.0 m bgl.
- Minimum observed depth: 0.40 m bgl.

Notes: Saturated conditions in peat-affected areas; groundwater near surface following rainfall; persistent high groundwater table.

Zone B – Central Undulating Dunes

- Found around piezometers: P08 to P10, with corresponding hand augers in moderate elevation terrain.
- Average depth to groundwater: ~1.5–1.8 m bgl.
- Notes: Fluctuating groundwater table within sandy dune deposits; locally variable depending on microtopography.

Zone C – Elevated Dune Ridges (Northwest and West)

- Found near piezometers: P11 to P12, and hand augers in elevated terrain.
- Average depth to groundwater: >2.5 m bgl.
- Notes: Groundwater not observed in several hand augers up to 5.0 m depth; well-drained sandy soils.

Table 7: Groundwater

Area	Representative Locations	Typical GW Depth (m bgl)	Comments
Low-lying Flats	P01–P07, HA/DCP in flats	0.1 – 2.0 (up to 4.0 m on some of the isolated dunes)	Saturated low areas, peat zones, strong surface influence
Central Undulating Dunes	P08–P10, central dunes	0.6 – 1.9	Undulating sandy terrain with variable groundwater depths
Elevated Dune Ridges	P11–P12, elevated dunes	>2.5	No groundwater in several hand augers to 5.0 m; deeply drained

The site exhibits a shallow, topographically influenced groundwater table. Low-lying eastern zones have near-surface groundwater and are constrained by peat and organic soils. Elevated dune areas are drier and more suitable for development.

10. Liquefaction

10.1 Liquefaction Analysis Methodology

Assessment of liquefaction potential has been undertaken using a combined dataset of Cone Penetrometer Tests (CPTs) from both recent and historic investigations across the site.

A total of 155 CPTs have been referenced to inform this assessment:

- 146 CPTs were completed in 2025 (145 in May/June; 1 in December) as part of the current subdivision investigation. These tests were distributed across the Low-lying Areas, Lower Dunes, and Upper Dunes to provide broad spatial coverage of the development area.
- 9 CPTs were sourced from previous site-specific investigations and reviewed to ensure data quality and relevance prior to inclusion.

The CPT data was processed using current best-practice guidance as outlined in *MBIE Module 1 (2021)*. Liquefaction potential was assessed using the Boulanger and Idriss (2014) method for triggering analysis, with liquefaction-induced free-field settlement calculated using the methodology proposed by Zhang et al.

The following seismic loading scenarios were assessed:

- SLS – Serviceability Limit State (1:25 Return Period): Mw 6.5, PGA 0.13g
- SLS2 – Extended Serviceability (1:100 Return Period): Mw 7.1, PGA 0.28g
- ULS – Ultimate Limit State (1:500 Return Period): Mw 7.7, PGA 0.68g

Groundwater conditions were determined from piezometer data and CPT pore pressure responses collected during the 2025 investigation. For the purposes of analysis, earthquake groundwater levels were conservatively adjusted based on geomorphic setting:

- Low-lying Areas: Assumed fully saturated (GWD = 0 m)
- Lower Dunes: Assumed 1.0 m above interpreted groundwater
- Upper Dunes: Assumed 0.5 m above interpreted groundwater

Isolated CPT's within the low-lying areas have been assessed using modified groundwater levels due to the CPTs being located on dunes within this area. The groundwater level used for analysis on these specific CPT's has been determined using the elevation relative to the surrounding land.

Of the 155 CPTs, 154 tests provided valid data for liquefaction assessment. CPT151 was excluded from analysis due to early refusal but is included in the total count of CPTs undertaken.

Detailed information on each CPT including location, depth, inferred groundwater levels and predicted settlement values is provided in Appendix F – CPT Summary Dataset.

Graphical outputs of raw CPT data and liquefaction triggering plots are presented in:

- Appendix G – CPT Plots
- Appendix I – Liquefaction Analysis Plots

The resulting liquefaction settlement predictions are summarised by geomorphic zones in the following section.

10.2 Liquefaction Induced Settlement

Predicted free-field liquefaction settlements have been summarised by geomorphic zones. A total of 155 CPTs with valid settlement data were analysed. These results reflect estimated post-earthquake vertical settlements under each design-level event (SLS, SLS2, and ULS), based on the adjusted groundwater assumptions outlined in Section 10.2.

Table 8: Low-lying Areas – CPT Based Liquefaction Analysis Results for Design Events

	Predicted Liquefaction Induced Settlement (mm)		
	SLS	SLS2	ULS
Minimum	5	10	10
Maximum	30	95	105
Average	15	30	50

Table 9: Lower Dunes – CPT Based Liquefaction Analysis Results for Design Events

	Predicted Liquefaction Induced Settlement (mm)		
	SLS	SLS2	ULS
Minimum	< 5	< 5	< 5
Maximum	30	40	90
Average	<5	10	30

Table 10: Upper Dunes – CPT Based Liquefaction Analysis Results for Design Events

	Predicted Liquefaction Induced Settlement (mm)		
	SLS	SLS2	ULS
Minimum	< 5	< 5	5
Maximum	5	40	60
Average	< 5	10	25

The settlements presented in Table 8 to 10 above are to the nearest 5 mm. Due to the inherent uncertainty in calculating liquefaction induced settlement, the calculated free field settlements (land settlement) are indicative only. Actual settlements on site may vary from those above and do not consider foundation influences; volume loss from surface expression, loss in bearing strength and influences from lateral spreading.

These results indicate the following:

- Liquefaction-related settlement during SLS-level shaking is negligible across most of the site. However, at SLS2 levels of shaking, settlement increases across much of the site, with average index settlements in the low-lying areas increasing to approximately 30 mm and localised values up to 95 mm.
- Low-lying Areas: Moderate liquefaction-related settlements are predicted under ULS loading, with localised values exceeding 100 mm. The areas where ULS values exceeded 100 mm are located within the wetland areas not being developed. All assessed CPT's within to-be-developed areas returned ULS settlements less than 100 mm.
- Lower Dunes: ULS index settlements are typically in the range of 20–50 mm, with some CPTs indicating up to 95 mm of settlement.
- Upper Dunes: Predicted settlements are generally low, with ULS values mostly under 30 mm.

Liquefaction-related settlement during SLS-level shaking is negligible across most of the site. Based on the variability and spatial distribution of results, we estimate that differential settlements across typical residential building footprints are expected to be:

- Up to 60 mm under SLS
- Up to 70 mm under ULS conditions

10.3 Liquefiable Layer Trends

A total of 154 CPTs were included in the liquefaction analysis, distributed by geomorphic zone as follows:

- Low-lying Areas: 91 CPTs
- Lower Dunes: 49 CPTs
- Upper Dunes: 14 CPTs

Low-lying Areas (91 CPTs)

Liquefaction under SLS shaking is generally limited, with only thin, discontinuous layers occasionally present in the upper 3.0 m.

For SLS2, liquefiable soil layers are more consistent, typically occurring between 2.5 and 5.5 m, especially in areas with shallow groundwater and loose sandy layers.

Under ULS conditions, widespread triggering is predicted between 2.5 and 6.5 m, with increased thickness and severity. However, layers remain interbedded and moderately thick rather than continuous.

Lower Dunes Zone (49 CPTs)

Minor liquefaction is predicted under SLS conditions.

For SLS2, isolated liquefiable layers begin to appear between 3.0 and 6.5 m depth, primarily in dune flanks and shallow depressions.

For ULS, these layers become more continuous but are generally thinner and less severe than those in the Low-lying Areas.

Upper Dunes Zone (14 CPTs)

Minor liquefiable soils are typically present under SLS or SLS2 loading.

