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Brymer Development – Resource Consent / Substantive Fast Track
Application

Whatawhata, Hamilton, New Zealand

GEOTECHNICAL INVESTIGATION REPORT

Maven Ltd

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STATEMENT OF QUALIFICATIONS AND EXPERIENCE

Mark McVinnie

I am an Engineering Geologist at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since March 2022.

I hold the qualifications of BSc(Tech) in Earth Sciences from the University of Waikato, which I completed in 2016.

I have 8 years of professional experience in geotechnical engineering in the Waikato and Auckland regions. My experience has been primarily in land development, infrastructure, and landslide mitigation, and for the last several years have been focused on projects in the Waikato.

I confirm that, in my capacity as author of this report, I have read and abide by the Environment Court of New Zealand's Code of Conduct for Expert Witnesses Practice Note 2023.

David MacPherson

I am a Associate Geotechnical Engineer at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since August 2017.

I hold the qualifications of MEngSc from The University of New South Wales, which I completed in 2022 as well as an MSc from the University of Waikato, which I completed in 2016. I am a Chartered Professional Engineer and a Chartered Member of Engineering New Zealand.

With over 8 years of experience, I am a seasoned geotechnical engineer skilled in project management, design solutions, and team leadership. My background includes extensive work in infrastructure projects. My experience includes managing and participating in site investigations, instrumentation and monitoring, slopes and geotechnical hazard assessments, and slope stability design for major infrastructure projects such as the Peacockes Bridge in Hamilton.

I confirm that, in my capacity as reviewer of this report, I have read and abide by the Environment Court of New Zealand's Code of Conduct for Expert Witnesses Practice Note 2023.

Dave Sullivan

I am a Principal Engineering Geologist at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological sciences services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since May 2025.

I hold the qualifications of BSc in Geological Engineering from Mackay School of Mines at University of Nevada, which I completed in 1997, and a Master of Business Administration from the University of Phoenix, which I completed in 2023. I am a Chartered Professional Engineer, and Chartered Member of Engineering New Zealand. As well as a Category 1 Geoprofessional (TCC), and a Licensed Civil Engineer in California and Nevada, USA.

I have over 30 years of professional experience in engineering geology and geotechnical engineering. My experience has been primarily in land and building development, disaster relief and linear infrastructure, the last few years have been focussed in New Zealand on the Christchurch rebuild and the Waikato Expressway.

I confirm that, in my capacity as CMW's internal reviewer of this report, I have read and abide by the Environment Court of New Zealand's Code of Conduct for Expert Witnesses Practice Note 2023.

EXECUTIVE SUMMARY

This report presents the results of geotechnical investigations and a geohazards assessment to support a fast track substantive/Resource Consent application.

81Ha site located west of the current Hamilton City limits at Whatawhata, Hamilton, New Zealand, which is being considered for the development of approximately 1,043 lots (refer **Figure 1** right). The site is predominantly underlain by Holocene Swamp deposits in the valleys, flanked by rolling hills to the north and south, comprised of Pleistocene aged silts and clays.

The topography is split with low lying river flats in the centre of the site and rolling hills to the north and south ranging in height between RL34 m up to RL50. The regional groundwater table is shallow, usually within 1 metre of the surface. A network of closely spaced drains runs through the centre of the site from east to west.

We consider that the site is suitable for the proposed level of development subject to our geohazard assessment and geotechnical recommendations summarised as follows:

- The low hills are to be modified into a series of terraces and the clay spoil used to fill low lying areas with maximum proposed cut levels of up to 19.5m and proposed fill levels of up to 4.5m
- Fill induced ground settlements within the low-lying swamp deposits are anticipated to be excessive (300-800mm) without mitigation. A preliminary design for surcharging the site to reduce postconstruction settlements to within acceptable limits indicates that a 1 to 4m thick preload will be required for a duration of between 1 to 23 months, though we predict 9 to 12 months is more likely, further laboratory testing is required at design stage to confirm.
- TC2 type foundations are expected to be suitable for the elevated clay and silt units, subject to further laboratory testing at detailed design stage.
- Proposed slopes meet the requisite factors of safety. We recommend maximum gradients of up to 1V:2H for the cut slopes and 1V:3H for the fill slopes in the ridges, and 1V:4H for the low-lying areas are adopted for design. Steeper slopes will require additional retention measures such as geogrid reinforcement, retaining walls or soil nails.
- The potential ULS liquefaction risk is high for the low-lying soils. These effects can be reduced to acceptable levels provided the low-lying parts of the site are sufficiently preloaded and capped with engineer certified clay fill.
- We anticipate a preliminary geotechnical ultimate bearing pressure of 300kPa should be available for elevated areas, in the low-lying areas bearing capacity is reduced to 200kPa where fill is at least 1.5m thick overlying the low strength peat and Piako subgroup soils, subject to the short axis of those footing being no wider than 1.0m. For larger buildings such as the 5 storey apartments, or where fill is less than 1.5m thick in low lying areas, specific assessment will be required at the design stage once building loads are known.

Figure 1: Proposed Development Plan prepared by Barkers and Associates



1.0 INTRODUCTION

CMW Geosciences (CMW) was engaged by Maven Ltd to carry out a geotechnical investigation of an 81Ha site located west of the current Hamilton City limits at Whatawhata, Hamilton, New Zealand, which is being considered for the development of approximately 1,043 lots. The development is comprised of a combination of high-density units/apartments and low density dwellings with associated infrastructure and roading.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced HAM2025-0084AA Rev 1 dated 22 August 2025.

The purpose of this investigation is to carry out field investigation and prepare a Geotechnical Investigation Report suitable to support resource consent/substantive fast track application to Hamilton City Council.

2.0 PREVIOUS REPORTING

A previous geotechnical investigation was undertaken by Tonkin and Taylor, with results presented in their preliminary geotechnical assessment report referenced 1017075, dated July 2021.

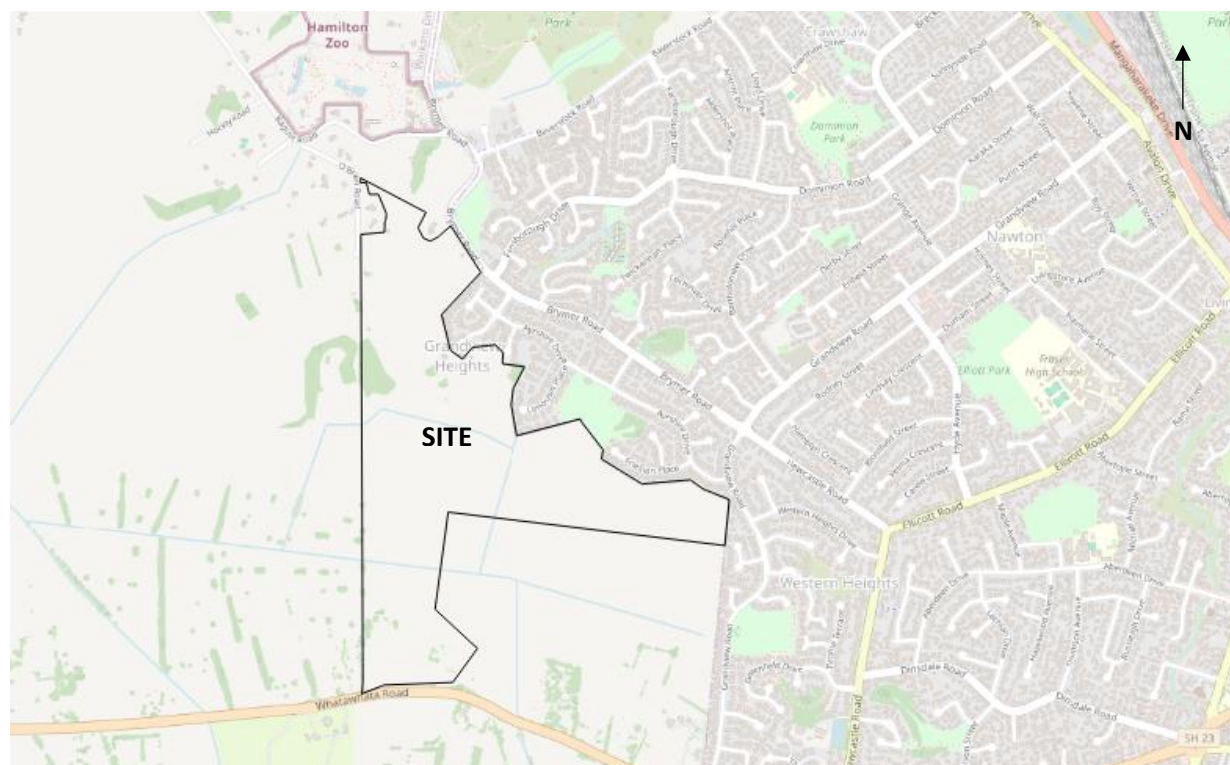
Additionally, CMW prepared a technical memorandum to provide technical details of options for constructing over the extensive peat. These reports are presented in **Appendix G**.

3.0 SITE DETAILS

3.1 Site Description

The site comprises multiple lots under different ownership titles with an area of approximately 81ha. The site is located between Whatawhata and Brymer Roads, as shown in **Figure 2**.

Figure 2: Site Location Plan



Details of the site are as follows: (Street Address, Legal Description)

- O'Brien Road, Rotokauri - Lot 3 Deposited Plan 385271 Brymer Farms Limited
- O'Brien Road, Rotokauri - Lot 1 Deposited Plan South Auckland 87291 Brymer Farms Limited
- Harrogate Place, Grandview Heights - Lot 22 DPS 79526 Brymer Farms Limited
- 584 State Highway 23 Whatawhata - Part Lot 2 DP 18355 Brymer Farms Limited
- State Highway 23 Whatawhata - Allotment 365 Parish of Pukete Brymer Farms Limited

The site is bound to the north by Brymer Road, to the south by Whatawhata Road, and to the east by the Hamilton city limits and the Western Heights suburb. At the time of our 2025 investigation, there are a number of residential dwellings and farm buildings within the site, along with farm raceways, drainage ditches, fencing and troughs. The land use is primarily utilised as farming, with a combination of grazing and crop pastures.

