23 July 2026

Stillhaven Limited
Attn: Caleb Pearson
110 Carlton Gore Road
Auckland 1023

Dear Caleb,

Preliminary Geotechnical Assessment – Stillhaven Development, Te Kowhai, Waikato

(Our Reference: 29937.000.001_06)

1 Introduction

ENGEO Ltd was requested by Stillhaven Limited to undertake a preliminary geotechnical assessment of five rural properties to the east of Te Kowhai, Waikato.

The Stillhaven development encompasses the following five properties where shown on the attached Geotechnical Risks Plan (Appendix 1), which we understand are currently zoned HPL-LUC2:

- 648 Te Kowhai Road, 21.5 ha.
- 570 A Te Kowhai Road, 40 ha.
- 509B Horotiu Road, 17.5 ha
- 493 Horotiu Road, 7.4 ha
- 463B Horotiu Road, 17.6 ha

This report provides a preliminary geotechnical assessment of ground conditions and comment on key potential geotechnical risks and site constraints based on our local knowledge, a site wide walkover and desktop assessment.

Further geotechnical investigation, analysis, design and reporting will be necessary to support the development through the substantive application of a Fast Track approval.

2 Scope of Assessment

The scope of the ENGEO geotechnical assessment comprises:

- A review of pertinent natural hazards that may affect the site and proposed development.
- A review of relevant geological maps and topographical maps.

- Discussion of potential geotechnical related development issues within the study area based on the results of the desk study, a site walkover, geomorphological mapping and ENGEO experience with similar sites.
- Discussion on general site suitability for managing development generated stormwater and wastewater by discharge to ground.

3 Proposed Development

Stillhaven is proposed to be a mixed-use development on approximately 104 hectares of land east of Te Kowhai within the Waikato District. A consultation document has been prepared by Barker and Associates Limited which outlines the proposed development concept, referenced 026304 dated July 2026.

The project is intended to be progressed through the Fast-track Approvals Act and will require land use, subdivision and regional resource consents from Waikato District Council and Waikato Regional Council. The development is proposed to integrate with Council's reticulated infrastructure networks, supporting the efficient delivery of services and long-term growth for the area.

Stillhaven propose to deliver over 700 new homes, approximately 200-unit retirement village with supporting facilities, approximately 1,500 m² of commercial floor space across two centres, and an integrated network of open space, stormwater infrastructure and ecological enhancements.

The development is proposed to be designed around a connected transport network, diverse housing typologies, active transport links and enhanced waterways, creating a well-connected and sustainable community. The development is likely to be staged to align with infrastructure delivery and market demand over time.

Stormwater infrastructure is proposed to utilise existing east-west drainage networks to convey concentrated stormwater flows generated within developed areas towards eight stormwater basins located on the periphery of the development areas, ranging in size from 0.4 Ha to 2.0 Ha. Discharge from these attenuation systems is proposed to be conveyed to existing networks beyond the development area at not greater than pre-development rates or volumes.

Wastewater is proposed to be fully reticulated within the development and connect to a new wastewater main under construction on Horotiu Road.

4 Development Area Review

4.1 General

The development covers approximately 104 Hectares of predominately rural farmland, collectively referred to herein as ‘site’. The site is located to the east of Te Kowhai village, between Horotiu Road to the north and Te Kowhai Road to the south. It is bound further to the east by rural properties.

The landform is level to very subtly undulating between 25 and 27 m RL (NZVD2016 Datum) with several open drains and streams evident within and adjacent to the site suggesting perched or elevated groundwater conditions may be present. A small surface water feature is evident within the central portion of 509B Horotiu Road.

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The subject site appears to be used primarily as pastoral farmland, containing cropping, stock fences, farm tracks, dwellings and other ancillary buildings typical of farming operations.

4.2 Geomorphology

The topography across the site can be broadly described as level to very gently undulating and low lying, located approximately 5 km west of the Waikato River and 3 km east of the Waipa River, the two major river systems of the area.

An incised tributary gully system of the Waipa River is located approximately 100 metres south of the study area and appears to contain a stream that flows from east to west. Its invert is approximately five to seven metres lower than the subject site.

4.3 Geology

A geological map of the site, sourced from Earth Sciences NZ Interactive Web Map[1], is shown in Figure 1 below. It indicates the site is underlain by a gently north-westward sloping plain (Hinuera Surface) of alluvial sediments deposited by the ancestral Waikato River. The majority of the site is shown to be underlain by Piako Subgroup, with the southern part of the site underlain by the older Hinuera Formation.