Under ULS, some CPTs show minor to moderate liquefaction between 4.5 and 8 m.

The Upper Dunes are dominated by dense, unsaturated sands, resulting in low liquefaction susceptibility.

10.4 Lateral Displacement Assessment

The site can be broadly divided into low-lying areas and elevated dune environments. The risk of lateral displacement varies accordingly.

Low-lying Areas:

These areas are generally near local drainage channels, low points, or former watercourses. While no major water bodies are present, the combination of shallow groundwater and moderate liquefaction potential at depth increases the likelihood of minor to moderate lateral movement in ULS-level shaking.

Dune Areas:

The dunes are elevated, with groundwater typically deeper and liquefaction potential low or confined below the base of slope. These areas are considered to have a negligible risk of lateral displacement under all design-level seismic events.

10.4.1 Global Lateral Movement

- Low-lying Areas: Minor to moderate global lateral movement (i.e. <100 mm at ULS) may occur, particularly adjacent to drainage channels or local low points.
- Dunes: Global lateral movement is expected to be negligible due to elevation, material density, and liquefaction depth.

10.4.2 Lateral Stretch

- Low-lying Areas: Minor lateral stretch (<50 mm at ULS) is likely in areas with variable ground conditions or adjacent to waterways.
- Dunes: No appreciable lateral stretch is expected.

10.5 Expected Future Land Performance

The MBIE Guidelines provide classification criteria for expected future land performance based on predicted settlement and lateral movement. These categories help guide appropriate foundation design and land development decisions in liquefaction-prone areas.

Based on the CPT-based liquefaction and settlement analysis, the site has been divided into the following technical categories:

Low-lying Areas

The Low-lying Areas exhibits moderate liquefaction-induced settlement and minor to moderate lateral movement under seismic loading. While SLS-level shaking is expected to result in minimal deformation, widespread liquefaction is predicted between approximately 2.5 and 6.5 m depth under ULS conditions.

We consider the Low-lying Areas to be consistent with Technical Category 2 (TC2), due to the depth and continuity of liquefiable layers, lateral deformation, and predicted settlement.

Dune Areas (Lower and Upper Dunes)

Liquefaction potential in the dune areas is generally low. Liquefaction induced settlement is not expected under SLS or SLS2 events, and is limited to thin, deep layers under ULS shaking. These areas are underlain by dense, unsaturated sands with deeper groundwater, resulting in low susceptibility to deformation.

We consider these areas to be consistent with a Technical Category (TC1) performance.

Isolated TC3 Observations

A total of 3 CPTs returned liquefaction induced settlement values meeting the MBIE thresholds for TC3 classification. All of these are located within the Low-lying Areas. These assessed TC3 areas are spatially isolated and generally associated with localised soft or organic soils and shallow groundwater and are located within areas not being developed (i.e. within areas remaining natural area and will not supporting dwellings or infrastructure).

The recommended Technical Category mapping is shown in Figure 10.

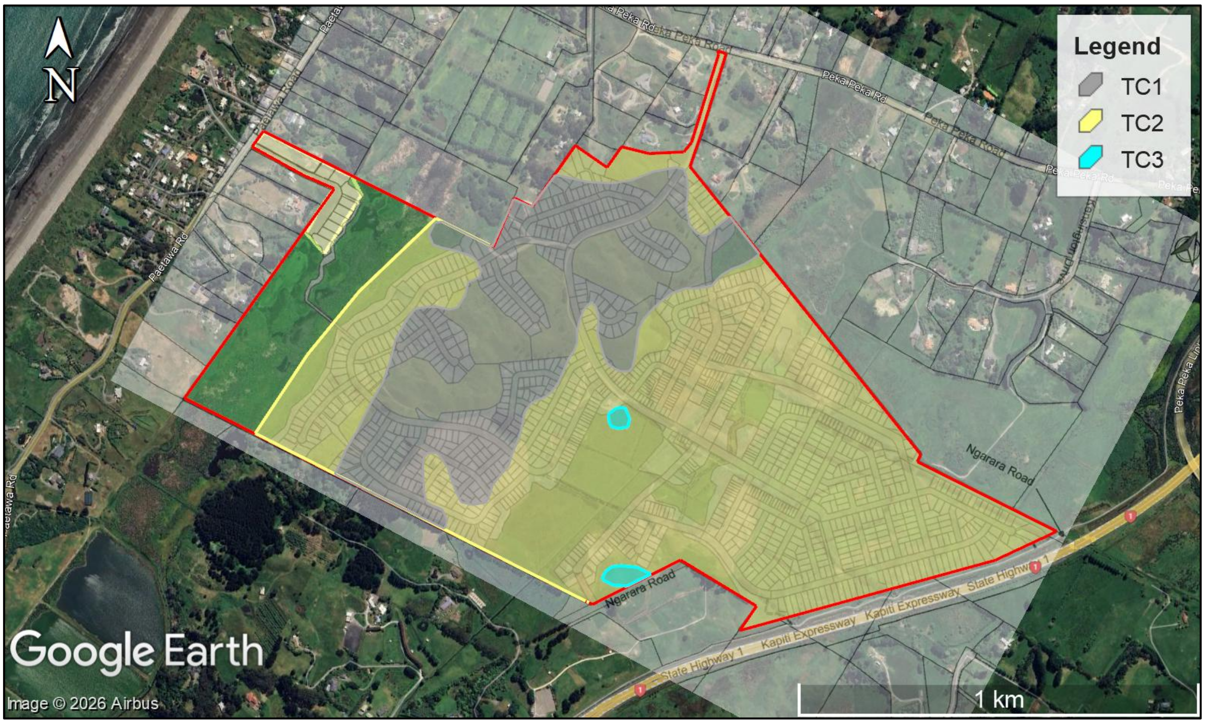

Figure 10. Proposed MBIE Technical Category Zoning Based on CPT Interpretation

11. Static Settlement Assessment

To assess static settlement potential within the low-lying areas underlain by peat, we used the software CPeT-IT to analyse CPTs 11–20, which are representative of zones where organic soils were encountered. The settlement analysis simulated a 300 mm-wide strip standard footing subject to a 50 kPa vertical load, approximating a typical residential waffle slab or strip foundation.

Settlements were modelled for:

- Primary consolidation (assumed duration: 1 month), and
- Long-term creep settlement (over a 600-month / 50-year design life).

Across all CPTs analysed, total static settlements ranged from 40 mm to over 100 mm, with secondary (creep) settlements often exceeding primary settlement values. These high settlements are due to the very low stiffness and high compressibility of the near-surface peat layers (typically from ~0.2 m to ~2.5 m depth).

These results indicate that natural peat deposits at the site are not suitable for direct residential development without ground improvement. Shallow foundations placed on untreated peat would experience significant long-term settlement, well in excess of typical serviceability limits for residential structures.

In contrast, the dune environments (Lower and Upper Dunes) are underlain by loose to dense sand with no identifiable organic soils. These sands are expected to result in negligible static settlement under typical residential loads, subject to appropriate subgrade compaction during construction. As such, static settlement is not considered a limiting factor in these areas.

12. Slope Stability Analysis

A preliminary slope stability assessment has been undertaken for the proposed residential development at Waikanae North. Four representative cross sections (Profiles A–A′, B–B′, C–C′, and D–D′) were selected based on the available Digital Elevation Model (DEM) to represent typical and worst-case slope geometries across the dune terrain. These profiles were analysed to evaluate current (undeveloped) slope conditions only.