3.2 Landform

The current general landform, together with associated features located within and adjacent to the site is presented in the attached Geomorphology Plan, **Drawing 02** in **Appendix A**.

The site landform can generally be classified into four zones:

- The southern portion of the site, immediately north of Whatawhata Road, comprises an isolated raised knoll, which forms part of the rolling hill topography observed across much of the Hamilton basin. Ground levels vary from approximately RL34 m from road level up to RL50 m (16 m elevation change). The slopes are gentle to moderately steep (up to 40 degrees to the horizontal).
- Within the central portions of the site, the topography is typically level at an elevation of approximately RL25m. A series of farm drains run both north to south, and east to west across the low-lying areas, where a series of culverts and farm bridge crossings provide access.
- The northern portions of the site comprise gully incised, rolling hill topography, which is typical for the area. Elevations change from approximately RL32 m in the low-lying regions, up to RL56 m in the elevated portions of the site, with slope gradients ranging from moderate (approximately 20 degrees) to moderately steep (approximately 50 degrees). A series of scarps and creep faces are present in these hills though they appear to be relict and stable.
- A lower-lying area at the northern tip of the site lies at a relatively level gradient, surrounded by rolling hills to the north, east and south.

3.3 Historical Aerial Imagery

Historical aerial photographs dating back to 1943 show that the site has generally been used for agriculture over the past 80 years, with some development close to the roads. The aerial imagery is presented in **Appendix B** and summarised below:

- 1943: majority of the site is in pasture, with dense patches of vegetation within the gully heads. Some isolated farm buildings are present across the site, with the structures particularly prevalent adjacent to Whatawhata Road within the southern portion of the site.
- 1979: increase in residential development to the east of the site as part of Hamilton City expansion.
- 2008: Increased residential development observed at the northwest boundary of the site.
- 2024: Between February 2022 and February 2024 extra drains were installed in the central low-lying areas.

The site is likely to have undergone minor earthworks as part of historical rural developments, which may comprise earth fill pits, drains, and stockpiles.

4.0 PROPOSED DEVELOPMENT

The Brymer Farms Limited development (Brymer) is a comprehensively planned, multi-staged residential development providing for the urbanisation and subdivision of the project site. The Project is the subject of a Notice of Decision by the Minister, dated 4 September 2025, to refer the Project. It seeks the land use, environmental, subdivision and associated infrastructure consents required to enable the development to be delivered progressively and in a coordinated manner.

The Project will provide approximately 1,043 residential units, including approximately 248 apartment units, across a range of housing typologies and densities. Housing will comprise a mix of terraced, duplex and detached dwellings, conventional and compact residential lots, irregular lots, and eight larger rural-residential lots. This range of typologies is intended to provide housing choice while enabling the development pattern to respond appropriately to the physical characteristics, environmental features and constraints of the site.

The master plan also provides for two mixed-use commercial sites, identified as C1 and C2, which will form a neighbourhood scale commercial component within the development. C1 will provide approximately 910 m² of ground-floor commercial gross floor area (GFA), with 24 two bedroom apartments above. C2 will provide approximately 1,274 m² of ground-floor commercial GFA, with 16 two bedroom apartments above. Both sites will incorporate basement car parking, dedicated vehicle access, servicing areas and landscaping. Collectively, C1 and C2 will provide approximately 2,184 m² of commercial GFA and 40 apartments, supporting the day-to-day needs of residents and contributing to a mixed-use neighbourhood environment.

The development will be supported by the necessary transport and three-waters infrastructure. Water supply and wastewater servicing will connect to the IAWAI reticulated networks, with internal reticulation and associated infrastructure established progressively as development is staged. This may include pump stations, storage, reservoirs and mains, where required. Stormwater will be managed through an integrated stormwater management system, with discharge ultimately occurring to the receiving environment within the Waikato District. The stormwater system will incorporate appropriate treatment, attenuation and conveyance measures and will integrate, where appropriate, with proposed wetlands, watercourses, stormwater reserves and other water-sensitive design features throughout the development.

Implementation of the Project will require staged development related earthworks to establish suitable building platforms, a well connected road network, infrastructure corridors, reserves and other components of the master plan. These works will include site formation, cut and fill, retaining, erosion and sediment control, and other enabling works required to facilitate the staged subdivision and development of the site. The proposal will also require the undergrounding of existing power lines within the Applicant's site.

A significant component of the Project is the provision and enhancement of open space and ecological values. The development will provide an integrated network of public and recreational open spaces, pedestrian connections, wetlands, watercourses, stormwater reserves, landscaped areas and retained vegetation. Environmental and ecological mitigation and offsetting will also be provided where required to address effects arising from the development and to support the protection, restoration and enhancement of ecological values across the site and wider receiving environment. Where appropriate, these measures will be integrated with the development's open-space and stormwater networks.

The overall open-space and movement network will connect with the surrounding urban environment and existing reserves, including Rakautahi Park, Brymer Glen Reserve, Brymer Park and Te Kootii Park. These connections will provide links between residential neighbourhoods, commercial and community areas, recreational spaces and the wider transport network.

Overall, Brymer is a large scale, comprehensively planned urban development that will enable approximately 1,043 residential units, including approximately 248 apartments, together with approximately 2,184 m² of commercial GFA. The Project encompasses the subdivision and development of the land and the associated development related earthworks, transport and access infrastructure, water supply and wastewater connections to the IAWAI reticulated networks, stormwater management and discharge, ecological mitigation and offsetting, landscaping, open space, recreation, and other supporting infrastructure necessary to establish the new community.

The Project will be implemented through a multi-staged consenting and development framework, enabling subdivision, land use and infrastructure works to proceed progressively while ensuring that each stage is appropriately serviced and integrated with the wider development and surrounding environment.

The development will be subject to a comprehensive suite of proposed conditions of consent, including a design guide, developed with input from the Applicant's relevant technical and environmental specialists. These conditions will provide the framework for managing the construction, staging, servicing and operation of the development, including earthworks, transport, three-waters infrastructure, stormwater discharges, ecological effects and offsetting, landscaping and open space, and other potential environmental effects associated with the Project.

Collectively, the master plan (**Appendix C**), supporting technical assessments and proposed conditions of consent provide an integrated framework for the staged delivery and management of Brymer, including measures to avoid, remedy or mitigate adverse environmental effects and, where appropriate, provide environmental offsets.

5.0 INVESTIGATION

5.1 Field Investigation

Following a dial before you dig search and onsite service location, the site investigation was carried out on 2nd to the 6th October 2025 during spring conditions.

The scope of fieldwork completed is shown in Table 1 below:

Table 1: Investigation Summary

Test ID	Test Type	Depth (m bgl)	Methodology
TP25-01, TP25-02, TP25-04 to TP25-16 (15 total)	Test Pit (TP)	3.5 (hole collapse) to 6m	1.2m (w) x 4m (l) holes dug by 19T excavator. Shear vanes completed within and DCP tests carried out alongside where appropriate.
SO25-01 to SO25-06 (6 total)	Falling Head Soakage (SO)	0.3 to 1.6m (encountered shallow groundwater)	90mm dia. Hand auger, slotted pipe installed, falling head test performed
CPT25-01 to CPT25-20 (20 total)	Cone Penetrometer Test (CPT)	6.8 (early refusal) to 20m	GeoMil Panther 100 CPT rig with C10 Cone, drives probe rod to target 20m depth

The geotechnical investigation was carried out following a 'fill-in-the-gaps' methodology to supplement the existing historical data for the site.

Engineering logs for this recent investigation are presented in **Appendix D**. Logs from previous site investigations are presented in the Tonkin and Taylor report presented in **Appendix G**.

The approximate locations of the respective investigation sites referred to above are shown on the Geotechnical Investigation Plan as **Drawing 01** attached in **Appendix A**.

5.2 Laboratory Testing

All testing was scheduled by CMW and carried out by WSP an IANZ registered Testing Authority. Results are summarised in Table 2 and results presented in **Appendix F**.

Table 2: Laboratory Testing Summary

Test ID/ Location	Type of Test	Sample Depth (m bgl)	Results (t/m ³)
TP25-15	Standard Compaction	0.3 – 1.0	1.16
TP25-16		0.3 – 1.0	1.21
TP25-15	Solid Density	0.3 – 1.0	2.7
TP25-16		0.3 – 1.0	2.72

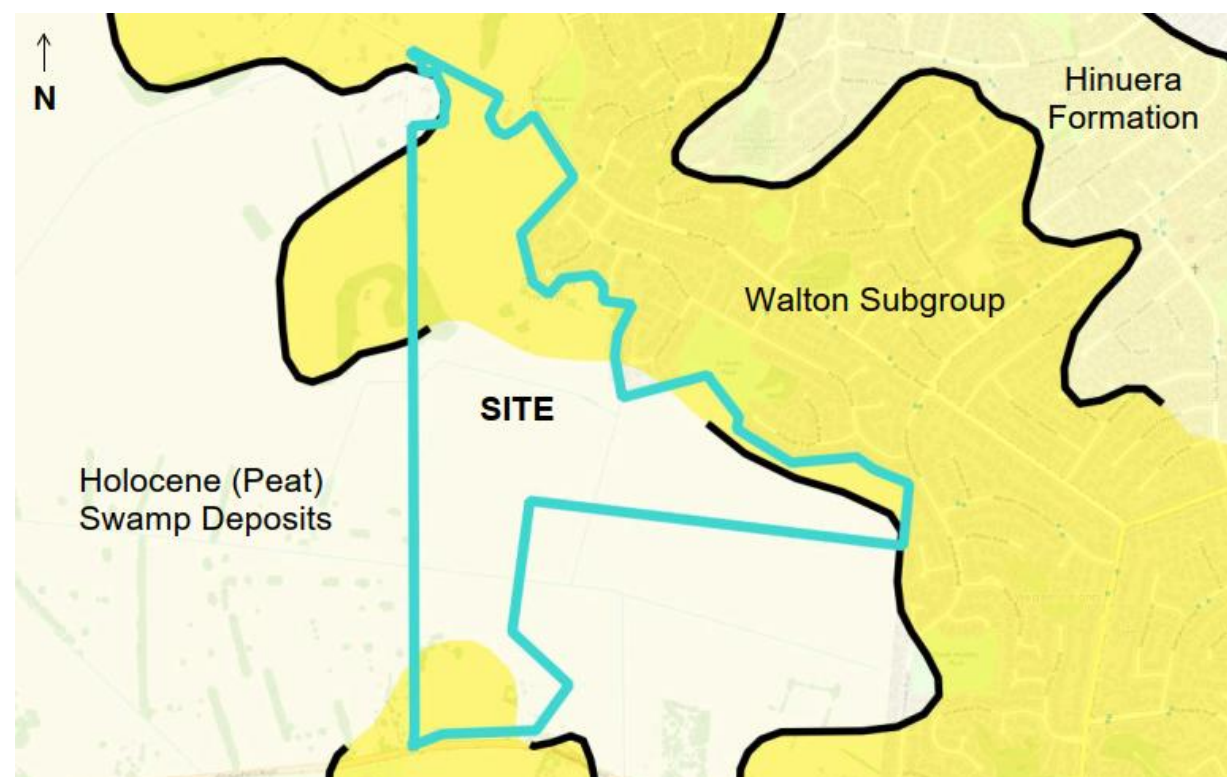
6.0 GROUND MODEL

6.1 Published Geology

The published geological map for the area depicts the regional geology for the low-lying areas as comprising Holocene peat deposits. The northern and southern hills are comprised of Pleistocene aged, fluviially reworked igneous silt/clay of the Walton Subgroup Formation, which is typically overlain by fine grained volcanic ash.