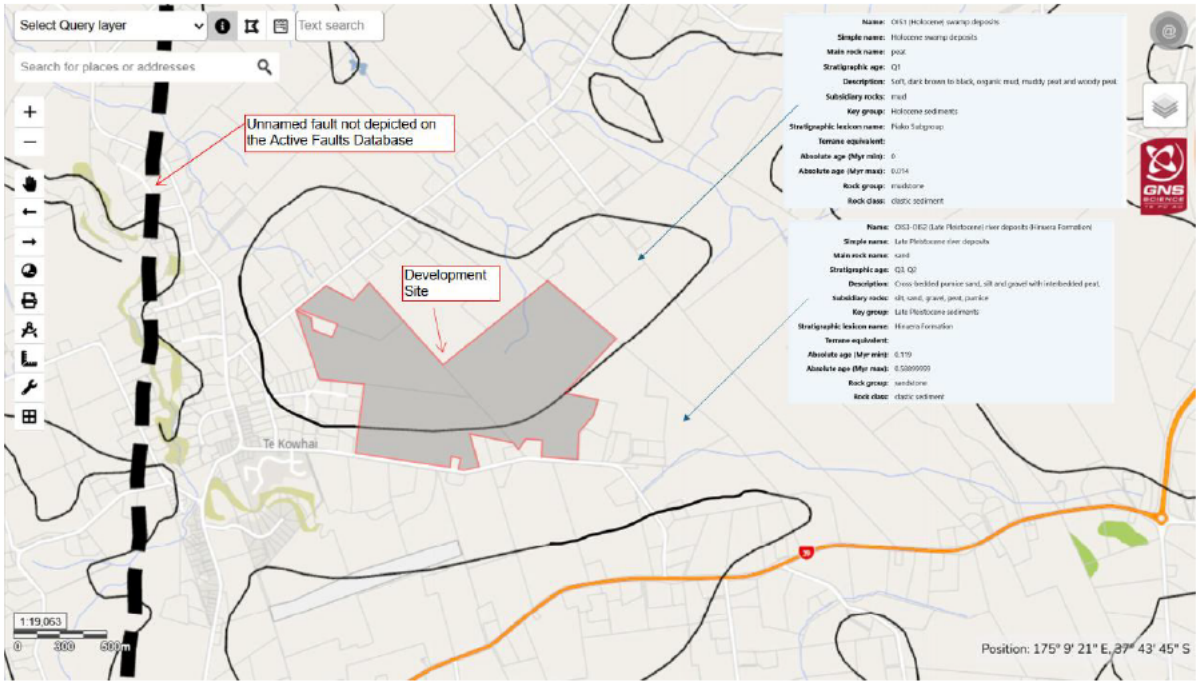


Figure 1: Mapped Geology, Earth Sciences NZ

The Piako Subgroup accumulated during the last 10 to 12 thousand years and is described as Holocene swamp deposits comprising soft, dark brown to black, organic, mud, muddy peat and organic peat.

[1] https://data.gns.cri.nz/geology/

The Hinuera Formation is an older alluvial sediment deposited between 50 and 170 thousand years ago by an ancient braided river system. It consists of variably pumiceous and rhyolitic gravel, sand and silt. Young surface and old subsurface peat and organic silt (lacustrine) deposits can be present. There is distinct cross-bedding throughout this formation, and soil conditions can be highly variable. In areas with poor drainage, peat swamps have formed. These swamps (and associated lakes) are understood to have formed by the Hinuera Formation sediments damming valleys within rolling hill topography. Some of the valleys are broad, forming large areas of peat, for example the Driver Road valley (Kainui bog) to the east of Ngaruawahia as well as several smaller peat bogs present in the Te Kowhai area.

Undifferentiated alluvial and colluvial deposits are expected to have accumulated in the incised tributary gully to the south, beyond Te Kowhai Road.

4.4 Existing Subsurface Information

The New Zealand Geotechnical Database[2] (NZGD) does not show existing geotechnical investigation data within the subject site. It does show shallow hand auger borehole investigations or deeper cone penetration testing (CPT) have been undertaken in several nearby sites.

Review of this investigation data suggests 20 to 30 m of very loose to medium dense interbedded silts and sands (CPT tip resistance (qc) of typically < 1 to 10 MPa). There were no distinct or obvious trends in terms of lateral variability or density and / or soil type with depth.

They also suggest a shallow water table at approximately 1.5 to 2.0 m depth below ground level.

4.5 Council Hazard Maps

4.5.1 Flooding

The Waikato Regional Hazards Portal includes flood modelling from the Waipa and Waikato River catchments for 1% AEP flooding. This modelling includes the tributaries which lead into the Waipa River within the western portion of Te Kowhai township. Modelling suggests that the site is outside the flood extent from these catchments. The drainage systems and stream to the south of the site are not identified as floodable under this in this hazard portal.

4.5.2 Earthquake and Landslides

The Waikato Regional Hazards Portal outlines areas of active faulting, previous significant earthquakes, and regional liquefaction categories. Earth Sciences NZ also provides active faulting and significant mapped faulting

The site and surrounding Te Kowhai township have no active or significant mapped faulting. No significant earthquakes have been recorded historically. The site and surrounding area is mapped regionally to have possible liquefaction risks.

4.6 Historic Aerial Photography Review

Aerial photographs dating from 1941 to 2026 were reviewed during preparation of this report. The aerial photographs were sourced from Retrolens and Google Earth Pro.

Photos from the 1940's confirm that Horotiu Road and Te Kowhai Road were present at this time with the site being largely undeveloped as a series of paddocks with several small buildings.