Ground conditions were interpreted from nearby CPTs, with stratigraphy and soil strength parameters assigned based on typical values for topsoil and dune sand layers of varying densities. Materials were modelled using the Mohr-Coulomb failure criterion.

The following ground and loading conditions were modelled:

- Static (Normal Groundwater) – based on site observations
- Extreme Groundwater (EGW) – worst-case groundwater levels due to prolonged rainfall
- Seismic SLS Event – using a Peak Ground Acceleration (PGA) of 0.13 g
- Seismic ULS Event – using a Peak Ground Acceleration (PGA) of 0.68 g

These PGA values were derived in accordance with *MBIE Module 1 (2021)* for residential structures of Importance Level 2 (IL2).

The slope stability analysis results for each profile are presented in the following section and summarised in Table 12. Profile locations are shown in Figure 11.

Figure 11. Locations of slope profiles used in undeveloped slope analysis.

The soil parameters used for the analysis are summarised in Table 11 below. As much as possible, conservative parameters were allowed for in our modelling. Slope stability analysis models are presented in Appendix J.

Table 11: Design Parameters for Slope Stability Analysis			
Soil Type	**Unit Weight (kN/m³)**	**Cohesion (kPa)**	**Friction Angle (°)**
Topsoil	17	1	26
Loose Sand	18	2	28
Medium-Dense Sand	19	10	35
Dense Sand	20	15	40
Very Dense Sand	21	20	45
Silt	19	25	30

12.1 Slope Stability Analysis and Discussion

Slope stability analysis has been undertaken for the four profiles using assumed circular failure surfaces for the slope in its current undeveloped state. Best practice for slope modelling in New Zealand indicates the following minimum Factors of Safety (FoS) are required when modelling slopes for lightweight buildings:

- Static (normal groundwater) - >1.5
- Extreme (worst credible) - >1.3
- Seismic SLS Event - >1.2
- Seismic ULS Event - >1.0

Table 12 below present the minimum FoS's from the analyses for scenarios for profile A through profile D. The complexity of ground conditions, the adequacy of information obtained from site investigation, the certainty of the design parameters, such as friction angle and shear strength, determine the confidence in the Factor of Safety.

Table 12: Slope Stability Results				
Section	**Scenario**	**Minimum FoS Achieved**	**Minimum Required FoS**	**Comments**
Cross Section A-A′	Static	1.712	1.5	Minimum FoS achieved.
	Extreme GW	0.865	1.3	Few slope failures within sand near slope face. Drainage management critical.
	Seismic SLS	1.278	1.2	Minimum FoS achieved.
	Seismic ULS	0.552	1.0	Significant instability in upper sand.
Cross Section B-B′	Static	1.168	1.5	Instability in Topsoil. FoS generally achieved in underlying natural soils.
	Extreme GW	0.529	1.3	Significant slope failures within L and MD sand. Drainage management essential.
	Seismic SLS	0.923	1.2	Below recommended limit. Slope failure confined to uppermost shallow layers.
	Seismic ULS	0.405	1.0	Widespread instability in upper slope.
Cross Section C-C′	Static	2.190	1.5	Minimum FoS achieved.
	Extreme GW	1.267	1.3	Slightly below min FoS. Localised shallow movement possible.
	Seismic SLS	1.687	1.2	Minimum FoS achieved.
	Seismic ULS	0.754	1.0	Below min FoS. Shallow instability predicted.

Table 12: Slope Stability Results				
Section	**Scenario**	**Minimum FoS Achieved**	**Minimum Required FoS**	**Comments**
Cross Section D-D′	Static	1.393	1.5	FoS generally achieved. Few slope failures confined to localised slippage in topsoil layer
	Extreme GW	0.687	1.3	Localised failures in mid-toe zone. Drainage critical.
	Seismic SLS	1.070	1.2	Below recommended limit. Slope failure confined to uppermost shallow layers.
	Seismic ULS	0.465	1.0	Below min FoS requirement. Shallow instability predicted.

All four profiles demonstrate acceptable performance under static and SLS conditions only in some cases. Under extreme groundwater and ULS seismic loading, the minimum recommended Factors of Safety are not achieved for most profiles. From experience in similar soils, instability typically originates in loose and medium-dense dune sands, particularly near the mid-slope and toe zones.

Effective drainage control will be critical to maintaining slope stability, especially on the steeper dune faces. Without intervention, prolonged rainfall or elevated groundwater tables could trigger shallow slope failures even in the absence of seismic loading.

12.2 Preliminary Developed Model Assessment

A preliminary developed slope stability assessment has been undertaken for the proposed residential development at Waikanae North. Three representative cross sections (Profiles 2, 3 and 6) were provided by the client to confirm how the proposed lot layout and earthworks would or would not be susceptible to slope instability following development. The cross sections were based on the available digital elevation model (DEM) and revised slope profiles to represent typical and worst-case slope geometries across the sloping dune terrain incorporating the proposed lot layout.

The following ground and loading conditions were modelled:

- Static (Normal Groundwater) – based on site observations.
- Extreme Groundwater (EGW) – worst-case groundwater levels due to prolonged rainfall.
- Seismic SLS Event – where a seismic coefficient K_h of 0.08 g is assigned to the site to model a serviceability limit state type event.
- Seismic ULS Event – where a seismic coefficient K_h of 0.41 g is assigned to the site to model an ultimate limit state type event.

The Seismic Coefficient (K_h) for the SLS and ULS cases has been calculated using the design equations within the Earthquake Geotechnical Engineering Practice Module 6 (revised Nov 2021):

$$K_h = a_{max} \times A_{topo} \times W_d \qquad K_h = a_{max} \times A_{topo} \times W_d$$

$$K_{h(ULS)} = 0.68 \times 1.2 \times 0.5 \qquad K_{h(SLS)} = 0.13 \times 1.2 \times 0.5$$

$$K_{h(ULS)} = 0.411 \qquad K_{h(SLS)} = 0.078$$

A topographic amplification factor (A_{topo}) of 1.2 and a Wall displacement (W_d) factor of 0.5 has been derived from Table 5.1 and 5.2 respectively within the Earthquake Geotechnical Engineering Practice Module 6 (revised Nov 2021).

The slope stability analysis results for each profile are presented in the following section and summarised in Table 13. Profile locations are shown in Figure 12.

Figure 12. Locations of slope profiles used in preliminary developed slope analysis.

Slope stability analysis has been undertaken for a small sample of developed profiles using assumed circular failure surfaces. Table 13 below presents the minimum FoS's from the analyses for scenarios for profiles 2, 3 and 6. These profiles were chosen as the worst-case scenario interaction between the proposed lot layout and the steepest slope sections.

The complexity of ground conditions, the adequacy of information obtained from site investigation, the certainty of the design parameters, such as friction angle and shear strength, determine the confidence in the calculated Factor of Safety.

Table 13: Developed Slope Stability Results

Section	Scenario	Minimum FoS Achieved	Minimum Required FoS	Comments
Profile 2	Static	1.28	1.5	Minimum FoS generally achieved. Failures confined to topsoil layer. No failures intersected proposed property boundaries.
	Extreme GW	0.618	1.3	Few slope failures within the slope face. Drainage control critical. Failures close to intersecting proposed property boundary.
	Seismic SLS	1.278	1.2	Minimum FoS generally achieved. Failures confined to topsoil layer. No failures intersected proposed property boundary.
	Seismic ULS	0.646	1.0	Minimum FoS generally achieved. Instability limited to topsoil and loose sand. No failures intersected proposed property boundary.
Profile 3	Static	1.525	1.5	Minimum FoS achieved.
	Extreme GW	0.777	1.3	Slope failures identified within slope face. Drainage control critical. Failures close to intersecting proposed property boundary.
	Seismic SLS	1.279	1.2	Minimum FoS achieved.
	Seismic ULS	0.714	1.0	Minimum FoS generally achieved. Instability limited to topsoil and loose sand. No failures intersected proposed property boundary.

