

28 August 2026

Te Kowhai East Industrial Development

270 Te Kowhai Road and 28 Mathers Road, Hamilton

GEOTECHNICAL INVESTIGATION REPORT

Te Kowhai East Limited Partnership

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Hamilton

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AUTHOR QUALIFICATIONS, EXPERIENCE AND CODE OF CONDUCT

The qualifications and experience of the author and technical reviewers of this report are summarised below. We confirm that we have read the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and agree to comply with it. We confirm that the items addressed in this report are within our area of expertise, except where we state that we are relying on the evidence or advice of another person. We have not omitted to consider material facts known to us that might alter or detract from the opinions expressed in this report.

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André is an Engineering Geologist based in the Waikato region with more than seven years' experience within the land development sector. His background includes involvement in a variety of earthworks and land development projects, providing geological input from early-stage assessment through to monitoring and certification.

EXECUTIVE SUMMARY

This report presents the results of geotechnical investigations and geohazards assessment for the proposed 187.5ha Te Kowhai East industrial development, located at 270 Te Kowhai Road and Mathers Road, Hamilton.

Most of the site is generally level and characterised by low-lying plains at RL27 to 29m. In the north, the landscape transitions into rolling hills, where slopes rise moderately from the plains to approximately RL40m at the site boundary.

The site is underlain predominantly by interbedded sands and silts of the Hinuera Formation across the entire extent of the low-lying plains with the rolling hill topography underlain by a sequence of Walton Subgroup soils consisting of weathered volcanoclastic and alluvial deposits comprising clays, silts and sands.

The development will require downcutting the rolling hill landform by up to 10m to generate fill material for placement across the low-lying plains, enabling the formation of near-level building platforms and industrial roading. The excavation of the hill slopes will necessitate retained heights of up to approximately 6m, with a 1V:3H graded slope above and a 10m buffer to the site boundary.

Geotechnical analysis and recommendations for the proposed development are summarised as follows:

- Liquefaction triggering is anticipated during a design-level seismic event, with moderate to severe ground deformation, between 50mm to 200mm vertical settlement is anticipated under IL2 Ultimate Limit State (ULS) conditions. This can be managed through appropriate structural and foundation design. Further lab testing may refine the liquefaction assessment.
- The proposed wastewater pump station will require specific geotechnical design to address potential liquefaction-induced settlement and buoyancy effects during seismic events. These risks can be managed through appropriate structural detailing, drainage measures, and, where necessary, ground improvement.
- Up to 100mm of primary settlement is expected beneath new fills (and up to 60mm additional primary settlement occurring beneath a nominal 20kPa building load), with most movement occurring rapidly during construction due to sandy underlying soils. Settlement monitoring during earthworks is recommended to confirm when ground movements have stabilised, and preload may be required to reduce primary settlements beneath foundations, or alternatively foundations can be designed to tolerate the predicted residual and differential settlements.
- Stormwater soakage is considered feasible across the site; however, permeability varies spatially, and final design must account for soakage rates, groundwater levels, and soil stratification, noting that poor soakage performance is expected on the elevated rolling hill terrain underlain by low-permeability Walton Subgroup clays and silts.
- Foundations are expected to achieve suitable bearing capacity once earthworks are completed.
- Earthworks must follow NZS 4431:2022 and Waikato Regional Council (Regional Infrastructure Specification) requirements under geotechnical supervision, plus:
 - The predominantly stiff and non-sensitive silt and clay (weathered volcanic ashes) across the rolling hills (to depths of nominally 2m to 4m) should be suitable for re-use as engineered fills with appropriate moisture conditioning and compaction. Excavation of these materials will be readily achieved with normal earthworks plant, such as scrapers and bulldozers/tractors with scoops.
 - Some soils (Hinuera Formation and Puketoka Formation silts) are sensitive which may be susceptible to rapid strength loss when disturbed. These sensitive soils will require significantly more moisture conditioning and careful handling to compact effectively.
 - Blending sensitive silts with non-sensitive volcanic ashes required to achieve suitable fill quality.

- Groundwater will be encountered at shallow depths, requiring drainage measures (localised dewatering), particularly for excavations / trenches across the low-lying plains.
- Roading subgrade should be designed using an initial conservative California Bearing Ratio (CBR) value of 3% for natural soils, with higher values expected in engineered fill. Use of a Subgrade Improvement Layer (SIL) is recommended.
- The proposed reservoir location in the northeastern portion of the site is considered generally suitable from a geotechnical perspective. While substantial engineered fill and associated settlement, slope stability, and liquefaction considerations will require detailed design and mitigation measures, these geotechnical risks can be appropriately managed using established engineering solutions.
- Acid sulphate soil risk is considered low across the site, and any localised risks can be readily managed through typical construction and environmental controls.