[2] https://nzgd.org.nz/

Construction of the Te Kowhai airfield occurred in the 1960's south of the site.

Aerial photos and mapping were unavailable for review between 1970 and 1990. Based on review of earlier and latter aerial photographs it appears that the majority of the other farm buildings and farm houses were constructed during this time.

A residential subdivision was formed in the 2010's at 556 Te Kowhai Road on the southwest of the subject site, with construction of residential dwellings undertaken throughout this decade.

Aerial photography reviews noted previous earthworks and the potential for farm burial pits within the entrance to 648 Te Kowhai Road to the farm shed area. The extent of which is indicated on our Geotechnical Risks Plan (Appendix 1).

5 Site Walkover

An ENGEO geo-professional conducted a walkover of the development area on 2 June 2026. The following site observations are relevant to proposed development.

The sites topology and geomorphology aligns with the desktop review, being gently undulating farmland with land drainage features in the form of farm ditches separating large paddocks. The site visit was undertaken in early winter following several weeks of heavy rain and drainage ditches appeared to be functioning and carrying the majority of concentrated stormwater flows from the paddocks towards lower areas of the site and wider gully features.

Recently excavated drainage ditches and farm races were exposed in several areas of the site which confirmed the near surface ground conditions in those locations. Topsoil thickness appears to be of the order of 200 mm to 500 mm thick, underlain by silty clay alluvial soils which align with the Piako Subgroup. Groundwater levels were not able to be confirmed from our walkover, however open drains and localised depressions across the paddocks included reeds and similar vegetation indicating either elevated permanent groundwater conditions or perched groundwater within the upper stratum.

Farm offal holes were noted in several locations, some of which were locally excavated at least 1.5 m below current ground level. Historic earthworks within 648 Te Kowhai Road were not evident as part of the walkover. Unmapped features may exist across the site that were not identified during our walkover.

Photos from our walkover noting some of these features are provided below.

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Photo 1: Ponding surface water from recent rain, approx. centre of the development area



Photo 2: Drainage ditches operating as intended following recent rain. 684 Te Kowhai Road.



Photo 3: Recently excavated drainage ditch exposing silty clay subgrade and shallow topsoil. 648 Te Kowhai Road.



Photo 4: Natural soils, Inferred Piako Subgroup exposed in drainage ditches. 648 Te Kowhai Road.



Photo 5: Farm Pit, 509 Horotiu Road

Figure 2: Site Walkover Photographs

6 Geohazards

6.1 Introduction

Geotechnical constraints are constraints associated with geological hazards (geohazards) that have the potential to impact the development of land if the geohazard is not addressed. A geohazard is a broad term that refers to geological hazards caused by a combination of the structure, type, strength and topography of the land.

The main geohazards considered relevant to the study area include:

- Active Faults
- Slope instability
- Static Settlement (settlement, primary consolidation, secondary consolidation (soil creep))
- Liquefaction
- Lateral Spreading
- Poor ground conditions (elevated groundwater, undocumented fills, bearing capacity, lateral load and retaining structure issues, infrastructure vulnerability)

These hazards are further discussed in the following sections.

6.2 Active Faults

With reference to the Active Faults database[3] held by Earth Sciences NZ, the closest faults that are currently identified are the Turi Fault, Wairoa Fault and the Kerepehi Fault.

The Turi Fault is located off the west coast of the Waikato, approximately 90 km southwest of Te Kowhai, and is estimated to be capable of generating a M7.2 earthquake approximately every 1,600 years.

The Wairoa Fault (60 km north of Te Kowhai) and the Kerepehi Fault (40 km west of Te Kowhai) are estimated to be capable of generating M6.6 earthquakes, with return periods of 22,000 and 2,500 years respectively.

These faults are sufficiently distant to the site such that specific mitigation measures in addition to typical liquefaction / lateral spreading analysis are not considered necessary.

6.3 Slope Stability

The incised gully feature to the south of the site is greater than 100 m away and the risk of associated slope instability significantly impacting the subject site due to this feature is considered to be low. This 100 m offset is depicted on the attached ENGEO drawing.

[3] New Zealand Active Faults Database - Earth Sciences New Zealand | GNS Science | Te Pū Ao

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Given the site gradient and distance to this slope, global slope stability is not considered to be a significant risk provided that cut and fill batters within the development are engineer designed and appropriate setbacks are established from internal open drains (or the drains are infilled). Where open drains are in proximity to site boundaries, specific assessment of their long-term stability will need to be considered with respect to the proposed landform and stormwater applications in these areas.

6.4 Liquefaction

Soil liquefaction results from the loss of strength during cyclic loading, such as those imposed by earthquakes. Soils most susceptible to liquefaction are geologically young, clean, loose, saturated, uniformly graded, cohesionless materials. Typically soils within the Piako Subgroup and Hinuera Formation are prone to liquefaction where below the groundwater table, as low density / strength sands and silty sands are common in these geological units.