Table 13: Developed Slope Stability Results				
Section	**Scenario**	**Minimum FoS Achieved**	**Minimum Required FoS**	**Comments**
Profile 6	Static	1.328	1.5	Minimum FoS generally achieved. Failures confined to topsoil layer. No failures intersected proposed property boundary.
	Extreme GW	0.618	1.3	Slope failures identified within slope face. Drainage control critical. Failures close to intersecting proposed property boundary.
	Seismic SLS	1.127	1.2	Minimum FoS generally achieved. Failures confined to topsoil layer. No failures intersected proposed property boundary.
	Seismic ULS	0.649	1.0	Minimum FoS generally achieved. Instability limited to topsoil and loose sand. No failures intersected proposed property boundary.

The analysis indicates that all modelled developed profiles maintain acceptable stability under static and seismic conditions, with only marginal exceedance of acceptable limits under extreme groundwater scenarios Under extreme groundwater, the minimum recommended Factors of Safety are not achieved for the profiles. Instabilities typically originate in loose and medium-dense dune sands, particularly near the mid-slope and toe slope zones.

Effective drainage control is still critical to maintaining slope stability, especially on the steeper dune faces. Without intervention, prolonged rainfall or elevated groundwater tables could trigger shallow slope failures even in the absence of seismic loading.

Based on the above preliminary results, a nominal setback or no-build zone from the dune crests is recommended to further mitigate risk to proposed structures. The exact setback distance should be confirmed during detailed design; however, the current lot layout is considered generally acceptable from a geotechnical stability perspective.

12.3 Recommendations and Further Modelling

This assessment covers undeveloped slope conditions and limited preliminary developed models. The preliminary developed models provide an initial indication of slope performance under the proposed conditions. These results should be refined as part of the detailed design stage once final earthworks and drainage layouts are confirmed

During detailed design, when final development plans and earthworks plans are available, further slope modelling should be undertaken to:

- Evaluate development-induced surcharge loading
- Inform the design of stabilisation measures if required
- Propose setbacks when necessary.

Where FoS falls below acceptable thresholds, we recommend mitigation through:

- Local slope regrading
- Installation of subsoil drainage
- Structural reinforcement (e.g. palisade wall or piles) near slope crests

A detailed, slope-specific geotechnical assessment will be required at detailed design for all development adjacent to dune faces to confirm design parameters, setback distances, and any required stabilisation or drainage measures. A detailed slope-specific geotechnical assessment will be required during the detailed design phase for any development adjacent to the dune faces.

13. Fault Avoidance Zone

Part of the Waikanae North site lies within a Fault Avoidance Zone (FAZ) associated with the Ōhāriu Fault, as mapped by GNS Science. The fault in this area is classified as "uncertain – poorly constrained", with a recurrence interval of 2,000–3,500 years (Class II). The mapped FAZ traverses the southern portion of the site and is based on interpreted alignment with the wider regional fault trace, extrapolated from trenching and geomorphic analysis further south and east. This FAZ is not identified in the Operative Kāpiti Coast District Plan (2021), which does not include any mapped fault-avoidance overlay or associated building setback controls for this location.

As shown in Figure 13, a portion of the proposed subdivision layout extends into the FAZ.

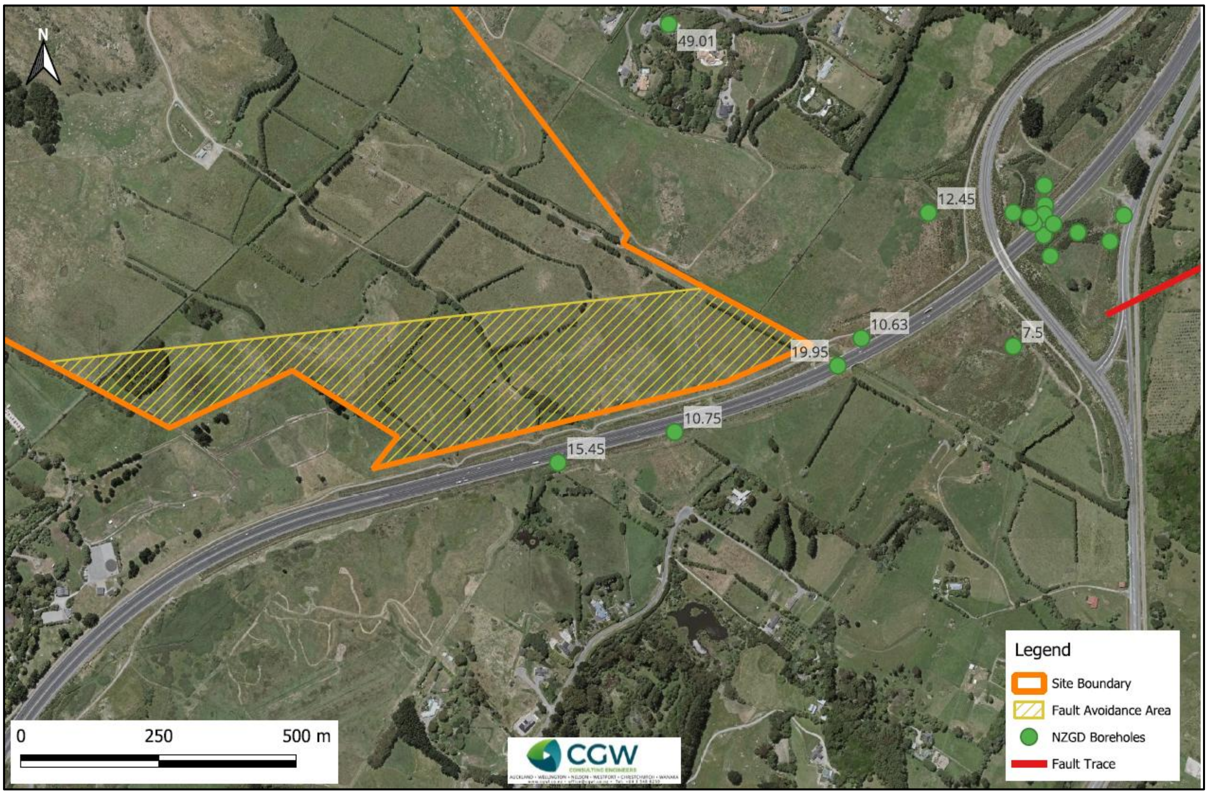

Figure 13. Location of the Fault Avoidance Zone (FAZ) over the southern portion of the site.

A review of NZGD boreholes located within or near the mapped FAZ found no subsurface evidence of active faulting. Specifically:

Table 14: Nearby Testing near FAZ			
Hole ID	**Location relative to the site**	**Depth of Hole (m bgl)**	**Comments**
BH308 (2011)	On eastern boundary within FAZ	10.63	2.5 m of silty peat over fine sand. Dense sand from 7.5 m.
BH347 (2013)	FAZ – eastern alignment	19.95	Clean sand to 12.3 m. Dense sand and gravel below 10.5 m.
BH217 (2010)	Southeastern boundary	10.75	Peat to 2 m. Fine sand becoming dense below 4.5 m.
BH13 (2010)	Southeastern boundary	15.45	Like BH217: Peat to 2 m; dense sand below.
R26/6207 (2003)	~350 m SW of site	32.20	Interbedded peat/sand to 20.5 m; sand/gravel to 25.5 m.