An excerpt from the geological map¹ showing the approximate distribution of the above units is shown on **Figure 3**.

Figure 3: Regional Geology Map Excerpt (GNS Web Map)



¹ Edbrooke, S.W. (compiler) 2005: Geology of the Waikato area: scale 1:250,000, Map No4, Institute of Geological and Nuclear Sciences.

6.2 Stratigraphic Units

The ground conditions encountered during our investigation and inferred from previous investigations are generally consistent with the published geology. Our investigation locations from our current investigation and approximate locations from previous investigations are presented in **Drawing 01**.

The distribution of the various units encountered is presented in the appended Geology Map **Drawing 02** and Geological Sections **Drawing 04** and can be generalised according low lying and elevated areas and to the units below:

Table 3: Summary of Strata Encountered – Low Lying Areas

Geology	Unit Description	Top of Unit (mBGL)		Thickness (m)	
		Min	Max	Min	Max
Colluvium (debris deposits)	Stiff to very stiff Clayey Silt and medium dense Sand	0.2	0.3	0.8	4.3
Peat (Swamp deposits)	Firm to very stiff fibrous organic Peat	0.0	0.2	0.3	2.7
Piako Subgroup	Very soft to very stiff Clayey Silt and medium dense Sand interbedded with minor organic Silt layers	0.2	2.5	0.2	7.5
Hinuera Formation	Loose to medium dense Sand with interbedded stiff to very stiff Silt, and lenses of Organic Silt	0.2	7.2	5	12.0

*Note: encountered in TP25-01 and TP25-05 only

Table 4: Summary of Strata Encountered – Elevated Areas

Geology	Unit Description	Top of Unit (RL m)		Thickness (m)	
		Min	Max	Min	Max
Hamilton Ash	Stiff to hard Clayey Silt and Silty Clay	0.2	0.3	0.5	3.0
Walton Subgroup	Stiff to Hard Clay/Silt and dense to very dense Sand	0.2	0.3	0.9	>25.6

6.3 Groundwater

During the investigation, completed in spring conditions (October 2025), groundwater was encountered at depths ranging from 0m to 4m below existing ground levels in low-lying areas and 0.5 to 3.7m depth in the elevated ridges with a full summary presented below.

Table 5: Groundwater Depths Summary

Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)
BH102	0	CPT25-18	0.5	TP108	4.0
BH103	0	CPT25-19	0.5	TP109	3.2
CPT25-03	0.1	CPT25-20	3.7	TP110	3.1
CPT25-04	0.6	SO25-01	0.3	TP25-01	1.3

Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)
CPT25-07	3.6	SO25-02	0.3	TP25-05	2.0
CPT25-08	1.0	SO25-03	0.2	TP25-07	1.0
CPT25-09	0.4	SO25-04	0.5	TP25-09	2.7
CPT25-10	1.0	SO25-05	0.3	TP25-10	1.4
CPT25-11	0.5	SO25-06	1.5	TP25-11	1.8
CPT25-12	0.4	TP103	2.5	TP25-12	1.2
CPT25-13	0.1	TP104	3.5	TP25-13	1.0
CPT25-14	0.1	TP105	1.2	TP25-14	0.8
CPT25-15	0.2	TP106	3.5	TP25-15	3.1
CPT25-16	0.3	TP107	3.0	TP25-16	1.3

Groundwater was not encountered on several of the investigation locations in the elevated ridges.

6.4 Recommended Geotechnical Parameters

The various geological strata encountered across the site have been rationalised based on geotechnical engineering properties and group into broad units to define a geotechnical model that has been used in subsequent assessment and analysis.

Parameters are based on results of the investigations, known correlations from literature, and our experience with similar soils in the Hamilton area. The geotechnical design parameters adopted are presented in Table 6:

Table 6: Geotechnical Design Parameters

Unit Description	γ (kN/m ³)	c' (kPa)	ϕ' (deg)	S_u (kPa)
Engineered Clay Fill	16	5	30	120
Stiff to very stiff Clayey Silt and medium dense Sand (Colluvium)	16	1	26	60
Firm to very stiff fibrous organic Peat (Swamp Deposits)	16	1	26	40
Firm to very stiff Clayey Silt and medium dense Sand interbedded with minor organic Silt layers (Piako Subgroup)	16	1	26	50
Loose to medium dense Sand with interbedded stiff to very stiff Silt, and lenses of Organic Silt (Hinuera Formation)	17	1	34	-
Stiff to hard Clayey Silt and Silty Clay (Hamilton Ash)	17	5	30	70
Stiff to Hard Clay/Silt and dense to very dense Sand (Walton Subgroup)	16	2	32	140
Notes: γ = soil unit weight (conservative value determined from CPT correlations / typical published values for similar soil types)				

Unit Description	γ (kN/m ³)	c' (kPa)	ϕ' (deg)	S_u (kPa)
c' = effective cohesion (conservative industry accepted value) ϕ' = effective friction angle (conservative industry accepted value/back analysis) S_u = undrained shear strength (lower bound value determined from vane shear testing / CPT correlations)				

7.0 GEOHAZARDS ASSESSMENT

7.1 Geohazard Risk Register

Section 106 of the Resource Management Act² (RMA) requires an assessment of the risk from natural hazards to be carried out when considering the granting of a subdivision consent. S106 RMA specifically states that the assessment must consider the combined effect of the natural hazard likelihood and material damage to land or structures (consequence).

Table 7 presents a summary of the potential geohazards to the site and is based on the information to date. This risk register of the geohazards at the site is a live document that will be updated as the project progresses. A summary of the natural hazard risk classification is provided in **Appendix H**.

Table 7: Geohazard Assessment Summary

Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
1	Earthquake	Seismicity	Whole site	site subsoil class = Class D ULS PGA = 0.25g Importance Level 2	1	5	5	Not applicable	1	5	5
		Fault Rupture	Whole site	The nearest active fault approximately 34km east of the site (Kerepehi Fault). Recurrence interval between 2,000 to 3,500 years, which is outside of the MfE guidelines ³ and not need to be considered further	1	5	5	Mitigation not required	1	5	5
		Liquefaction Damage	Whole site	Refer to Section 7.5	3	4	12	Liquefaction induced settlement and reduced bearing capacity should be accounted for within the foundation design	3	3	9
		Lateral Spread Damage	Swale drains	Refer to Section 7.8	1	5	5	Mitigation not required	1	5	5
		Cyclic Softening	Entire site	Refer to Section 7.6 Predicted to occur in the Walton Subgroup Soils only	1	5	5	Reduced shear strengths have been considered for the slope stability analysis in Section 7.8	1	4	4
2	Slope Instability / Landslide	Global Instability	Whole site	Refer to Section 7.8. Elevated areas show signs of instability, no instability anticipated in low areas except around SW ponds/swales	2	4	8	Slope stability should be confirmed in the design within areas adjacent to the swale drain Proposed maximum slope angle of 1V:3H, our analyses show that this slope angle will be appropriate	2	4	8
3	Problematic Soils	Expansive Soils	Entire site	Refer to Section 7.11. Low-lying soils on the site are considered expansive (Class P AS2870:2011)	3	4	12	Engineered clay soils (Class M) will be used to cap low-lying expansive soils. Details in Section 8.	3	3	9

² Resource Management Act (1991), as at 29 October 2019

³ Planning for Development of Land on or Close to Active Faults, (Ministry for the Environment and The Institute of Geological & Nuclear Sciences), July 2003

Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
4	Concrete Foundation corrosion	Acid Sulphate Soils	Plains	Refer to section 8.4	2	2	4	Initial field testing indicates low risk of concrete corrosion from acid sulphate soils	2	2	4
5	Uncontrolled Fill	Uncontrolled Fill	Entire site	Thin isolated pockets of uncontrolled fill and colluvium were encountered in the previous investigations within the site and may be encountered during earthworks elsewhere on the site.	2	3	6	Undercut and replace materials Consider potential environmental / contaminated land constraints	1	3	3
6	Debris and Existing Improvements	Debris and Existing Improvements	Entire site	Existing foundation elements, burn piles, offal pits, abandoned underground utilities, soak holes and septic systems may be encountered during earthworks	2	3	6	Undercut and replace materials if encountered. Consider potential environmental / contaminated land constraints	1	3	3
7	Settlement	Compressible Soils	Whole site	Compressible soils were encountered on site	5	4	20	Settlement in low lying areas is predicted to be excessive and soil consolidation through preloading will be required prior to construction, as outlined in section 7.10	5	1	5
8	Erosion	Bearing Capacity Failure	Building Platforms	Loose or unsuitable material may be encountered beneath the building platforms	2	3	6	Densify or undercut and replace low strength materials beneath building platforms, use appropriate foundation design	1	3	3
		Soil Erosion	Swale Drains	Some minor erosion is possible to the banks of pond and swale drains during construction	2	3	6	Construction monitoring by geotechnical engineer to ensure good earthworks practices are in place to prevent erosion	1	3	3
9	Groundwater	Groundwater Impacts	Stormwater Controls	Groundwater was encountered at shallow depths below the site. Will need to be considered for: swale design, temporary works dewatering impact, assessment of drawdown effects required, etc.	2	4	8	Stability analysis required at design stage and mitigation measures implemented as required	2	4	8
10	Geothermal activity	Site not within geothermal area									
11	Construction Risks	Excavatability	Building Platforms	Excavations and placement of engineered fill is expected to be readily achieved with normal earthworks plant Drying and conditioning of fill could be needed for earthworks subject	3	3	9	Construction monitoring by geotechnical engineer to ensure good earthworks practices	1	3	3

Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
				to review at detailed design. Without which the soil becomes more wet or saturates over time could reduce the foundation design parameters for fill							
		Sediment Retention Ponds	Building Platforms	Depending on their size, sediment retention ponds may require geotechnical input at design stage to ensure batter stability is achievable	2	3	6	Consideration given to batter stability of proposed ponds at design and construction phase	1	3	3
		Subgrade Preparation	Building Platforms and Road Alignment	Topsoil and existing vegetation within the building footprints and road alignments will be cleared as part of the proposed development earthworks. Saturated, over optimum moisture condition causing pumping and yielding of the subgrade	5	2	10	Undercut to remove topsoil and roots and other materials For saturated subgrade, allow to dry or undercut and replace	1	2	2
		Service Trenches (trench collapse / long term settlement)	Building Platforms and Road Alignment	Trench collapse may occur in surficial soils / if proposed service trenches extend below GW level	3	3	9	Mitigation should be considered in the form of trench support or temporary dewatering such as regularly spaced pumps	1	3	3
		Short term erosion of swale drains	Swale drains	Erosion of swale drains may occur, due to the battered slope gradient on unvegetated slopes	2	3	6	Mitigation may comprise rapid revegetation, erosion control matting. This is to be confirmed in the design phase	1	3	3
		Low road/pavement subgrade strengths	Roads and Paths	Low subgrade strengths and/or saturation of pavement subgrade	3	4	12	Further site investigations and specific pavement design is required	1	4	4

7.2 Seismicity

Seismic loadings for geotechnical earthquake engineering assessment have been obtained from Module 1 of the MBIE/NZGS Geotechnical Engineering Guidelines⁴. The design peak ground acceleration (PGA) and magnitude (M) specified for the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) design cases are summarised in Table 8.

Table 8: Peak Ground Acceleration (PGA) for Two Limit States

Limit State	AEP	PGA (g)	Magnitude _{eff}
SLS	1/25	0.06	5.9
ULS (IL2)	1/500	0.25	5.9
ULS (IL3)	1/1000	0.32	5.9
Notes: SLS = Serviceability Limit State; ULS = Ultimate Limit State; AEP = Annual Exceedance Probability			

7.3 Site Subsoil Classification

Based on those ground conditions and specific geophysical data gathered by the University of Waikato and Auckland University, the typical site period for the site is expected to be about 3 seconds where the seismic site subsoil category is assessed as being Class D (deep soil site) in accordance with NZS1170.5

7.4 Fault Rupture

As indicated on the GNS Community Fault Model, the nearest known active fault is the Kerepehi Fault, located 34km west of the site. Due to the distance offset from the site, the risk of fault rupture of this fault to this site is low.

In addition, the Kututaruhe Fault is identified in the Community Fault Model, and is located approximately 1.2km beyond the eastern boundary of the site running northeast to southwest. This fault is classified as inactive and have a long return period of > 100,000 years. As such, the risk of fault rupture of this fault to this site is low.

7.5 Liquefaction

7.5.1 Methodology

In accordance with MBIE/NZGS guidance the liquefaction susceptibility of the soils at this site was assessed with respect to geological age and compositional (soil fabric and density) criteria, based on the following assumptions:

- Only saturated soils below an assessed seasonal average groundwater level were modelled as being susceptible to liquefaction.
- In accordance with MBIE/NZGS guidance⁵ and in conjunction with site-specific shear wave velocity measurements, no aging / strength gain factor has been applied to the Pleistocene aged soils.
- Soils are also classified with respect to their grain size and plasticity to assess liquefaction susceptibility. For this project, a cut-off threshold soil behaviour type index value (I_c) of 2.6 was used to distinguish between liquefiable ($I_c < 2.6$) and non-liquefiable ($I_c > 2.6$) soils.
- Specific liquefaction analyses were undertaken for IL2 and IL3 structures, using the software package CLiq and the Boulanger and Idriss (2014) method. The cyclic stress ratio (CSR), being a function of the PGA corrected for

the relevant magnitude for the design return period event, was compared to the cyclic resistance ratio (CRR), being a function of the CPT data.

- Free-field liquefaction induced settlements were determined in accordance with Zhang et al. (2002). With respect to liquefaction response, consideration was given to a 10m cut-off depth to estimate index settlements as per MBIE guidance⁶ (foundation technical categories). These were compared to liquefaction settlement estimates over the full depth range of the CPT's with a depth weighting factor (ev) applied ranging from 1 at the ground surface to 0 at 20m depth.
- Engineered fill capping layer in the low-lying area was conservatively ignored in our analysis.

7.5.2 Liquefaction Analyses Results

A summary of liquefaction analyses results is presented below. A complete table of results (per CPT) is presented in **Appendix I**.

Table 9: Summary of Liquefaction Results

Landform	SLS Settlement (mm)	Total ULS (IL2) Settlement (mm)	Index ULS (IL2) Settlement (mm)	Total ULS (IL3) Settlement (mm)	Depth to Liquefied Layer (m)
Elevated Ridges	<5	20 - 90	10 - 70	25 - 100	0.9 - 5.2
Low Lying Areas	<5	25 - 110	10 - 110	30 - 120	0 - 4.3
Note: Settlements and depths based on existing ground profile. Index settlements are calculated based on the upper 10m of the soil profile using no depth weighting factor. Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied. Index settlements are for assessment of the site against the MBIE site Technical Category guidelines and are not comparable to the total ULS settlements.					

The results show that liquefaction is negligible (<5mm) in the SLS case. In the ULS case:

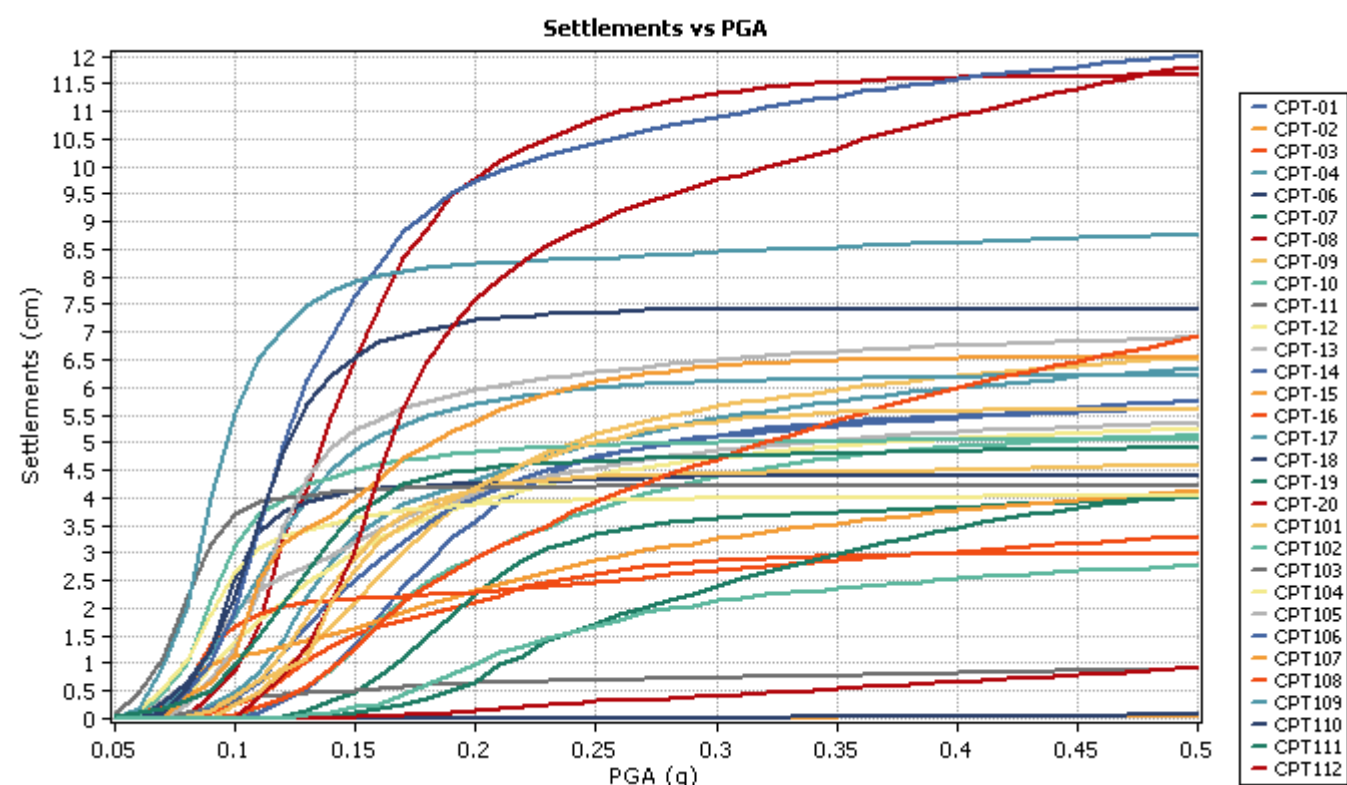
- Widespread liquefaction of the soils in the low-lying areas is predicted, with large total and differential settlements predicted to occur. In addition, the shallow depth to liquefaction presents a risk of surface manifestation, which may induce additional settlement not accounted for in the estimates in **Table 11**. Surface manifestation is difficult to predict and can result in large differential settlements.
- For heavy buildings, (e.g. multi-storey apartment blocks, or some commercial/industrial buildings), significant load induced shear settlement may occur due to temporarily reduced bearing capacity resulting from liquefaction. This element of settlement is not considered in the above estimates, and should be assessed where relevant at detailed design.
- Liquefaction is predicted within the elevated Hamilton Ash and Walton Subgroup soils where the I_c is less than 2.6. However, experience with Waikato soils suggests that CPT tests can misclassify these soils or underestimate their strength due to their sensitivity. Visual assessment and laboratory testing of these soils may rule out susceptibility to liquefaction.