When soil liquefaction occurs, there is a significant decrease in the soil strength. While this is a temporary situation, foundation bearing capacity failure can occur if the liquefied soil is sufficiently close to the foundations.

Preliminary assessment of liquefaction risks has been undertaken using nearby CPT data sourced from the NZGD, using Boulanger & Idriss (2014) methods, adopting the *in situ* groundwater level during the earthquake event and considered free-field settlement assessments according to Zhang et al. The selection of CPT tests for this assessment was limited to several tests undertaken surrounding the nearby Te Kowhai Airfield, within approximately 500 m of the site.

Analysis was undertaken for the full depth of CPT traces and for 10 m depth to determine how the landform may be categorised in terms of residential zoning performance outlined in MBIE guidance developed following the Christchurch Earthquakes. The analysis considered a 25 year Serviceability Limit State (SLS), 100 year Intermediate Limit State (ILS) and 500 year Ultimate Limit State (ULS) events.

The results are summarised in Table 1 below and are for ground conditions at the Te Kowhai Airfield. As these are a nearby site and not within the study area, the actual results could vary due to lateral and vertical variability in ground conditions. As such, they are indicative and will need to be verified by site specific site investigation and analysis as part of the Substantive Application.

Table 1: Results of Preliminary Liquefaction Analysis

Scenario	PGA	Mw	LSN	LPI	Damage Category	Liquefaction Induced Settlement (mm)
SLS	0.06	5.8	0	0	L0 Insignificant	< 5 mm
ILS (Full Depth)	0.12	5.9	4 to 8	0	L1 Mild	25 to 80 mm
ILS (10 m depth)	0.12	5.9	4 to 8	0	L1 Mild	15 to 20 mm
ULS (Full Depth)	0.25	5.9	40 – 50	14 – 17	L4 Severe	160 to 320 mm
ULS (10 m depth)	0.25	5.9	40 - 50	14 - 17	L4 Severe	100 to 150 mm

Notes
1. PGA = Peak ground Acceleration; Mw = Magnitude; LSN = Liquefaction Severity Number; LPI = Liquefaction Potential Index

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Discussion on these results is provided in the Geotechnical Considerations section below.

6.5 Lateral Spreading

Liquefied material adjacent to a free-face, such as a terrace, gully or river, can allow non-liquefied material above it to move towards the free-face (lateral spreading).

Lateral spreading can result in ground displacement and cracks developing at significant distance from slopes. Lateral spreading is particularly damaging to buildings, roads and services with the zone of displacement. Buried services such as pipes, manholes, tanks and pump stations can become buoyant resulting in significant damage to infrastructure. This can also be worsened by total and differential ground settlement. Surface infrastructure such as roads, ponds, bridges / crossings and pump stations can also be damaged by ground settlement and lateral spreading.

As noted above, this feature is greater than 100 m away from the subject site and associated risk is therefore considered to be low.

The concept development scheme provided as part of the referral notes that the existing east-west drainage ditches will be used to convey stormwater into eight stormwater ponds or attenuation basins located on the periphery of the development area. Modification of the existing shallow drainage ditches is expected to be required as part of the development works to achieve desired flow rates and fall towards these stormwater attenuation features.

Lateral spreading associated with significant earthquake events into these proposed drainage features will require specific investigation and assessment. Based on the depth of the farm drainage features, we consider a 50 m buffer zone where lateral spreading risks may occur adjacent to these features. Likely remediation measures for future development are outlined in Section 7 below.

6.6 Static Settlement

Settlement and consolidation can occur where subsoils are subjected to a load greater than they have previously experienced. Settlement is relatively rapid and occurs in granular soils whereas consolidation is a longer duration process in generally fine grained soils.

In normally or near-normally consolidated subsoils, such as those expected under the site, this can result in significant total and differential settlement beyond the tolerance of normal residential building construction, pavements and underground services (essentially any structures or infrastructure).

Exposed areas of the site during our walkover indicated that topsoil thickness was of the order of 500 mm where observed. No evidence of significant organic peat soils were observed however, the Piako Subgroup typically includes organic rich, loose and low strength unconsolidated silts and sands that are prone to consolidation effects on loading.

Assessment of consolidation effects is difficult without site specific geotechnical investigations to characterise the thickness, stiffness and consistency of these soils. However, in general consolidation effects on granular dominant soils will typically react in an elastic manner, with settlement occurring relatively quickly upon loading.

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Where low strength alluvium, and peat soils are present, consolidation effects involve both primary consolidation where pore water pressures need to be relieved for volume change to occur (a process that can often take some time due to the permeability of the soil), and ongoing secondary consolidation where the soils particles are realigning under plastic deformation from the constant load imposed. Secondary consolidation can often take considerable time for the soils to reach an equilibrium with the impose load state (years) and are highly affected by prior stress history. Magnitudes of secondary consolidation can be reduced by preloading.

It is typically more cost effective and resilient to future development to undercut near surface organic / peat soils that are less than 2 m depth and preload areas where these soils are thicker or are underlain by a significant natural crust of less compressible soils.