These logs show no abrupt displacement, crushed zones, shear indicators, or duplicated sequences that would suggest the presence of a near-surface fault rupture. Layering is consistent, and profiles are geotechnically typical of coastal Holocene terrain.

Based on the NZGD boreholes reviewed:

- The mapped fault trace likely reflects an inferred regional alignment, not a surface-rupturing fault at this location.
- The absence of disturbance in subsurface logs, including at depths exceeding 10–15 m, supports the conclusion that the fault does not traverse the site in this location, or if it does, it does not present a rupture hazard at shallow depth.
- The alignment of the FAZ with flat, swampy terrain further supports the view that the mapped trace here is geomorphically inferred, not directly trenched.

Given the lack of supporting subsurface evidence and the absence of a mapped FAZ in the Operative Kāpiti Coast District Plan (2021), the mapped feature is considered a regional-scale indicator rather than a confirmed site-specific constraint.

In accordance with the MfE Active Fault Guidelines (2003) for Class II faults with uncertain location, low-importance structures (Importance Level 1 and 2) may be considered appropriate within this area if other geotechnical risks are managed.

Subject to site-specific confirmation, TC2-type foundations (e.g. stiffened slabs or shallow reinforced strip footings) are expected to be suitable in areas within the FAZ.

14. Geotechnical Assessment

14.1 Site Subsoil Classification

We consider that the appropriate site subsoil classification for the Waikanae North site, in accordance with NZS 1170.5:2004 Clause 3.1.3, is Class D (Deep or Soft Soil Sites). This classification is based on the following:

- Site-specific geotechnical testing encountered thick sequences of loose to medium dense sands, soft organic soils (peat), and marine/alluvial deposits, extending to depths greater than 10 m in many areas.
- No shallow rock or dense gravel layer was encountered within the upper 10 m in any CPTs.
- The site's geomorphic setting includes low-lying swampy flats and coastal dune systems, consistent with the characteristics described in Table 3.2 of NZS 1170.5:2004 for Class D soils.
- Supporting geological information (GNS mapping and NZGD data) confirms the absence of shallow bedrock and presence of deep unconsolidated deposits across the site.

14.2 Expansive Soils

In line with the AS2870 methodology of utilising a visual-tactile identification and knowledge of the site soils, we have based our assessment on our investigations and the available geotechnical information for the site. We consider the proposed building to be underlain with Class A, Non-Expansive soils.

14.3 Geotechnical Ultimate Bearing Capacity

Based on the results of the Dynamic Cone Penetrometer (DCP) testing, and in accordance with NZS 3604:2011 and the MBIE Guidance (2012), we assess the following:

- A preliminary Ultimate Bearing Capacity (UBC) of 200 kPa is generally available within the natural sandy soils beneath the topsoil layers across most of the site.
- Following completion of bulk earthworks (including stripping of topsoil, organic material and subgrade compaction), we expect a UBC of 200 kPa will be reliably achieved to support shallow residential foundations.
- "Good ground" as defined by NZS 3604:2011—being soil that can support a UBC of 300 kPa—is generally not achieved until depths of 1.0 m to 2.0 m below ground level and is therefore not typically available for shallow footings without extensive excavation and replacement.

In accordance with the principles of AS/NZS 1170.0:2002 Section 3.2, and as specified in MBIE B1/VM4 Section 3.5, a strength reduction factor of $\phi = 0.5$ should be applied to the Ultimate Bearing Capacity. The resulting design bearing pressure must equal or exceed the factored Ultimate Limit State (ULS) load effects from the proposed structure.

15. Assessment Against RMA Section 106

15.1 Criteria

In accordance with the Resource Management Act 1991 (RMA), the site has been assessed in accordance with Section 106 for natural hazards. Section 106 of the Resource Management Act provides for specific considerations for a subdivision consent in respect of natural hazards. Section 106 states that a consent authority may refuse to grant a subdivision consent, or may granted a subdivision consent subject to conditions, if it considers that -:

- There is significant risk from natural hazards; or
- Sufficient provision has not been made for legal and physical access to each allotment to be created by the subdivision.

For subsections 1a, an assessment of the risk from natural hazards requires a combined assessment of:

- The likelihood of natural hazards occurring;
- The material damage to land in respect of which consent is sought, other land or structures that would result from natural hazards;
- Any likely subsequent use of land in respect of which the consent is sought that would accelerate, worsen, or result in material damage of the kind.

15.2 Assessment

The Waikanae North site has been assessed for the following natural hazards based on site investigations, mapping, and regional datasets:

Fault Rupture

Part of the Waikanae North site lies within a Fault Avoidance Zone (FAZ) associated with the Ōhāriu Fault, as mapped by GNS Science. In this area, the fault is classified as "uncertain – poorly constrained" with a recurrence interval of 2,000–3,500 years (Class II). The FAZ traverses the southern portion of the site and is based on inferred regional fault alignment, extrapolated from trenching and geomorphic evidence further south and east.

While a portion of the proposed subdivision layout intersects this FAZ, no subsurface evidence of faulting was identified in borehole logs located within or adjacent to the mapped zone. The logs show no signs of displacement, shearing, or duplicated sequences, and are consistent with typical coastal Holocene stratigraphy. This FAZ is not identified in the Operative Kāpiti Coast District Plan (2021)

This suggests that the mapped FAZ reflects a modelled alignment, rather than a confirmed surface-rupturing fault at this location. In accordance with the MfE Active Fault Guidelines (2003), low-importance structures (Importance Level 1 or 2) may be considered appropriate within FAZs associated with Class II faults of uncertain location, where geotechnical risks are otherwise addressed.

Subject to site-specific confirmation, TC2-type foundations (e.g. stiffened slabs or shallow reinforced footings) are expected to be suitable in FAZ-affected areas with adequate bearing strength, provided liquefaction and settlement risks (discussed elsewhere in this report) are appropriately managed.

Slope Stability

Slope stability analysis was undertaken for four representative cross sections across the dune terrain. All sections showed adequate performance under static and SLS loading, but reduced factors of safety under extreme groundwater and ULS conditions, particularly in steep dune areas. These risks can be managed through drainage design and site-specific assessment during the building consent stage. Western flat areas are considered low risk. Preliminary detailed design analysis has shown that the proposed lot layout is acceptable.

Inundation – Debris Flow

No mapped debris flow paths, fans, or associated depositional features were identified. There was no evidence of recent erosion or debris transport observed on site. The risk of inundation from soil or rock debris is considered low.

Inundation – Flooding

Parts of the low-lying eastern flats may be susceptible to ponding or overland flow during extreme rainfall events. However, no residential building platforms are proposed in these areas.

Flood risk is being mitigated through the subdivision's earthworks and civil design, including:

- Raising road and platform levels where needed;
- Incorporating swales, culverts, and engineered drainage systems;
- Providing stormwater detention to manage runoff.

With these measures, flood risk to future development is considered low and appropriately managed.

Liquefaction

Liquefaction assessment across 154 CPTs confirms low to minor liquefaction risk under SLS conditions. Under ULS, moderate to severe settlements are predicted in the Low-lying Areas and Lower Dune zones. No significant lateral spreading (exceeding the TC2 levels) was identified. These results support TC2 and TC3 categorisation for Low-lying Areas and TC1 or better for dune areas. Peat and soft soils are present in parts of the Low-lying Areas and are not suitable for shallow foundations without mitigation.

Static Settlement

Settlement modelling using CPeT-IT confirms that significant static settlements (>100 mm) may occur in peat-rich areas under typical residential loads. These zones are unsuitable for direct development without engineered ground improvement, which we understand will be taking place as part of the subdivision earthworks. Dune sands, in contrast, exhibited low compressibility and are not expected to pose any static settlement risk.