Results indicate liquefaction is triggered at a seismic load of PGA 0.07, which is slightly above the SLS return period. Results presented below.

⁴ NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (March 2016)

⁵ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

Figure 4: Parametric Summary of predicted vertical settlements from soil liquefaction following a ULS seismic event (IL3: PGA 0.32g)



Recommendations to mitigate effects of liquefaction induced settlement on the proposed development are provided in Section 8.8.

7.6 Cyclic Softening

The fine-grained alluvium, while not liquefiable due to its high plasticity, may be susceptible to some strength loss, referred to as cyclic softening, during a ULS seismic event.

Cyclic softening analyses of those soils was carried out in accordance with Boulanger⁶ and Idriss⁷. This correlates earthquake magnitude to the estimated number of equivalent stress cycles (Figure 5) and then correlates number of cycles to a cyclic shear strength ratio (Figure 6).

Figure 5: Relationship between earthquake magnitude and mean number of uniform stress cycles

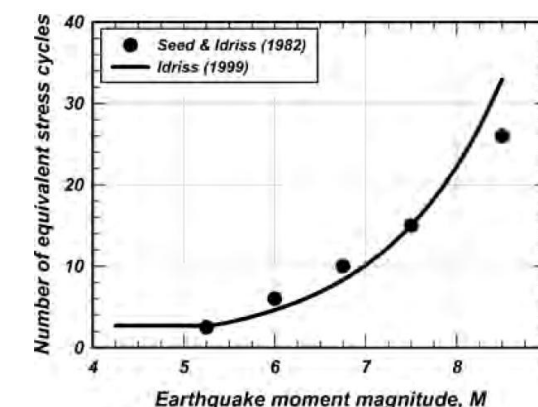
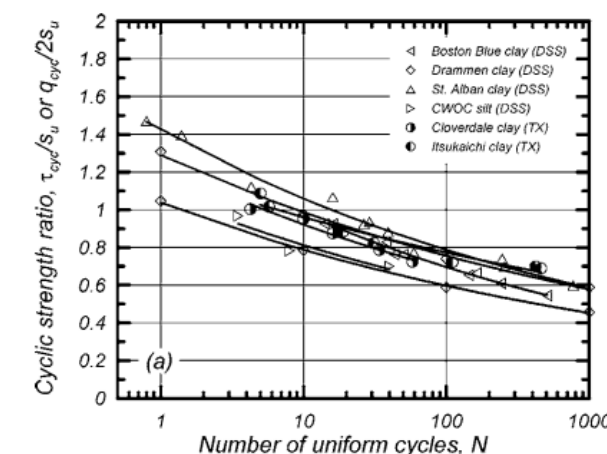


Figure 6: Relationship between cyclic strength ratio and number of uniform stress cycles



Cyclic softening is predicted to occur within isolated layers of the Piako Subgroup and Walton Subgroup units. Based on the above assessment, 6 stress cycles are estimated during the ULS M5.9 earthquake resulting in a worst-case cyclic shear strength of 85% of the peak shear strength. Reduced shear strengths have been considered for the slope stability analysis in Section 7.8 below.

7.7 Lateral Spread

Following the onset of liquefaction, the liquefied soils behave as a very weak undrained material, which can give rise to lateral spreading where a free face is present within the vicinity of the site or where proposed cut and fill batters are proposed over or within liquefied soils.

The potential for and estimated magnitude of lateral spread beneath the site has been assessed with respect to liquefied shear strengths ($S_{u(liq)}$) within the liquefied soil units using a Newmark Sliding Block approach as discussed in Section 7.8 below.

⁶ Boulanger, R.W. and Idriss, I. M. (2007) Evaluation of Cyclic Softening in Silts and Clays, Journal of Geotechnical and Environmental Engineering, Vol 133, Issue

⁷ Idriss, I. M. and Boulanger, R. W. (2008) Soil Liquefaction During Earthquakes. Monograph 12, Earthquake Engineering Research Institute.

7.8 Slope Stability

7.8.1 Design Criteria

The stability of the swale drains under a range of design conditions is expressed in terms of a factor of safety, which is defined as ratio of the resisting forces to destabilising forces acting on a slope. The table below presents the performance standards⁸ for recommended for slope stability assessment:

Table 10: Slope Stability Factor of Safety Criteria

Condition	Required Factor of Safety
Static long-term conditions (drained soil conditions, normal groundwater)	1.5
Transient short-term conditions (elevated groundwater and pore pressures)	1.3
Seismic (ULS) case (undrained soil conditions for cohesive soils, drained for granular soils)	1.0*
Note: * = Factor of safety <1.0 acceptable where displacement-based approach is adopted.	

7.8.2 Slope Stability Analysis

The slope stability analysis was carried out using the Morgenstern-Price method of slices using the proprietary software Slide2.

We have assessed the slope stability of the following critical sections from the supplied Maven Landform Drawings:

- Section I (Low Lying): 3.2m deep stormwater pond with a slope gradient of 1V:4H. Note this case is the ‘worst case’ model for proposed stormwater ponds in the low-lying areas, these results and subsequent recommendations can be conservatively applied to all ponds in low-lying areas. Where this pond cuts into peat, we have adopted reduced effective stress parameters. When cutting into peat, it damages the soil matrix and reduces its strength, effectively resulting in it behaving with a residual soil strength and being susceptible to “ooze”.
- Section J (Northern Ridges): 16m high cut fill slope with a slope gradient of 1V:3H, with a 4m deep stormwater pond at the crest of the slope. Note this case is the ‘worst case’ model for proposed stormwater ponds in the elevated ridges, these results and associated recommendations can be conservatively applied to all ponds in elevated ridges.
- Section K (Northern Ridges): 6m high retaining wall beneath a proposed road corridor. Toe slope gradients of 1V:3H.
- Section L (Northern Ridges): 21m high cut slope with series of multi-tier retaining walls, largest wall is 9m high 2-tier wall, in cut slope with mid slope gradients of 1V:2.5H.
- Section M (Southern Ridges): 10m high cut slope with series of multi-tier retaining walls, largest wall is 3m high.

Where a retaining wall is proposed, a shear pile has been modelled in Slide2 with the embedment depth conservatively (from a global stability perspective) taken as equal to the retained height. These walls are subject to detailed design, and design retaining wall pile embedment depths will likely be deeper. Global stability for all walls must be reassessed during detailed design.

Analyses have been completed for the various conditions stated in **Table 10** based on the following:

- Drained soil conditions were analysed for the prevailing case, with the groundwater table based on the available groundwater data.
- When analysing transient/elevated groundwater conditions, an elevated pore pressure of $r_u = 0.3$ was applied to the soils above the groundwater table.
- Seismic displacements were estimated based on a Newmark Sliding Block approach using 50th percentile correlations published in Bray and Macedo (2019)⁹, and Macedo et al. (2023)¹⁰.
- A liquefied shear strength ratio is predicted based on methods discussed by Olson & Stark (2002) and Idriss & Boulanger (2008), which takes into account overburden pressure above the liquefiable layers. Liquefiable layers have been modelled where within 2H (vertically) of the slope.

7.8.3 Analysis Results

Selected stability printouts are attached in **Appendix J** with the factor of safety (FoS) for each scenario outlined below.

Table 11: Slope Stability Analyses Results

Condition	FoS: Section I (Low Lying)	FoS: Section J (N Ridges)	FoS: Section K (N Ridges)	FoS: Section L (N Ridges)	FoS: Section M (S Ridges)
Design Case - Static long-term conditions (drained soil conditions, normal groundwater)	1.7	2.1	2.4	2.3	2.8
Transient short-term conditions (elevated groundwater and pore pressures)	1.9	1.6	1.9	1.4	3.4
Seismic (ULS) case (peak strengths)	1.0	1.2	2.2	1.5	1.8
Yield Acceleration (softened/liquefied strengths) (g)	0.1	-	-	-	-
Liquefied displacements (mm) in worst-case scenario – Liquefied soil post (ULS) seismic event	50	-	-	-	-

The results show that the proposed slopes meet the requisite factors of safety in the static cases. In the seismic (liquefaction) case, displacement of up to 50mm is anticipated for lots around the low lying ponds.

Recommendations for foundation design are outlined in Section 8.3. Further assessment is required to determine the extent of the risk of lateral spread relative to the subdivision design at the detailed design stage after the extent of the earthworks is finalised.

7.9 Load Induced Settlement

Proposed fill embankments and future building loads will induce settlements within the underlying Swamp Deposits and Piako Subgroup soils. CPT testing was carried out to assess the soil modulus parameters for load induced settlement analysis.

The Swamp Deposits and Piako Subgroup soils are near-normally consolidated meaning that for the proposed fill embankment and foundation pressures, these soils will experience significant consolidation settlement following the virgin compression (c_c) line.

⁸ CoLab, (October 2025). Regional Infrastructure Technical Specification (RITS) V2, Table 16: Proposed factors for safety for different slope stability scenarios

⁹ Bray, J. D., & Macedo, J. (2019). Procedure for estimating shear-induced seismic slope displacement for shallow crustal earthquakes. *Journal of Geotechnical and Geoenvironmental Engineering*, 145(12), 04019106.

¹⁰ Macedo, J., Bray, J. D., & Liu, C. (2023). Seismic slope displacement procedure for interface and intraslab subduction zone earthquakes. *Journal of Geotechnical and Geoenvironmental Engineering*, 149(11), 04023104.