6.7 Ground Conditions for Building Development

6.7.1 Non-Engineered Fill

Our site walkover and aerial photo interpretation indicated the presence of localised farm dump and waste stockpiles within the southern portion of 648 Te Kowhai Road, near the existing site entrance and within the centre of 570A Te Kowhai Road adjacent to an existing farm shed, and a small stockpile in the northern portion of 463B Horotiu Road. Previous earthworks were also indicated near the entrance to the farm at 648 Te Kowhai Road. The depth and extent of these features have not been determined as part of our work. It is likely that farm dump areas are likely to exist beyond these two locations.

Any waste materials associated with farm dumps are not suitable to remain in place as part of future development works. While not confirmed during our walkover or desktop study, undercutting of any undocumented fill encountered during future earthworks is recommended to provide consistent bearing and reduce development risks. Environmental investigations, sampling and assessment of these materials will be required as part of any future works to confirm contaminants and suitable locations for disposal.

6.7.2 Bearing Capacity Risks

Proposed structures and infrastructure are likely to be located within close proximity to the water table where poor founding conditions are present and where liquefaction is expected to trigger due to an absence of a non-liquefiable crust to suppress the effects of liquefaction.

Structure and infrastructure design will therefore need to consider low bearing capacity requirements as well as liquefaction effects including loss of bearing, liquefaction induced settlements (total and differential) and the risk of buoyancy and liquefaction ejecta which can result in significant total and differential settlement. Infrastructure resilience may be considered through redundancy design in the piped reticulation networks.

Discussions on foundation considerations from this risk are outlined in Section 7 below.

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7 Geotechnical Considerations

Based on the inferred ground model, near surface conditions exposed during our walkover and geotechnical hazards identified above, the following engineering considerations and will need to be assessed following site -specific geotechnical investigations and during the development of the site as follows:

7.1 Consolidation and Preloading Recommendations

Based on anticipated ground conditions including the likely presence of Holocene aged low strength alluvium, filling and development of building platforms will need careful consideration to manage total and differential settlements prior to subdivision infrastructure and building development. Where low strength alluvial soils are present at thicknesses in excess of 2.0 m, surcharging / preloading fills are typically the most cost-effective approach.

A specific preload design would need to be carried out; however, we anticipate a temporary surcharge of 2 to 3 m will be required to achieve 90% primary consolidation and although difficult to predict will likely take between six to 12 months to occur.

The location of the preload shall consider the distance to any existing structures, infrastructure, or nearby properties to ensure they are not adversely affected by preload settlements. Based on experience, ground deformation reduces significantly at a horizontal offset equal to the soft soil thickness beyond the toe of the fill embankment.

For feasibility purposes a 10 to 15 m setback should be adopted from site boundaries. Stormwater attenuation features proposed on the fringes of development areas do not need to be preloaded.

Preloading designs undertaken in close proximity to boundaries will need assessment to the lateral extent of settlement beyond the development extents as part of the assessment of effects during the substantive application process.

7.2 Earthworks and Settlement Monitoring Recommendations

All earthworks must be carried out in general accordance with the requirements of NZS 4431 and the Regional Infrastructure Technical Specification (RITS) under periodic observations by an appropriately qualified and experienced Chartered Professional Engineer. Specific earthworks recommendations would be provided in a Geotechnical Investigation Report prepared to support the resource consent application.

Several localised areas of the site noted the presence of undocumented filling, waste concrete / construction debris and buried farm dumps. These materials will require undercut and removal as part of any future earthworks activities at the site prior to fill placement, building foundation construction and / or civil infrastructure works (including pond / swale construction).

Earthfill quality control testing of the structural fill raft will be required during construction. The source of the fill should be discussed with and approved by the Chartered Professional Engineer to verify its appropriateness and quality control testing requirements.

Settlement monitoring requirements such as pin locations and monitoring frequency would need to be confirmed prior to the commencement of the earthworks.

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Ground settlement monitoring will be required to be undertaken by a Licenced Surveyor during earthworks to verify settlement trends. Monitoring frequency would be dependent on the program and the rate of filling. However, is typically in the order of once a week during construction and fortnightly thereafter for the duration of the preload. Following the anticipated surcharge period and ongoing monitoring the preload fill embankment may be trimmed to the designated ground level.

Preload embankments between stages need to be carefully managed as consolidation of consecutive preloads may significantly impact previously preloaded stages. Typically, stage boundaries would dictate an extension to the preloaded area beyond the stage boundary based on the depth of soft soil and height of preload required.

Liquefaction and lateral spreading remediation works may include the use of geogrid reinforced fill embankments to provide resistance to lateral movement against to stormwater ponds, swales and wetland areas where standing water and edge stability risks occur.

7.3 Building Foundation Recommendations

Given the elevated groundwater levels and expected liquefaction risks triggering near surface levels, traditional foundation construction for residential development such as shallow strip and pad foundations are unlikely to be suitable without geotechnical remediation of the underlying ground or a sufficient thickness of engineered fill to provide a non-liquefiable crust.