In our opinion, under Section 106 of the RMA, there are no geotechnical reasons preventing the development, provided the developer takes the appropriate measures as recommended in this report.

16. Conclusions and Recommendations

- A total of 155 CPTs, supported by hand augers, DCPs, and piezometers, were completed across the Waikanae North site. Subsurface conditions vary across the three main geomorphic zones:
  - Low-lying Areas: Peat, loose sands, and marine silts dominate, with significant variability and soft ground conditions typically found between 0.2 m and 2.6 m depth.
  - Lower Dunes: Loose to medium dense dune sands with occasional silt, underlain by dense sands; no peat was encountered.
  - Upper Dunes: Clean sands, generally dry and dense below 1.5 m, with good bearing conditions and low deformation potential.
- Groundwater was encountered at shallow depths (0.1–4.6 m BGL), with zone-specific averages of ~1.14 m in the Low-lying Areas, ~2.15 m in the Lower Dunes, and ~3.03 m in the Upper Dunes.
- Liquefaction analysis, using CPT-based methods under SLS, SLS2, and ULS loading, concluded:
  - Negligible liquefaction under SLS events across all zones.
  - Moderate liquefaction-induced settlement under ULS conditions in the Low-lying Areas and Lower Dunes, with differential settlements typically ranging from 20 mm to 35 mm.
  - The Upper Dunes are not expected to experience liquefaction under any design-level seismic event.
- The additional investigation undertaken in December 2025 on the 107 Paetawa Road property comprised one CPT to assess this additional plot of land. The additional CPT confirmed that the area was TC2 in line with other areas of the Lower Dunes. However, as it was a single CPT, further testing may be warranted during the earthworks to confirm the consistency of the underlying ground.
- A preliminary lateral spreading assessment was undertaken using representative cross sections. The analysis indicates that the proposed lot layout is generally acceptable, with potential lateral displacements able to be mitigated through a nominal setback of building platforms from nearby waterways. Further confirmation is recommended during detailed design.
- Static settlement analysis using CPeT-IT on CPTs 11–20 (within peat-rich zones) confirmed predicted long-term settlements exceeding 100 mm under a 50 kPa load. These areas are unsuitable for shallow foundations without ground improvement. Dune sands showed negligible static settlement.

- Part of the site lies within a mapped Fault Avoidance Zone (FAZ) associated with the Ōhāriu Fault, classified as a Class II fault with an uncertain location and a recurrence interval of 2,000–3,500 years. Review of borehole data within and near the FAZ showed no evidence of faulting at shallow depth. In accordance with the MfE Active Fault Guidelines (2003), residential development may proceed within the FAZ subject to a risk-based assessment. TC2-type foundations may be suitable in FAZ-affected areas.
- Slope stability modelling was undertaken along four representative slope profiles. All profiles were stable under static and SLS conditions, but reduced Factors of Safety were observed under extreme groundwater and ULS seismic conditions, particularly in steep dune zones. Instability was generally confined to shallow loose sand. These risks can be mitigated through drainage and slope-specific engineering at the building consent stage.
- A preliminary developed slope stability assessment was also completed using updated seismic parameters derived from *MBIE Modules 1 and 6 (2021)*. The analysis indicated that the proposed earthwork profiles and lot layout generally achieve acceptable stability under static, SLS, and ULS conditions. Localised reductions in Factors of Safety were observed under extreme groundwater scenarios, emphasising the need for effective drainage and confirmation of slope setbacks during detailed design.
- Based on DCP results, an Ultimate Bearing Capacity (UBC) of 200 kPa is typically achieved within natural soils from shallow depth (~0.2 m BGL). "Good ground" (UBC ≥ 300 kPa), as defined by NZS 3604:2011, is generally not reached until depths of 1.5–2.0 m in dune areas.
- The site is preliminarily classified as Subsoil Class D under NZS 1170.5:2004, due to the presence of soft soils and deep unconsolidated deposits in parts of the Low-lying Areas.
- Lateral spreading analyses indicate that potential displacements can be effectively mitigated through a nominal setback of building platforms from site boundaries, reducing the risk of deformation-related damage under seismic loading.

16.1 Recommended Technical Category Zoning

The site has been assigned preliminary MBIE Technical Categories based on liquefaction and settlement modelling, as shown in Figure 14:

- TC1 (Grey zones): Assigned to elevated dune ridges with dense, non-liquefiable sand. Suitable for standard NZS 3604 foundations.
- TC2 (Yellow zones): Applied to much of the gently sloping Lower Dunes and Low-lying Areas. Loose to medium-dense sands with minor liquefaction risk under ULS. Suitable for TC2-type foundations (e.g. stiffened slabs).
- TC3 (Blue zones): Localised zones underlain by soft peat and exhibiting significant static and seismic settlement potential have been identified outside the proposed

development footprint. As these areas are not proposed for development, no further work is considered necessary from a geotechnical perspective.

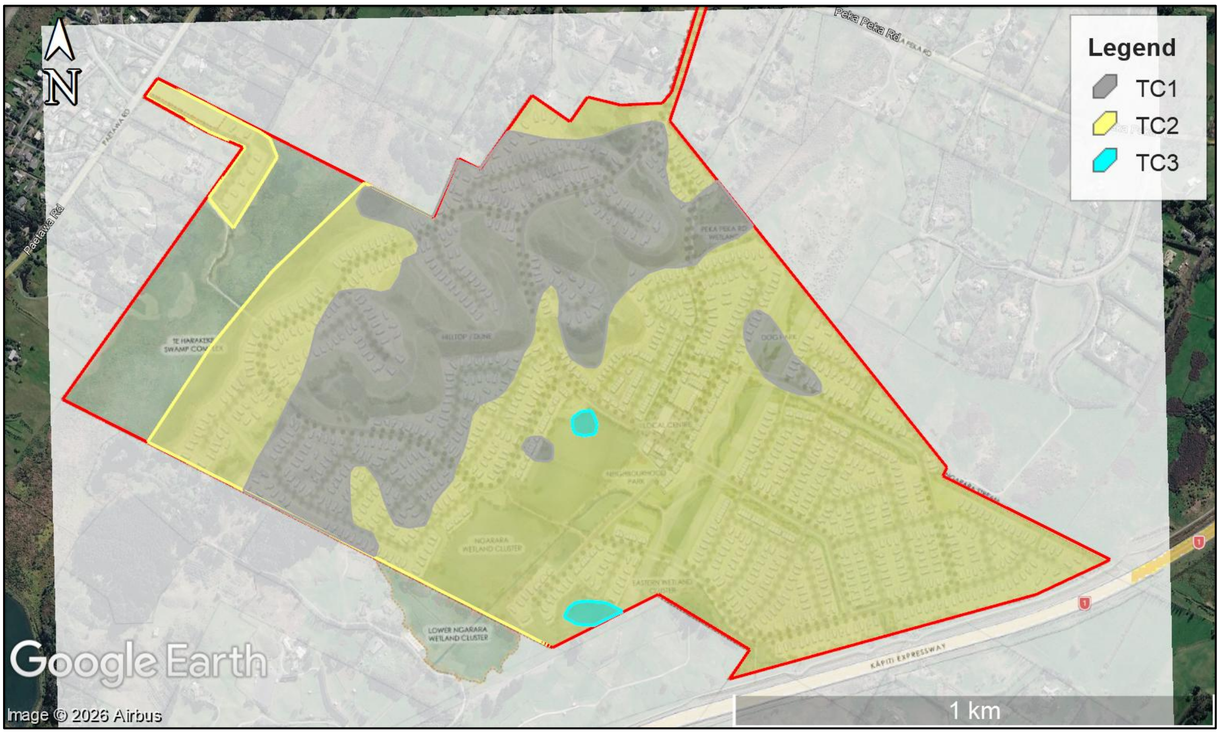

Figure 14. Recommended Technical Category Zoning

16.2 Required Earthworks:

Earthworks will be required to support development across much of the site, particularly within TC2 and TC3 zones. This is expected to include:

- Excavation and removal of peat and soft organic soils, especially in eastern low-lying areas;
- Replacement with compacted site-won dune sand and/or imported clean fill;
- Regrading and installation of subsoil drainage to support future slope and platform stability.