A summary of the parameters adopted for settlement assessment is summarised as follows:

Table 12: Summary of Consolidation Settlement Assessment Parameters

Parameter	Value
Compression Index (c_c)	1.0
Recompression Index (c_r)	0.05
Initial Void Ratio (e_0)	1.9
Secondary / Primary Compression Index Ratio (c_{α}/c_c)	0.05
Coefficient of vertical consolidation ($c_v - \text{m}^2/\text{year}$)	10

7.9.1 Settlement Predictions

The construction of earthfill embankments over Swamp Deposits and Piako Subgroup soils will induce consolidation and post construction creep settlements. An assessment of static settlements was completed for the proposed range of fill embankment heights. For the range of proposed fill heights, it is assessed that the underlying compressible soils will behave in a normally consolidated state following the c_c compression line.

Primary consolidation settlements were assessed using a CMW in house spreadsheet in accordance with the following Terzaghi 1-dimensional consolidation theory for normally consolidated soils (use alternative formula for over consolidated soils):

Primary consolidation was calculated using Terzaghi's 1-dimensional consolidation theory for normally consolidated soils:

$$S_{consol} = \frac{C_c}{1 + e_0} \cdot H \cdot \log\left(\frac{\sigma'_v + \Delta\sigma_v}{\sigma'_v}\right)$$

Post-construction (secondary or creep) settlements were then calculated based on the method described in Mesri et al (1994). A nominal 10kPa building surcharge has been used for this assessment.

$$S_{creep} = \frac{C_{\alpha}}{1 + e_0} \cdot H \cdot \log\left(\frac{t}{t_i}\right)$$

We have conservatively adopted these compression parameters for both the peat and underlying Piako Subgroup units for this concept design. Additional testing within the Piako Subgroup at detailed design is likely to show less conservative consolidation parameters which can be incorporated into the final design.

Estimated primary consolidation static settlements are predicted to range between 25mm and 780mm. Predicted post construction settlements (45mm to 120mm) are considered to be excessive for the proposed development and specific ground improvement measures will be required.

7.9.2 Time Rate of Settlement

In near normally consolidated soils, the rate of consolidation is governed by the rate at which excess pore-water pressures dissipate within the Swamp Deposit and Piako Subgroup units, which in turn are dictated by their permeability and thickness. The ground model presents a maximum compressible Swamp Deposit/Piako Subgroup thickness of 10m. These units are generally underlain by sandy material thereby providing two-way drainage.

The rate of settlement is notoriously difficult to estimate due to their depositional environment where there are inherent material composition lateral and depth variability and presence of intermediate sand lenses. A c_v of 10 m^2/year has been adopted on this project as a lower bound estimate based on experience with peat soils around Hamilton.

Based on this, a primary settlement duration of 1 to 23 months is estimated, with most areas anticipated to take 9 – 12 months. If c_v was $5\text{m}^2/\text{year}$, the required surcharge duration would double, and if it was $20\text{m}^2/\text{year}$ the surcharge duration would half.

7.10 Bearing Capacity

We anticipate a preliminary geotechnical ultimate bearing pressure of 300kPa should be available for elevated areas, in the low-lying areas bearing capacity is reduced to 200kPa where fill is at least 1.5m thick overlying the low strength peat and Piako subgroup soils, subject to the short axis of those footing being no wider than 1.0m.

Where fill is less than 1.5m thick in low lying areas, specific assessment will be required at the design stage once building loads are known.

There may be areas where localised variations in shear strength within the natural cut ground occur, particularly where the depth of cut varies across the building platforms. Further confirmation of available bearing pressures will be addressed at the time of post earthworks soil testing.

7.11 Expansive site Classification

The expansive site class/s, as defined in AS 2870 (Residential Slabs and Footings – Construction), was assessed based on laboratory test results, visual-tactile assessment of the site soils, and BRANZ Report SR120A.

The assessed expansive site class with respect to AS 2870:2011 is P. Class P sites include soft or unstable foundations such as soft clay or silt or loose sands, landslip, mine subsidence, collapsing soils and soils subject to erosion, reactive sites subject to abnormal moisture conditions.

Measures are required to mitigate against the possible negative effects of seasonal shrinkage and swelling of expansive soils on structures, including their foundations, floors and superstructure. Note that engineer clay fill soils will be used to cap the peat soils discussed further in Section 8, mitigating the effects of expansivity.

We recommend that specific engineering design of foundations is undertaken by a suitably qualified engineer experienced in the design of footing systems for houses. The provisions of AS2870:2011 may be utilised, and Clauses 5.6 and 6.6 should be considered with respect to the additional detailing and construction requirements for Class P sites.

Waffle/pod/ raft floor systems are considered a suitable foundation type to address the risk posed by expansive soils this site. This is due to their ability to bridge shrinkable soils and withstand moderately expansive soils.

Foundation contractors should maintain the moisture content of exposed building platforms during foundation preparation, at all proposed building platform locations. The purpose of this is to prevent excessive drying of exposed clay soils and reduce the risk of soil strength reduction upon re-wetting. Appropriate measures include protecting the platform with polythene; a hardfill layer; pouring a site concrete blinding layer in footing bases; and wetting of the building platform with sprinklers, whilst being careful to avoid over-wetting and softening the soil.

8.0 RECOMMENDATIONS

8.1 Earthworks

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431:2022¹¹ and the requirements of the Waikato District Council, Waikato R.I.T.S¹² and under the guidance of a Chartered Geoprofessional.

The subgrade of the site will predominantly comprise engineered fill from site won material (Hamilton Ash clay/silt). Alternatively, approved imported fill would be suitable, with appropriate compaction specifications, including geotechnical testing and certification to be confirmed during detailed earthworks design.

The natural subgrade soils can be sensitive to remoulding and moisture ingress. Care will be required to avoid over working and trafficking of these materials, and to protect them from moisture ingress. Pumping and yielding conditions may be encountered near the groundwater level or during particularly rainy periods.

Other areas of unsuitable materials such as variable strength uncontrolled fill present within road or building areas should be undercut and may be used as landscape fill in reserve areas or be removed from site. This may include the geotechnical test pit excavation locations.

Further specific earthworks recommendations and specifications should be confirmed at the detailed design stage.

8.2 Ground Improvement for Static Settlement

8.2.1 Performance Requirements

8.2.1.1.1 Residential Dwellings

With respect to B1/VM4¹³, foundation design should limit the probable maximum differential settlement over a horizontal distance of 6m to no more than 25mm under serviceability limit state load combinations of AS/NZS 1170¹⁴, unless the structure is specifically designed to prevent damage under greater settlement.

8.2.1.1.2 Roadings and Gravity Lines

To avoid damage to the pavements by groundwater, the pavements will need to be constructed above existing ground level and will therefore induce settlement in the underlying soil, as will any subgrade improvement where the existing soils are undercut and replaced with heavier material. For preliminary design we have assumed a maximum allowable settlement of 25mm at the ground surface. Tighter settlement tolerances may be required for gravity lines depending on falls.

8.2.2 Concept Options

Concept ground improvement design options to mitigate the static settlement risk were assessed in the CMW technical memorandum (CMW Doc. Ref. HAM2025-0084AG Rev. 0) dated 18 June 2026 and presented in **Appendix G**. Options assessed included the following:

- Constructing a temporary surcharge fill above the design finished level, applying a load greater than that expected from a future structure. This process induces additional primary settlement and over consolidate the soft compressible material before removing the load and thereby limiting the post construction settlement experienced by the structure and infrastructure of the development.

- Constructing a pile supported embankment to mitigate post-construction settlements within the peat. The piles transfer the embankment and building loads down to the underlying dense sands.
- Excavate and replace the compressible soils with an engineered fill material.

The temporary surcharge fill option was recommended and is summarised further below.

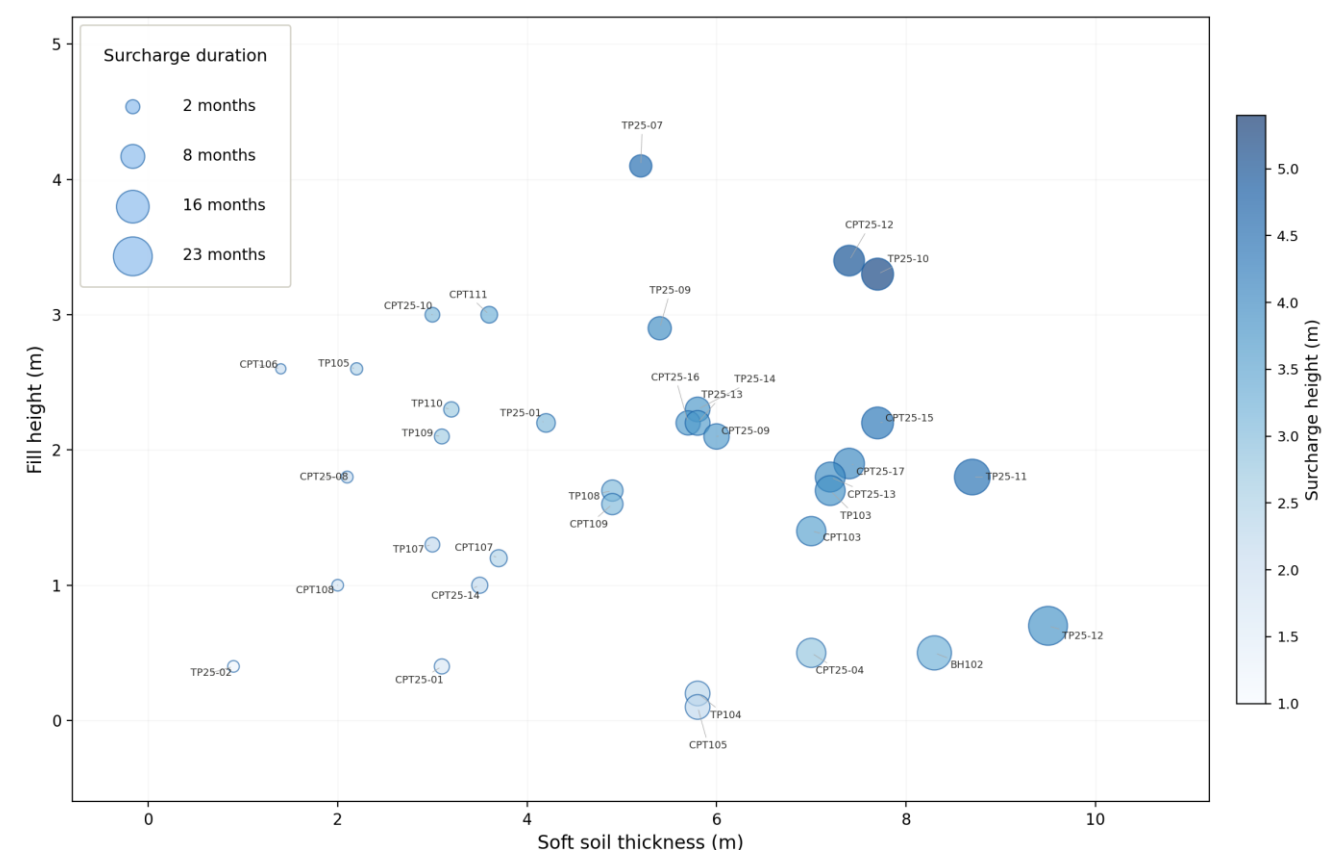
8.2.2.1 Concept Surcharge Design

Concept surcharge design has been completed based on the following:

- Soft soil contours developed as part of the 3-dimensional Leapfrog model of the site.
- Preliminary design finish levels supplied by Maven.
- A nominal 10kPa building surcharge has been used for this assessment with a 50-year design life. This is more conservative than the 100-year design life case for the road due to the permanent loading condition.
- A target 25mm secondary (creep) settlement was used to determine the required temporary surcharge height.
- Differential creep settlement along a gravity wastewater line has been assessed.