The use of raft foundation designs, together with shallow ground improvement, either through bulk earthworks fill, reinforced gravel rafts or specifically designed foundation systems to cater for liquefaction induced settlement and lateral spreading risks are likely within this site.

At the completion of bulk earthworks and removal of any preload surcharges (where required) to design subgrade levels upon settlement magnitudes reaching at least t_{90}, it is anticipated that the engineer certified fill exposed at finished levels should provide a geotechnical ultimate bearing pressure of 300 kPa for shallow strip and pad foundations. However, based on the expected magnitude of ULS liquefaction induced settlements the foundation solution for the site may comprise a TC3 foundation solution or a TC2 / TC3 hybrid foundation solution as outlined in the MBIE Canterbury Rebuild Technical Guidance. Consideration should be given to the quantum of fill proposed to meet the requirements below, together with the preload requirements particularly where staged development is considered.

To support future residential building development, the use of Technical Category 2 and 3 shallow building foundations options are considered appropriate based on the results of the preliminary liquefaction risk analysis undertaken above:

- Type G1a Raft: A nominal 2.0 m thick layer of dense engineered aggregate fill provided below finished ground level. This could be comprised of undercutting and / or raising of site levels. The excavation would extend at least 1 m beyond the edge of the building footprint and include two layers of geogrid near the base of the raft.
- Type G1d Raft: A nominal 1.2 m thick layer of dense engineered aggregate fill provided below finished ground level. This could be comprised of undercutting and / or raising of site levels. The excavation would extend at least 1 m beyond the edge of the building footprint and include several layers of geogrid in addition to the G1a Raft basal layers.

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- Type G2a Raft: A nominal 1.2 m thick layer of cement stabilised fill provided below finished ground level. This could be comprised of undercutting and / or raising of site levels. The excavation would extend at least 1 m beyond the edge of the building footprint and include one basal geogrid layer.
- Types G3 & G4: Deep foundation treatment that provides a significantly deep zone of treated materials. These are less cost effective than the shallower methods, but are included as an option where, for example, a heavier than normal house is to be constructed.

7.4 Stormwater Recommendations

The proposed stormwater open swale and pond development intent is considered geotechnically suitable for this site, provided the design of these features consider the risks of liquefaction induced lateral spreading on the adjacent development areas. This can be managed through either ground improvement works to mitigate lateral spreading risks or appropriate setbacks from these free-faces for future building development. Nominal setbacks of the order of 50 m should be considered from free-faces without geotechnical mitigation measures.

The design of pond embankment and drainage swale slopes will need careful consideration where proposed in natural soils, given the likelihood of lower strength soils being exposed. In such areas, we recommend cut batter undercuts to provide suitable strengths to mitigate static and seismic instability under cyclic water level changes.

Soakage to ground stormwater disposal options are not recommended at this site due to high water table and inferred poor ground conditions.

Overland flow generated for large development platforms can provide a significant risk for surficial erosion of subgrades and unsealed surfaces. The soils within this area of the Waikato are prone to internal erosion and tunnel gully erosion (piping failure effects) due to the high pumice and volcaniclastic content of the soils.

Specific design and staging of earthworks, including appropriate short term and long-term erosion and sediment controls will be required. This should include the stabilisation of finished platforms with topsoil cover as soon as possible following earthworks, formation of clean water diversions and sediment traps, topsoil bunds placed above permanent slopes and watercourses following earthworks prior to development of building platforms and ongoing monitoring of ponds and decanting structures during the works.

7.5 Wastewater Recommendations

We understand that the proposed development is to be reticulated to council wastewater treatment plant with recent upgrades to local infrastructure in the area.

7.6 Civil Pavement Recommendations

The very loose to medium dense silty and sandy subsoils present throughout the site are expected to have low subgrade CBR values averaging 1% to 3% *in situ*. Where higher CBR values are required for pavement design, reworking and compaction of the road subgrade during earthworks will be required.

For Engineered Fill, subgrade CBR values may average 3% to 7% *in situ*. We recommend a programme of *in situ* testing is carried out within pavement zones during earthworks construction to provide confirmation of subgrade CBR values and refine pavement designs.

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7.7 Retaining Walls

The development plans are not advanced to the point where earthworks levels and ground retention structures are proposed to support development platforms. However, given likely lower strength alluvial soils and elevated groundwater levels retaining options will need to consider reinforced fill embankments or gravity based systems.

Typical cantilever timber and steel retaining wall systems will need to consider reduced bearing, buoyancy effects from elevated groundwater and liquefaction prone embedment soils which add significant costs to these types of walls where constructed at or near existing ground levels.

8 Uncertainties

This preliminary geotechnical assessment report has been prepared to support a referral for Fast Track Consent. It is based on local knowledge, a site walkover of the development area only and review of site investigations undertaken for nearby sites with those nearby investigations not able to be specifically corroborated with the existing site conditions.

As such it must be accepted that ground conditions could vary from those assumed to exist. Further, the conclusions and recommendations contained herein may be subject to change during subsequent site specific investigation, analysis and reporting.