Further details on peat removal, reuse, and engineered fill placement are provided in Section 16.3 to 16.5 of this report.

16.3 Peat Excavation

Excavation and replacement of peat is considered the most practical and effective approach for future development within peat-affected areas. Peat can be stripped from identified low-lying areas and replaced with compacted site-won sand from dune zones or other locally sourced clean fine or granular fill, in accordance with NZS 4431:2022. Given the shallow groundwater observed in some areas (as low as 0.2 m bgl), temporary dewatering and appropriate groundwater control measures will be required during excavation.

The excavation process will require the use of large machinery and robust sediment control measures to avoid offsite impacts. A stringent Earthworks Management Plan should be developed, in consultation with the project geotechnical professional, to ensure that stripping, replacement, drainage, and compaction processes are implemented to an appropriate standard and verified during construction. Based on preliminary information provided by Landlink regarding peat volumes, we have undertaken preliminary calculations of the proposed peat dry back option, and we consider that drying the peat back will reduce the volume down to 275,000 m^3 from 475,000m^3.

16.4 Peat Re-use

The reuse of site-won peat is proposed in selected reserve areas of the subdivision. Subject to drying and geotechnical confirmation, peat may be re-used for landscape purposes, such as noise bunds, infilling shallow gullies, or smoothing local depressions resulting from fill extraction. Where reused in this manner, peat should be placed over natural subgrade, with batters no steeper than 1V:3H and finished surface grades not exceeding 1V:4H. These areas will be reserved for planting and low-intensity landscaping only, with no building or structural loads applied.

In some cases, peat may also be placed into excavated depressions, particularly where suitable clean fill has been removed. Pedestrian tracks may be constructed over such fills, subject to geotechnical review and verification.

Alternatively, site-won peat may be blended with dune sand to reduce the organic content to below 5%. Subject to laboratory testing (e.g. loss on ignition) and field trials, such blended material may be suitable for use as engineered general fill in accordance with NZS 4431:2022, under controlled placement and compaction with professional supervision.

All proposed reuse scenarios must be confirmed by a geotechnical specialist during detailed design and implemented under appropriate observation and testing.

16.5 Sand Dune Reuse

Site-won sand from dune areas is expected to be suitable for re-use as engineered fill, subject to confirmation through laboratory testing. Testing should confirm grading, plasticity, and organic content in accordance with NZS 4431:2022. Where suitable, this material can be incorporated into bulk earthworks following the preparation and approval of a project-specific Earthworks Specification, with placement and compaction carried out under geotechnical supervision.

16.6 Foundation Recommendations

Following completion of bulk earthworks, it is expected that the site will contain areas with varying Technical Category (TC) assignments, which will guide the appropriate foundation types across the subdivision.

- Low-lying areas, including parts of the Fault Avoidance Zone (FAZ), are expected to be assigned MBIE Technical Category TC2. In these areas, TC2-style foundations will be required due to the moderate liquefaction risk.
- Dune environments (both lower and upper dunes), where clean and dense sand is present with minimal deformation potential, are expected to be assigned MBIE Technical Category TC1. In these areas, standard TC1 style foundations (Waffle Slab or SED NZS3604 Foundations) are likely to be suitable, subject to confirmation.

Preliminary assessment indicates that Specific Engineered Design (SED) foundations are likely to be appropriate, with design to be based on an Ultimate Bearing Capacity of 200 kPa, typically available below the topsoil level in undisturbed natural soils or engineered fill.

Additional site-specific geotechnical testing—including test pits, hand augers, and Dynamic Cone Penetrometer (DCP) tests—will be required at each proposed building location during the building consent phase, in accordance with NZS 3604:2011 Section 3.3.1 and MBIE guidance.

Final foundation design must be confirmed at the building consent stage, following geotechnical review of local ground conditions.

16.7 Soakage Testing

To assist in the civil design component of the site development, CGW also undertook 4 soakage tests at various locations on 14/05/2025 across the development at the request of the client. The soakage tests were completed across the northwestern side of the large sand dunes. An additional soakage test was undertaken on 21/05/2025 near the driveway entrance to the site. The testing was completed as per E1/VM1 Section 9.02. The test locations are shown in Figure 15 below.

Figure 15. Soakage Test Locations

Testing was conducted in 1.0 m to 1.2 m deep and 0.1 m diameter hand auger holes. All holes were pre-soaked for 2 hours prior to recording being undertaken.

The results are presented in Table 15. As per good practice, calculated soakage rates should be reduced by a factor of 0.5 in the design of soakage pits to allow for loss of performance over time. Soakage rates should be confirmed on-site in each individual soak pit at the time of construction. These soakage testing results are included in Appendix K.

Table 15: Soakage Test Results

Soakage Test	Depth (m)	Diameter (m)	Presoak Time (min)	Test Length (min)	Minimum Soakage Rate (mm/hr)
ST01	1.0	0.1	120	7	3,000
ST02	0.6*	0.1	120	6	1,800
ST03	1.0	0.1	120	10	2,700
ST04	1.0	0.1	120	9	1,200
ST05	1.2	0.1	120	26	1,500

* Hole collapsed during testing

17. Recommended Resource Consent Conditions

CGW recommends the following consent conditions are applied to the Resource Consent.

17.1 Earthworks Compliance

The consent holder shall undertake all earthworks in accordance with NZS 4431:2022 *Code of Practice for Earth Filling for Residential Development*, and in full compliance with the findings and recommendations of the *Geotechnical Assessment Report for Subdivision – Waikanae North Subdivision* (Document No. 21512-RPT-G-004-A), prepared by CGW Consulting Engineers and dated 22 October 2025.

17.2 Site-Specific Geotechnical Investigations

Prior to the issue of a Section 224(c) certificate for any stage of the subdivision, site-specific geotechnical investigations shall be undertaken by a suitably qualified and experienced geotechnical professional. These investigations shall confirm the appropriate foundation type, required building setbacks, no-build zones or SID areas, and any other lot-specific requirements. The findings shall be documented and submitted to the Council's Development Engineer as part of the geotechnical completion report required under Condition 3.

17.3 Geotechnical Completion Reporting

Upon completion of subdivision earthworks, the consent holder shall provide to the Council's Development Engineer:

a) A Geotechnical Completion Report (GCR) prepared and certified by the project geotechnical professional, confirming that all earthworks and ground improvement measures have been constructed in accordance with the approved geotechnical design and NZS 4431:2022.
b) An earthfill certificate in the form of Appendix A of NZS 4431:2022 completed by the inspecting engineer, verifying compaction testing and fill placement.

Note: If the GCR identifies development limitations that should be communicated to future landowners, the Council Consenting Officer may require a consent notice to be registered on the relevant title(s) under Section 221 of the Resource Management Act 1991. The decision to require consent notices will be made prior to the issue of any Section 224(c) certificate.