Based on the findings of the geotechnical investigations, we consider that the site is suitable for the proposed development provided our recommendations are followed. With the outlined mitigations implemented, we consider that ongoing effects from geohazards would be suitably managed.

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1.0 INTRODUCTION

This technical report presents the results of geotechnical investigation and subsequent assessment for the Te Kowhai East Industrial Development. The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced HAM2025-0116AC, Rev0 dated 14 November 2025.

The report has been prepared to support Te Kowhai East Limited Partnership's substantive application for approvals under the Fast-Track Approvals Act 2024 (FTAA).

The Project is a large-scale industrial development located on the north-western edge of Hamilton, adjacent to the established Te Rapa industrial area. The application provides for the creation of an integrated industrial precinct, including approximately 210 industrial lots, land for neighbourhood centre activities, reserves for recreation, stormwater management and ecological mitigation, and associated roading and infrastructure services.

The application also seeks approval for the ultimate land use outcomes anticipated across the site, being predominantly industrial activities, along with a limited range of compatible non-industrial activities suited to an industrial environment. The development will form a logical extension of Hamilton's primary industrial node within the Upper North Island economic corridor and responds to projected growth and demand.

A comprehensive description of the Project is provided in the Substantive Application Report (SAR).

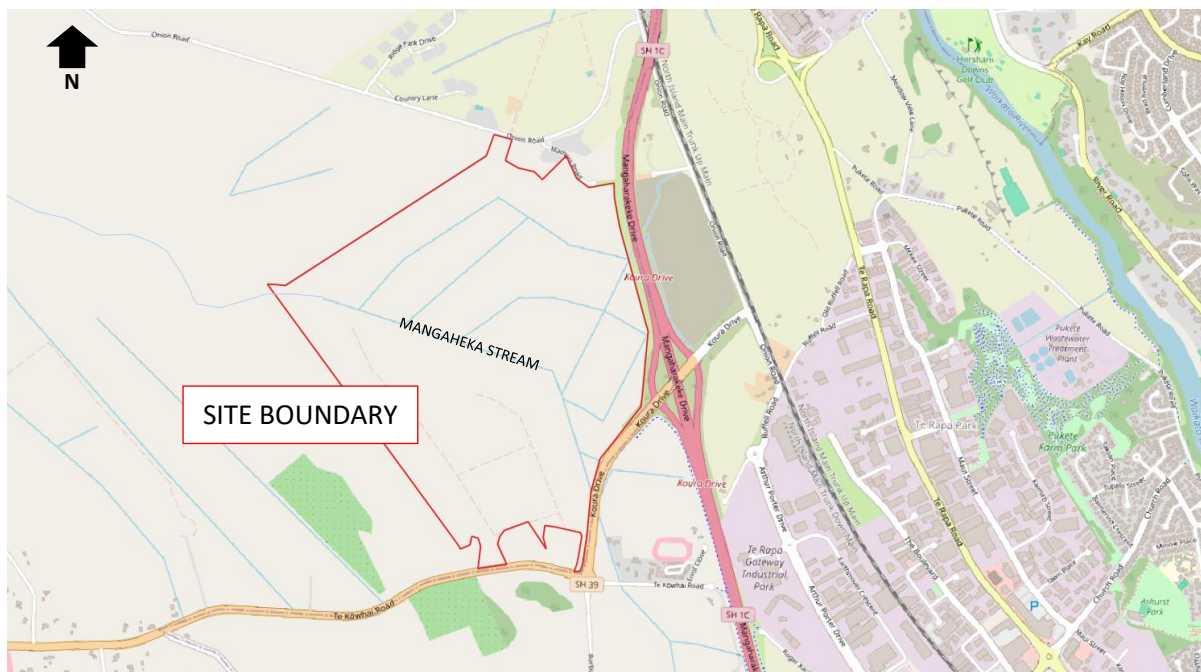
2.0 SITE DESCRIPTION

2.1 Site Location

The Project site comprises a large, contiguous landholding located on the north-western edge of Hamilton. It is generally bounded by Te Kowhai Road to the south, Koura Drive and State Highway 1C to the east, and Onion Road to the north. The site is made up of multiple land titles, legal descriptions, and associated physical addresses.