A preliminary surcharge design is summarised visually in **Figure 7**. Calculations are presented in the technical memorandum **Appendix G**. The calculations show that the settlement of the peat can be mitigated by this surcharge fill.

Figure 7: Concept Surcharge Design



Note: Surcharge height is in addition to fill height. Duration estimated based on $c_v = 10 \text{ m}^2/\text{yr}$.

¹¹ Standards New Zealand (2022) Engineered fill construction for lightweight structures, NZS 4431:2022

¹² WLAAS Regional-Infrastructure-Technical-Specification-V1.0, dated May 2018

¹³ Acceptable Solutions and Verification Methods For New Zealand Building Code Clause, B1 Structure, 1st Edition, Amendment 21

¹⁴ AS/NZS 1170.0:2002, Structural Design Actions, Part 0: General Principals

8.2.2.2 Time Rate of Settlement

As discussed in Section 7.9.2, a required surcharge duration of 1 to 23 months is estimated, with most areas anticipated to take 9 – 12 months. If c_v was $5\text{m}^2/\text{year}$, the required surcharge duration would double, and if it was $20\text{m}^2/\text{year}$ the surcharge duration would half.

Where surcharge durations need to be accelerated, the following options could be considered:

- Installation of wick drains. These work by significantly reducing soil flow path lengths and utilising the higher horizontal permeability, rather than relying on vertical soil permeability alone. These are constructed prior to building of the fill embankment.
- Increasing the surcharge height. By increasing the surcharge height, we can target a reduced percentage of consolidation completed for the target post construction creep settlement. This can shorten the amount of time the surcharge would be required to remain in place for.
- Use lightweight fill/materials to construct the fill embankment. This would reduce the overall load imposed on the compressible soils, reducing the total settlement and surcharge height required. This may be applicable for areas with less compressible soil thickness.
- Constructing a piled embankment or excavating and replacing the compressible soils.

Preload should be undertaken over the entire low-lying areas of the subdivision or alternatively limited to building platforms and roads.

8.2.2.3 Settlement Monitoring

Detailed settlement monitoring will be required during construction.¹⁵ As there will inevitably be some variation in soil composition and resulting settlement profiles from one location to the next, it is imperative that settlement monitoring is undertaken during construction to back analyse consolidation settlement parameters and progressively refine the pre-load design. It is expected that additional surcharge heights will be required at transition points from cut to fill.

Due to material composition and depth variability, regular monitoring points would be required, such as for example, at 50m longitudinal intervals along the road centreline to demonstrate that differential settlement compliance can be achieved.

We recommend the following monitoring is carried out while surcharging:

- Surface settlement plates, placed over the ground surface prior to filling, to assess total settlement magnitudes, which provide a cost-effective robust monitoring technique;
- Vibrating wire piezometers installed at multiple depths within the peat unit to assess pore water pressure dissipation profiles for settlement and stability purposes.
- Extensometers to monitor the settlement within individual soil layers.

8.2.3 Peat Degradation

In addition to consolidation through settlement, peats can lose further volume through degradation and decomposition over long periods of time. This risk is considered low for this development due to the following:

- Filling across the site is required to raise the site, which will induce settlement within the peat and result in the peat remaining below the groundwater (saturated) over the design life of the structures.
- We have adopted a conservative c_α in the preliminary surcharge design of 0.06 to account for some long term degradation of the organic soils.

¹⁵ The above settlement magnitude and time rate estimates are preliminary only, based on a limited amount of testing data, and have been averaged across the site.

8.2.4 Cross Boundary Effects

Cross boundary settlement caused by proposed earthworks is expected to be negligible;

- Neighbouring structures along the eastern arm of the development are situated on stiff to hard clay hills that are unlikely to settle. Additionally, the adjacent buildings are all located uphill of the proposed fills, and a perimeter swale (mostly cut/negligible fill) is proposed between the development and the neighbouring properties, further diminishing any potential cross boundary effects.
- >5m offset to the nearest neighbouring building beyond the northern boundary.

Based on the current landform design, we are of the opinion that there are suitable setbacks from neighbouring structures outside the site and cross boundary settlement will be negligible, subject to review at detailed design.

8.3 Slope Stability

Proposed slopes meet the requisite factors of safety. We recommend maximum gradients of up to 1V:2H for the cut slopes in the ridges, 1V:4H for cut slopes in the low-lying areas and 1V:3H for the fill slopes in the ridges are adopted for design. Steeper slopes will require additional retention measures such as geogrid reinforcement, retaining walls or soil nails.

Regarding lateral spread, preliminary results of the liquefied slope stability analyses estimate 50mm lateral spread displacement during the ULS event. The estimated lateral spread is within the limits for TC2 Foundation Technical Category outlined within Table 3.1 of the MBIE Canterbury rebuild guidelines Part A.

Lateral displacements should be reassessed during detailed design. If they are >50mm within 50m of the wetlands, buildings in this zone will require specific foundation design to withstand the lateral displacement. This requirement could be waived during detailed design, provided one of the following options are adopted to mitigate the effects of lateral spread:

- Building setback from stormwater wetlands;
- Wetland batter reinforcement with clay lining and geogrid reinforcing;
- Buttressing wetland batters with aggregate material.
- Reinforcing wetland batters with sheet piles.

8.4 Stormwater Soakage

A series of falling head permeability tests were conducted across the site to provide soakage rates. Results indicated low soakage rates ranging between 0.05 to 0.97m/day. Results of testing are presented in **Appendix K**.

Wetland design at the site will be constrained by shallow ground water observed across the site (<1m bgl).

8.5 Retaining Walls

Several retaining walls are proposed around the development, with retained heights up to 6m high. Where retained heights are greater than around 3m, timber pole cantilever retaining walls are unlikely to be suitable, the following walls types could be considered:

- Steel UC cantilever wall. This utilises a steel member to increase the stiffness of the wall, allowing the retained height to be increased.

- Utilising tiebacks / anchors within cantilever walls. These transfer some of the driving load to the soil beyond the retained active wedge, reducing the load on the wall and allowing the retained height to be increased.
- Soil nail walls. These are typically steel or fibreglass members drilled and grouted into the slope at regular intervals to apply resistance and achieve steeper slope angles. These require a facing design, which could comprise concrete or a reinforced mesh product such as MacMat R.
- MSE Walls. These utilise geogrid tiebacks at regular intervals to support the wall. Nominally, the tiebacks are required to be equal to the height of the wall, and require additional earthworks to cut the in-situ soil and lay the geogrids.

Detailed design of these walls should be undertaken by a Chartered Geotechnical Engineer once the design levels and layouts are known.

8.6 Acid Sulphate Soils

Field testing and soil sampling was undertaken by SLR with details presented in their Technical Memorandum¹⁶, to determine the presence and concentration of acid-soluble sulphate (ASS) content across the site. ASS indicators were identified across the Site; however, only one laboratory sample (SS47, 0.25 m bgl clayey silt) met the criteria for AASS or moderate-risk PASS and was classified as SH2 – Moderate Risk.

Overall, management will be required due to the scale of earthworks and low natural buffering capacity; however, the lime requirements are relatively low, and implementation through the Acid Soluble Sulphate Management Plan (ASSMP) during earthworks is considered appropriate and sufficient. The ASSMP¹⁷ outlines the procedures, responsibilities, and monitoring requirements necessary to ensure compliance with the ASSMP during construction activities.

From a concrete durability perspective, initial indications of possible acid-soluble sulphate contents are low and, in isolation, are not suggestive of elevated sulphate exposure for buried concrete. However, formal concrete exposure classification should be based on water-soluble sulphate in soil and/or groundwater, together with pH (per NZS 3101: Concrete Structures Standard). If concrete will be in contact with these soils, we recommend confirming with a 2:1 water-soil extract (water-soluble sulphate), groundwater sulphate and pH testing; pending that, normal good practice (appropriate cover, quality concrete, drainage to avoid ponding) is considered sufficient.

Additionally, possible acid-sulphate containing soils are restricted to the low lying parts of the site which will be capped by engineered clay fill, forming a barrier between the foundations and any possible acid sulphate containing soils, therefore we consider the risk of acid sulphate soils damaging foundations within the engineered fill to be low, and effects should only be considered for construction below the fill (e.g underground services, pump stations).

8.7 Road Subgrades

Based on the investigation results, low strength near-surface silts and loose sands with typical Dynamic Cone Penetrometer values of 1 to 2 blows per 100mm penetration are present to a depth of up to approximately 1.0m over the majority of the site. These equate to an estimated CBR of 2% to 3%.

For road subgrades, after topsoil stripping and cutting to design subgrade levels the following should be undertaken:

- Proof rolling should be carried out prior to placement of any filling/hardfill to check for any areas of excessive deflection. Where clean sandy soils are exposed, moisture conditioning and in-situ compaction may be required to achieve optimum water contents and enable subgrade compaction.
- Where low strength uncontrolled fill or natural soils are exposed at subgrade levels, an allowance for undercut and replacement with clean compacted sand should be allowed for.