17.4 Foundation Design and Construction

The foundation design and construction for any building or structure on each allotment shall be undertaken in accordance with:

a) The recommendations of the *Geotechnical Assessment Report for Subdivision – Waikanae North Subdivision* (CGW, 22 October 2025).

b) Any subsequent site-specific geotechnical reports prepared under Condition 2; and

c) The relevant provisions of NZS 3604:2011, MBIE Technical Categories, and NZS 1170.5:2004.

Note: A consent notice under Section 221 of the Resource Management Act 1991 shall be registered on the title of each affected lot to ensure ongoing compliance with this condition.

17.5 Slope Stability and Drainage

In areas identified as having slope stability risk (particularly within dune terrain), the consent holder shall ensure that:

a) Final lot layouts, earthworks batters, and drainage systems are designed and constructed under the supervision of a geotechnical professional.
b) Permanent slope cuts steeper than 3H:1V or taller than 1.5 m are supported by a site-specific slope stability analysis; and
c) Subsoil drainage and surface water management are installed to maintain long-term stability, as recommended in the geotechnical report.

17.6 Peat Management and Reuse

Where peat is excavated, its handling, disposal, or reuse shall be managed in accordance with the recommendations in Section 16.3–16.5 of the Geotechnical Report. Any proposed reuse of peat as engineered fill must be supported by laboratory testing and approved by the geotechnical professional.

17.7 Slope Stability and Drainage

All geotechnical aspects of bulk fill earthworks, foundation construction, and slope stabilisation shall be observed and certified by a geotechnical professional engaged by the consent holder. No engineering certification (including Geotechnical Completion Reports) shall be provided by CGW Consulting Engineers or other geotechnical practitioners unless adequate construction-phase observations have been carried out.

18. Construction Considerations

18.1 Earthworks and Retaining

All earthworks to be carried out on the site are to be in accordance with the requirements of NZS 4431:2022 Code of Practice for Earth Filling for Residential Development. All unsuitable materials (i.e. non-engineered fill, vegetation, tree roots/stumps, topsoil, organics, detritus material, demolished building foundations, and decommissioned services and their associated backfilled trenches) are to be stripped away from areas of earthworks and stockpiled clear of the operational area or carted off-site. The site is to be inspected by an experienced Geo-professional to ensure all unsuitable materials have been removed prior to fill placement.

Temporary excavations of up to 0.5 m in depth may generally be cut vertically, while deeper excavations may require temporary support and/or battering to maintain stability during construction. These recommendations are provided for situations where excavations are clear of site boundaries, and surcharges from existing pavements / buildings. In these situations, staged excavations or top-down constructed temporary retaining may be required. It must be understood, maintenance of excavation stability is the responsibility of the earthworks contractor, and these recommendations are provided as guidance only. Prior to commencing earthworks, a sediment control plan should be compiled to ensure that the Council requirements are met, particularly with respect to wind erosion of exposed and stockpiled soil materials.

Depending on the location on site, fill should be kept to no more than 500 mm above current ground level without referring the matter back to us. Fill should comprise materials approved by the Geo-professional and be compacted in 200 mm layers to achieve no less than 95% maximum dry density. Nuclear densometer testing is to be conducted on every second layer of fill, to be approved by the Geo-professional. The edges of any fill should be either battered at 2H: 1V or be suitably retained. Retaining of re-compacted site soil fill can be designed utilising a friction angle of 27° and unit weight of 18kN/m^3 to determine active earth pressures, while lateral load resistance should be calculated for embedment into suitable natural soils for a soil friction angle of 27° and unit weight of 18kN/m^3, or Cu=50 kPa if required for short term loading analysis.

18.2 Permanent Cut Batters

We consider any permanent slope cuts, battered to 1.5H:1V (to a maximum height of 1.5 m) will be generally stable. Permanent batter slopes of 3H:1V are expected to achieve adequate long-term stability, subject to confirmation of material strength and drainage performance during detailed design.

Heights greater than 1.5 m and steeper than 3H:1V will need to be assessed and analysed via slope stability analysis by a suitably qualified geo-professional. Consideration should be given to the cut/fill batter faces being hydroseeded, covered with geotextile cloth (suitable for protecting cut faces) or other additional support, if deemed necessary.

Any fill and/or backfill should be placed in maximum 200 mm-thick, loose lifts and compacted to the design specifications prior to placement of the next lift. The required compaction level, in accordance with NZS4431:2022, is minimum 95% standard maximum dry density (MDD). If required, additional imported fill materials will need to be tested for geotechnical suitability/compaction criteria and be approved by CGW prior to use. If compacted appropriately, fill batter edges should be more than capable of being stable at 1.5H:1V slope angle.

In-place density tests should be performed at appropriate spacings and lift intervals utilizing a Clegg Hammer or Nuclear Density Meter (NDM).

18.3 Pavements

We recommend a preliminary design CBR value of 3% for flexible or ridged pavements at the site. The thickness of the basecourse would depend on the final CBR used for the subgrade and the anticipated loading. The compaction of the basecourse should be carried out with a non-vibratory roller of appropriate static weight and energy. We recommend the placement of geotextile between the subgrade and basecourse (i.e. Bidim A19 or equivalent). This is to reduce the mixing of the natural soil with the engineered fill and will increase the performance of the basecourse. Additionally, high strength geogrid may be required to reinforce the base of any underlying granular fill below pavements within the east and south of the site.

Any vegetation, organic or deleterious material, including topsoil and non-engineered fill should be removed from under pavement areas prior to aggregate placement. The subgrade should be subject to inspection by an experienced Geo-professional who is familiar with the findings of this report. Stormwater Control & Underground Services

All concentrated stormwater from impermeable areas should be collected, conveyed, and discharged via appropriately designed and certified soakage systems. Stormwater management must be integrated with the subdivision's overall drainage design to ensure discharge rates and locations are compatible with underlying soil infiltration characteristics and groundwater levels.

Any uncontrolled stormwater must not be permitted to infiltrate or saturate ground near structures, as this may adversely affect the performance of foundations and pavements. Roof water and paved-surface runoff should be directed to sealed conveyance systems or approved soakage pits located clear of building platforms and retaining structures.

Soakage design should be confirmed at the detailed design stage based on local permeability testing, depth to groundwater, and stormwater modelling to ensure adequate capacity and setback from foundations.

19. Further Geotechnical Involvement

19.1 Earthworks Specification

During the detailed design a Geo-professional familiar with the findings of this report should be engaged to provide an earthworks specifications report for the proposed subdivision. This should additionally comprise a review of the earthwork's drawings (cut and fill plans) and confirmation that they follow the recommendations of this geotechnical report. Further sampling and laboratory testing will be required once the proposed earthworks sequencing has been finalised.

19.2 Construction Observations

A Geo-professional familiar with the findings of this report should be engaged to carry out observations during subdivision earthworks to confirm our above recommendations are being wholly followed. Of note is that:

- Inspections will not be carried out prior to the issue of Council Resource and/or Building Consents; unconsented works will not be inspected.
- We recommend that once received, the Consent be forwarded and reviewed by us. Following the Consent review a schedule of inspections can be issued to the Client.
- Without sufficient observations during the foundation construction phases, CGW Consulting Engineers will not be able to provide engineering certification (i.e. Geotechnical Completion Report; GCR).

19.3 Lot Specific Geotechnical Investigations

Lot specific geotechnical investigations will be required following subdivision earthworks to confirm the requirements for individual dwelling development within each new lot. Subject to the findings of the lot specific testing and reports, geotechnical inspections at the time of individual foundation excavations may also be required.

The findings and recommendations of the subdivision GCR or lot specific investigation reports may supersede those given herein once the geotechnical conditions of individual areas are further defined.

20. References

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