For identification purposes, the site is commonly referred to by the primary addresses of 270 Te Kowhai Road and 28 Mathers Road. Full details of the landholdings are provided in the Substantive Application Report (SAR). The site is identified in **Figure 1** below.

Figure 1: Site Location Plan (ref. Open Street Maps)



2.2 Landform

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

Most of the site is generally level and characterised by low-lying plains, with existing ground elevations ranging from approximately RL27m to RL29m. Along the northern boundary, the landscape transitions into rolling hill country, where slopes rise moderately from the plains to approximately RL40m. Slopes typically grade at about 1V:3.5H, with minor locally steeper sections up to 1V:2.5H. Beyond the northern site boundary, slopes reach a maximum elevation of around RL53m.

The low-lying plains are predominantly used as pastoral grazing paddocks, with a network of farm tracks providing access throughout the property. Several farm drains intersect the low-lying plains, including the Mangaheka Stream that enters site through a culvert at the southeastern boundary and flows north before diverting west through the central portion of the site, exiting to the northwest. Multiple smaller tributary drains, as shown on **Figure 1**, branch from this main drain. The Te Otamanui Stream crosses the site at the southern boundary, flowing in an easterly to westerly direction, parallel with Te Kowhai Road.

In addition to the existing drainage features, an infilled palaeochannel known as the Te Kowhai Channel is interpreted to extend through the southwestern portion of the site. While no longer expressed at the ground surface, this buried geomorphic feature is considered to influence local subsurface conditions and is discussed further in Section 5.2.

The rolling hill country in the north accommodates an existing farm residence and milking shed, with several smaller ancillary structures located nearby. Historic aerial imagery shows that farming activities were already well-established by the earliest available photographs from 1941, with two farmhouses present at that time and the land across the property cleared of trees and bush. To the west of the milking shed, at the base of the hillside, are three dairy effluent ponds. Two of these ponds have been excavated into the natural soils, while the third is an above-ground constructed pond. Each pond has an approximate surface area of 800 m².

3.0 PROPOSED DEVELOPMENT

The drawings provided by Maven Associates (ref. Proposed Industrial Development, CON200 to C221, dated August 2026) detail the layout for the proposed development at the site. This development will involve downcutting of the rolling hill topography up to 10m to provide fill material for placement across the low-lying plains with the intent to create near level platforms and form roading for industrial development.

As part of the development, upgrades to the Mangaheka Stream, construction of eight stormwater wetlands, and excavation of several perimeter swales are proposed to provide stormwater management across the site. The perimeter swales are generally proposed to be excavated to depths of 1.0m to 2.0m below existing ground level, with the connecting wetlands ranging from approximately 2.5m to 5.0m in depth.

The low-lying plains will generally be raised by approximately 1.0m to 2.0m of engineered fill to create near-level development platforms that grade gently towards the proposed ponds and swales. Localised fills of up to 3.0m are proposed adjacent to the base of the northern cut area, while up to 4.0m of fill is proposed along the northwestern boundary to form a gently sloping embankment supporting a proposed T-intersection road connection from Onion Road into the development.

Primary access to the development is proposed via a new roundabout connection to Te Kowhai Road along the southern boundary.

Downcutting of the northern rolling hill topography will require construction of vertical retaining structures up to approximately 6m in height. Several retaining solutions are currently under consideration, with preliminary options including a cantilever pole wall and a segmental block retaining wall system.

As part of this application, a conceptual water reservoir has been considered comprising two reservoirs, each with a storage capacity of approximately 6ML and a nominal diameter of 30m. The conceptual arrangement is illustrated on the Maven Associates drawings contained in **Appendix B**, Drawing **C206 Rev A**. To accommodate the proposed reservoirs, engineered fill of up to approximately 9.5m in height will be required within the northeastern corner of the site to establish suitable platforms for the reservoirs.

As part of the proposed development, a wastewater pump station is planned for installation in the western corner of the site as shown on the Maven Associates drawing **C530 RevA**, dated August 2026. The pump station will comprise a wet well approximately 9.0m deep, together with associated storage tanks and wastewater pipeline infrastructure.

Selected drawings are presented in **Appendix B**.

4.0 INVESTIGATION SCOPE

4.1 Previous Investigations

HDGeo have previously conducted investigation across the proposed development area and issued a Preliminary Geotechnical Report referenced HD1996 PGR RevA, dated 23 June 2023. Fieldwork carried out and provided within that report included:

- Eight hand auger boreholes, denoted HA01 to HA08, drilled to depths of up to 3.0m below ground level and included vane strength testing and dynamic cone penetration testing. Engineering logs of the hand auger boreholes, together with peak / remoulded vane shear strengths and dynamic cone penetration testing results are presented in **Appendix C**.
- Six cone penetration tests, denoted CPT01 to CPT06, pushed to depths of up to 20.0m below ground level. Results of the CPT's presented as traces of tip resistance (qc), friction ratio (fs) and porewater pressure are presented in **Appendix C**.