¹⁶ SLR, (5 August 2026) Technical Memorandum, Brymer Development - Acid Sulphate Soils Investigation, Ref: 810.031825.00001

¹⁷ SLR, (dated 5 August 2026) Acid Sulphate Soil Management Plan, Brymer Development, Ref: 810.031825.00001 Rev 1.0

It is recommended that a programme of penetration resistance testing is carried out at routine intervals along road alignments as part of the road pavement design prior to road construction to confirm actual CBR values.

Note that roadways cross over several waterways, further mitigations for this are discussed in the technical memorandum attached in **Appendix G**.

8.8 Trench and Open Drain Backfilling

All services trenches and open drains beneath road carriageways and within a 45-degree zone of influence of any building foundation should be backfilled with compacted trench spoil or imported pit-sand to meet engineered fill standards.

8.9 Foundations

It is our understanding that the subdivision will consist of predominantly one and two-story buildings, with up to five storey apartments in the centre in of the site.

8.9.1 One and Two Storey Buildings

As discussed in Section 7.5 there is potential for liquefaction induced ground deformation to occur at the site and that differential settlement is likely to occur in the low-lying soils following a ULS earthquake that will exceed the limits of typical TC2 foundations. These effects can be reduced provided the low-lying parts of the site are sufficiently preloaded and capped with engineer certified clay fill as per the recommendations outlined in Section 7.10.

On the ridges, TC2 type foundations are expected to be suitable, subject to further laboratory testing at detailed design stage.

A preliminary geotechnical ultimate bearing pressure of 300kPa should be available in the static case for shallow strip and pad foundations constructed within both the natural cut ground and engineered fill areas.

Table 13: Anticipated Foundation Details

Item	Methodology	Value	Comment
Geotechnical Ultimate Bearing Capacity (GUBC) for shallow foundations		300 kPa (static case)	Available within natural cut and engineered fill areas. TBC at GCR stage
Expansive Soil	AS2870	M (moderate)	Anticipated characteristic surface movement of up to 40mm (M)
Strength reduction factors	B1/VM4 ¹⁸	0.8	Load combinations involving earthquake overstrength
		0.5	All other load combinations
Seismic site Class(es)	NZS1170	D	

8.9.2 Multi-Storey Buildings

Specific site investigation and foundation suitability assessments including bearing capacity and static settlement should be carried out prior to Building Consent application for any larger buildings such as the 5 storey apartments.

High-Level foundation options include:

¹⁸ Ministry of Business, Innovation and Employment (2019) *Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure*, B1/VM4, Amendment 19

- Pile foundations founded in competent soils approximately 15-20m below existing ground level.
- Ground improvement such as deep soil mixing to improve the underlying soils.

In addition, we understand a basement carpark is proposed beneath each of these buildings. This will extend down into the underlying peat subsoils, presenting significant engineering challenges, including reduced bearing capacity, increased settlement, construction and groundwater risks. Options to mitigate these include:

- Construction of a perimeter secant bored pile wall around the basement. These piles could be used to both retain the peat and earthfill behind the wall and transfer loads from the building down to the underlying dense sand unit.
- Ground improvement of the underlying cohesive subsoils using deep soil mixing or similar to increase the strength of the soil.

The proposed carpark basements will be below the groundwater and will require specific drainage design and water sealing to prevent water ingress.

8.10 Certification

At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared. The GCR will:

- report on the works undertaken
- confirm foundation design parameters
- describe future building and/ or earthworks limitations
- apply any restrictions that require further engineering investigation and/ or design on individual lots to avoid future building works exacerbating a natural hazard

Restrictions that are expected to be applied in the GCR are outlined in the Project Risk Register.

8.11 FURTHER WORK

The following further work is recommended as the project progresses:

- Further geotechnical investigations comprising Laboratory Testing on cohesive soils: Atterberg/Fines Content Testing to refine lateral spread models, as well as compaction curve and solid density testing to determine potential fill quality and Oedometer testing to inform final preload design.
- Lateral Spread analysis for swale design.
- Preparation of a Geotechnical Effects Management Plan (GEMP) outlining settlement and groundwater monitoring requirements.
- For a definitive assessment of Acid Sulphate Soil (ASS) potential, testing such as the Chromium-Reducible Sulphur (CRS) method or the full SPOCAS suite is recommended in line with recognised ASS guidelines.
- Plan review, further analysis once a scheme plan is finalised, etc.
- Preparation of a Geotechnical Earthworks Specification prior to construction.
- During construction, regular inspections and testing will be required as outlined in the Geotechnical Works Specification.
- At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared by a Chartered Professional Engineer outlining the works completed and all recommendations and restrictions for future development on each lot.

9.0 STATEMENT OF PROFESSIONAL OPINION

Based on the results of previous investigations at the site and subject to the preliminary recommendations stated above, we consider that the site is suitable for the proposed development. The proposed plan to enable residential and industrial development, with associated roading infrastructure and stormwater management, is considered appropriate from a geotechnical perspective. This statement is dependent on further work as detailed in Section 8.9.

10.0 CLOSURE

Additional important information regarding the use of your CMW report is provided in the 'Using your CMW Report' document attached to this report.

This report has been prepared for use by Maven Ltd in relation to the R2 Growth Cell Development in, Whatawhata, Hamilton, New Zealand project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

APPENDIX A

Drawings

APPENDIX B

Historical Aerial Images

APPENDIX C

Development Plans

APPENDIX D

2025 Investigation Logs

APPENDIX E

Historic Investigation Logs

APPENDIX F

Laboratory Results

APPENDIX G

Previous Reports

APPENDIX H

Natural Hazard Risk Classification

CONTEXT

Section 2 of the RMA defines natural hazards as any atmospheric or earth or water related occurrence (including earthquake, tsunami, erosion, volcanic and geothermal activity, landslip, subsidence, sedimentation, wind, drought, fire or flooding) the action of which adversely affects or may adversely affect human life, property, or other aspects of the environment.

This appendix to CMW report reference HAM2024-0014AD Rev0 sets out the criteria for the assessment, with results presented within Section 7 of this report which outline the geotechnical-related natural hazards associated with this proposed subdivision development. The remaining hazards, i.e. tsunami, wind, drought, fire and flooding hazards are not covered by this assessment.

BASIS OF ASSESSMENT

Risk Classification

The occurrence of natural hazards and their potential impacts on the proposed subdivision development is assessed in terms of risk significance, which is based on likelihood and consequence factors. A risk table is used to help assess the likelihood and consequence factors, the form of which used by CMW for this project is presented in Table D1.

Table D1: Natural Hazard Risk Classification						
Risk Matrix		Consequence				
		Insignificant 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	Almost Certain 5	Medium 5	High 10	Very high 15	Extreme 20	Extreme 25
	Likely 4	Low 4	Medium 8	High 12	Very high 16	Extreme 20
	Moderate 3	Low 3	Medium 6	Medium 9	High 12	Very high 15
	Unlikely 2	Very low 2	Low 4	Medium 6	Medium 8	High 10
	Rare 1	Very low 1	Very low 2	Low 3	Low 4	Medium 5

Likelihood

With respect to assessing the likelihood or chance of the risk occurring, the qualitative definitions used by CMW for this project are provided in Table D2 for each likelihood classification.

Table D2: Qualitative Natural Hazard Likelihood Definitions		
1	Rare	The natural hazard is not expected to occur during the design life of the project
2	Unlikely	The natural hazard is unlikely, but may occur during the design life
3	Moderate	The natural hazard will probably occur at some time during the life of the project
4	Likely	The natural hazard is expected to occur during the design life of the project
5	Almost Certain	The natural hazard will almost definitely occur during the design life of the project

Consequence

In terms of determining the consequence or severity of the natural hazard occurring, the qualitative definitions used by CMW for this project are provided in Table D3 for each consequence classification.

Table D3: Qualitative Natural Hazard Consequence Definitions		
1	Insignificant	Very minor to no damage, not requiring any repair, no people at risk, no economic effect to landowners.
2	Minor	Minor damage to land only, any repairs can be considered normal property maintenance no people at risk, very minor economic effect.
3	Moderate	Some damage to land requiring repair to reinstate within few months, minor cosmetic damage to buildings being within relevant code tolerances, does not require immediate repair, no people at risk, minor economic effect.
4	Major	Significant damage to land requiring immediate repair, damage to buildings beyond serviceable limits requiring repair, no collapse of structures, perceptible effect to people, no risk to life, considerable economic effect.
5	Catastrophic	Major damage to land and buildings, possible structure collapse requiring replacement, risk to life, major economic effect, or possible site abandonment.

Risk Acceptance

It is recognised that the natural hazard risk assessment provided herein is qualitative and, due to the wide range of possible geohazards that could occur, is somewhat subjective. Other methods are available to quantitatively assess an acceptable level of geotechnical related natural hazard risk, such as defining an acceptable factor of safety with respect to slope stability or acceptable differential ground settlements with respect to recommended building code limits.

Therefore, to give this qualitative natural hazard risk assessment some relevance to more commonly adopted numerical or quantitative geotechnical assessment techniques, a residual risk rating of very low to medium (risk value = 1 to 9 inclusive) is considered an acceptable result for the proposed subdivision development.

A risk rating of high to extreme (risk value ≥ 10) is considered an unacceptable result for the proposed subdivision development.

RISK ASSESSMENT

The natural hazards relevant to this proposed subdivision development and adjacent, potentially affected land have been assessed with respect to the criteria outlined above.

Assessment is based on proposed post development ground conditions with and without any geotechnical controls. The latent risk was first assessed with the site in its proposed developed state to consider the risks to the development and surrounding land, including assessment of land modifications from the pre-existing natural state, without any implemented geotechnical controls. The specific geotechnical mitigation measures and engineering design solutions outlined in the table below and CMW report, where relevant, were then considered to determine the natural hazard residual risk remaining after the proposed controls have been implemented.

Results of this assessment are presented in Section 7..t

APPENDIX I

Liquefaction Results

APPENDIX J

Slope Stability Results



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