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9 Assessment Against Section 106 RMA and 2025 NPS

The proposed development is situated on and near land that is subject to natural hazards. Section 106 of the Resource Management Act and the 2025 National Policy Statement (NPS) for Natural Hazards requires a combined assessment of the likelihood of the hazards, the material damage (consequence) of the hazards, and whether the proposed use of the land would accelerate or worsen the hazard.

We have assessed the natural hazards in Table 2 below in accordance with the 2025 NPS for Natural Hazards.

Table 2: 106 RMA and NPS for Natural Hazards 2025 Risk Assessment

Natural Hazard	Hazard Description specific to the site	Likelihood	Consequence	Assessed Risk	Details / Mitigation Measures	Likelihood	Consequence	Assessed Risk	Are the works likely to accelerate or worsen the hazard?
Flooding	We have not specifically assessed the risk of inundation from flooding as these assessments lie outside our area of expertise and has been addressed by others for the development. Existing council hazard mapping does not confirm the extent of flooding for this site and the surrounding land. Further assessments by others will be required as part of the substantive application to confirm the risk and consequence of flooding, with development levels considering these risks.								
Landslip	Instability of slopes both natural and filled causing damage to property and surrounding land	Possible	Major	High	Fill platform and cut natural slopes designed at appropriate slope gradients and with suitable fill material where required including undercutting natural soils to achieve the required factors of safety. Edge stabilisation of embankments adjacent to water courses and permanent ponds to resist lateral spreading risks / seismic slope instability.	Unlikely	Major	Medium	No

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Natural Hazard	Hazard Description specific to the site	Likelihood	Consequence	Assessed Risk	Details / Mitigation Measures	Likelihood	Consequence	Assessed Risk	Are the works likely to accelerate or worsen the hazard?
Erosion	Erosion of soil on fill slopes	Likely	Minor	Medium	Filled batters forming development platform to be formed at suitable slope gradients to mitigate scour. Planting of batters following preloading to be considered. Appropriate erosion measures to be installed around future stormwater swales, culverts, ponds.	Possible	Minor	Medium	No
Active Faults	Rupture of an active fault present within or within proximity to the site.	Very Rare	Major	Medium	Nearest active fault located 30 km away. No specific mitigation measures.	Very Rare	Major	Medium	No

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Natural Hazard	Hazard Description specific to the site	Likelihood	Consequence	Assessed Risk	Details / Mitigation Measures	Likelihood	Consequence	Assessed Risk	Are the works likely to accelerate or worsen the hazard?
Liquefaction	Ground settlement causing damage to structures	Possible	Major	High	Improve crust thickness with engineered fill to mitigate surface manifestation / rupture effects. Preloading of fills to reduce long term primary and secondary settlement risks	Possible	Moderate	Medium	No
	Lateral Spreading causing damage to structures and land	Possible	Major	High	Engineered fill platforms and undercut natural slopes within proximity to permanent free-faces and watercourses. Geogrid reinforcement or geotechnical remediation design may also be considered.	Possible	Moderate	Medium	No
Consolidation Settlement	Static settlement of soft soils beneath infrastructure / future development	Possible	Major	High	Fill and preloading designed for filling over compressible soils. Design of preload to limit long term creep settlement to minimum requirements.	Almost Certain	Minor	Medium	No

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10 Conclusions

We conclude that, although there are risks posed to the site now, the risk to the proposed development from natural hazards will be acceptable once the proposed mitigation measures described in this report and further work outlined in Section 11 below have been implemented. We consider that the proposed works will not accelerate, worsen, or result in material damage to the land so long as the recommendations in this report are followed as intended.

11 Further Work

This report has been prepared as high-level guidance for referral purposes. The above geotechnical comments and recommendations are based on the geological model derived from a review of available existing geotechnical investigations from nearby sites and a limited walkover.

As such, site specific investigations and assessments will be required to support a substantive application through Fast Track.

The following scope of investigation, analysis and assessment are expected to be required prior to Fast Track lodgement:

- A suite of hand auger boreholes and / or machine excavated test pits across the site to observe the near surface subsoils, with associated *in situ* strength testing;
- A suite of cone penetrometer tests (CPT's), with some incorporating shear wave velocity measurements and dMT, to define liquefaction potential and understand modulus parameters for static settlement analyses;
- Correlate investigation results to develop a representative ground model for geotechnical analyses;
- Geotechnical analyses (slope stability, static settlement, liquefaction susceptibility, ground improvement design, earthwork recommendations) for the proposed development;
- Hydrogeological assessment investigation / assessment including targeted investigations to assess and model groundwater response to soakage including installation and monitoring of piezometers within proposed stormwater detention zones. Assessment of effects to proposed development and adjacent private properties;
- Optioneering of specific foundation design and ground improvement options to demonstrate geotechnical suitability for resource consent;
- Assessment of geotechnical and construction effects to adjacent structures and developments; and
- Prepare geotechnical investigation and interpretive report relevant to the development and providing recommendations based on above.

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12 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Stillhaven Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, limited site walkover and previously conducted subsurface investigations beyond the site boundaries that were not undertaken by this consultancy. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground across the subject site has been broadly inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. This Limitation should be read in conjunction with the Engineering NZ / ACENZ Standard Terms of Engagement.

iv. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (07) 777 0209 if you require any further information.

Report prepared by



Matt Packard, CMEngNZ (CPEng)
Principal Geotechnical Engineer

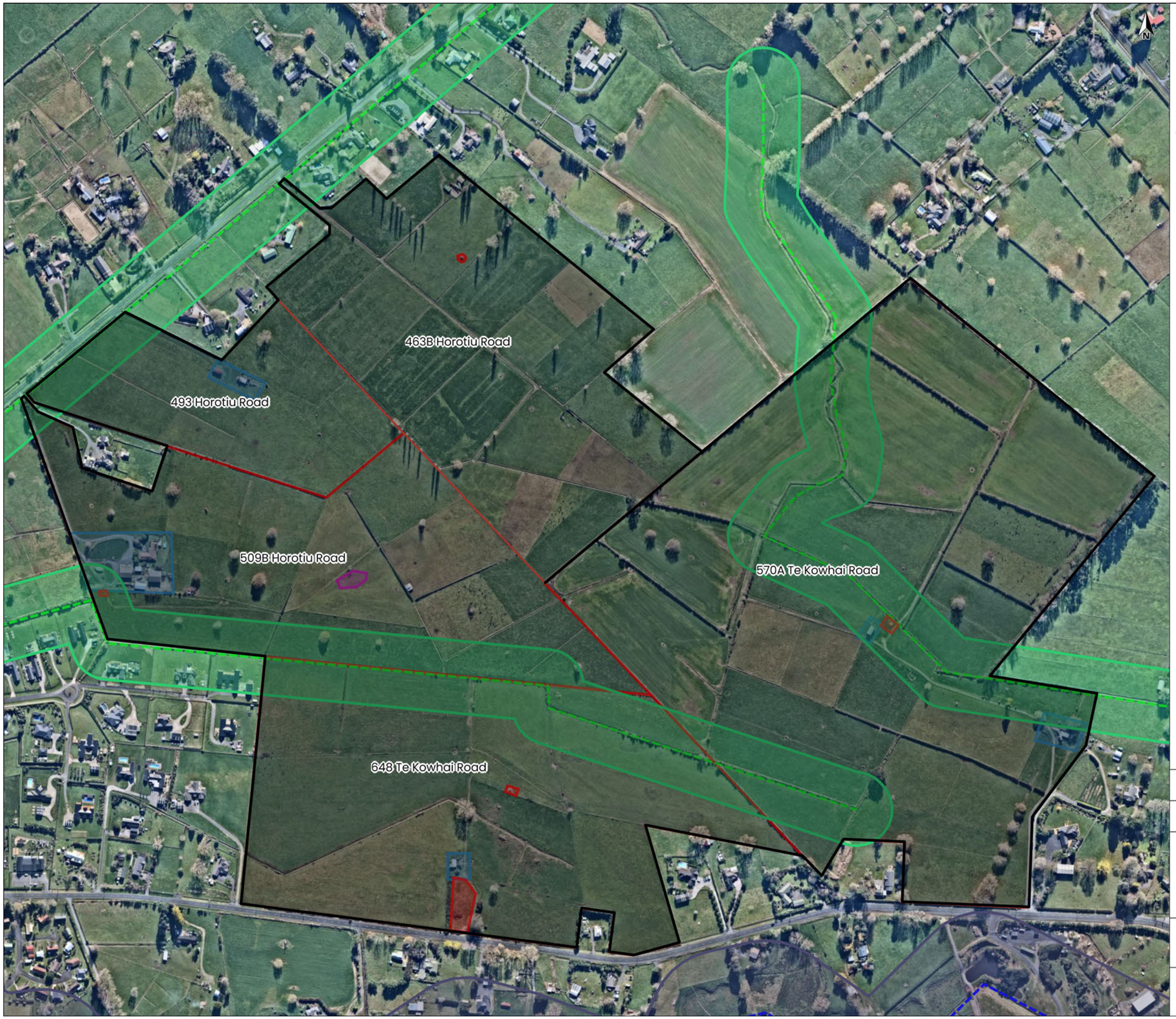
Report reviewed by



Greg Snook, CMEngNZ (PEngGeol)
Associate Engineering Geologist

Appendix 1:

Geotechnical Risks Plan



Legend

- Stillhaven Development Parcel Extent
- 100m setback for lateral spreading from Stream
- 50m setback for lateral spreading from open drains
- Undocumented Fill/Farm Dump Locations
- Developed Areas
- Inferred Open Drains
- Stream
- Surface Water

0 50 m 100 m

© Nearmap

ENGEO

Produced by **Datanest.earth**

Title: Geotechnical Risks Plan		
Client: Stillhaven Limited		Size: A3
Project: Stillhaven Development, Te Kowhai	Drawn: GMS	Appendix No.: 1
Date: 23-06-2026	Checked: MJP	
Proj No: 29937.000.001	Scale: 1:5000	Version: For PGAR