4.2 Recent Investigations

Additional investigation was undertaken under the direction of CMW Geosciences between November 2025 and February 2026 to provide additional information and support the FTAA substantive application. All fieldwork was carried out in general accordance with the NZGS specifications¹ and in accordance with NZGS guidance². The scope of fieldwork completed was as follows:

- A walkover survey of the site to assess the general landform, site conditions and adjacent structures / infrastructure;
- An on-site services search was carried out by a specialist contractor to identify the presence of any underground obstructions or hazards prior to the field investigation program commencing;
- Twenty-four hand auger boreholes, denoted HA25-01 to HA25-12 and SWALE HA01 to HA12, were drilled using a 50mm diameter auger to target depths of up to 5.0m below existing ground levels to visually observe the near surface soil profile and to facilitate in-situ vane shear strength testing. Engineering logs of the hand auger boreholes, together with peak and remoulded vane shear strengths are presented in **Appendix D**;
- Dynamic cone (Scala) penetrometer (DCP) tests were carried out adjacent to the hand auger borehole to depths of up to 5.0m to provide soil density profiles. Graphical results of the DCP testing are presented on the borehole logs in **Appendix D**;
- Twenty-five Cone Penetrometer Tests (CPT) denoted CPT25-01 to CPT25-28, were pushed to depths ranging between 10m to 15m to define the deeper ground model. Due to site access restraints at time of investigation, CPT25-13, CPT25-22 and CPT25-26 were unable to be undertaken. Results of the tests are presented as traces of tip resistance (q_c), sleeve friction resistance (f_s) and porewater pressure are presented in **Appendix E**.
- Four Seismic CPT denoted SCPT25-01 to SCPT25-04, were pushed to depths of 20.0m to define the ground model. Results of the tests are presented as traces of tip resistance (q_c), sleeve friction resistance (f_s) and porewater pressure are presented in **Appendix E**.
- A seismic data analysis report was prepared by BCE on behalf of Geo Data Solutions containing the results of the seismic cone penetration testing and assessment of the shear wave velocities. This report is included in **Appendix E**.
- Ten machine auger boreholes, denoted BH25-01 to BH25-10, were drilled using a 90mm diameter HQ drill rig to target depths between 10 to 15m below existing ground levels to visually observe the near surface soil profile, to facilitate in-situ SPT strength testing and to install piezometers across the development for groundwater monitoring to be undertaken. Engineering logs of the machine auger boreholes, together with standard penetration test results, are presented in **Appendix F**;
- Twelve hand auger boreholes, denoted SO25-01 to SO25-12, were drilled using a 100mm diameter auger to depths between 0.9m and 2.2m below existing ground level to facilitate in-situ falling head permeability testing within underlying soils. Engineering logs are presented in **Appendix D** with permeability testing results presented in **Appendix G**.

The approximate locations of the respective investigation sites referred to above are shown on the Site Investigation Plan (Drawing 01). Test locations were measured using handheld GPS. Elevations were inferred from the survey plans provided by Maven Associates.

¹ NZ Geotechnical Society (2017) NZ Ground Investigation Specification, Volume 1 – Master Specification

² NZ Geotechnical Society (2005), Field Description of Soil and Rock, Guideline for the field classification and description of soil and rock for engineering purposes.

4.3 Permeability Testing

Twelve in-situ falling head permeability tests were undertaken to assess the permeability of the site soils in the locations shown on the Site Investigation Plan (Drawing 01). The tests denoted SO25-01 to SO25-12, were undertaken by hand augering to depths of up to 2.2m below ground level to visually observe the near surface soil profile, then reaming the boreholes out to 100mm diameter and installing a perforated 90mm diameter PVC pipe.

Downhole data loggers ('divers') were installed in the base of the boreholes and the pipe filled with water to ground level. On cessation of filling, the fall in head of water above the diver was recorded at five second intervals during the test. The tests were each repeated, with the presented results being taken from the second recording.

Average permeability values of between 1.56×10^{-4} m/s and 9.8×10^{-6} m/s have been estimated based on the test data using Ciria calculation method, which is considered applicable for assessing this development.

The test data is presented in **Appendix G**.

5.0 GROUND MODEL

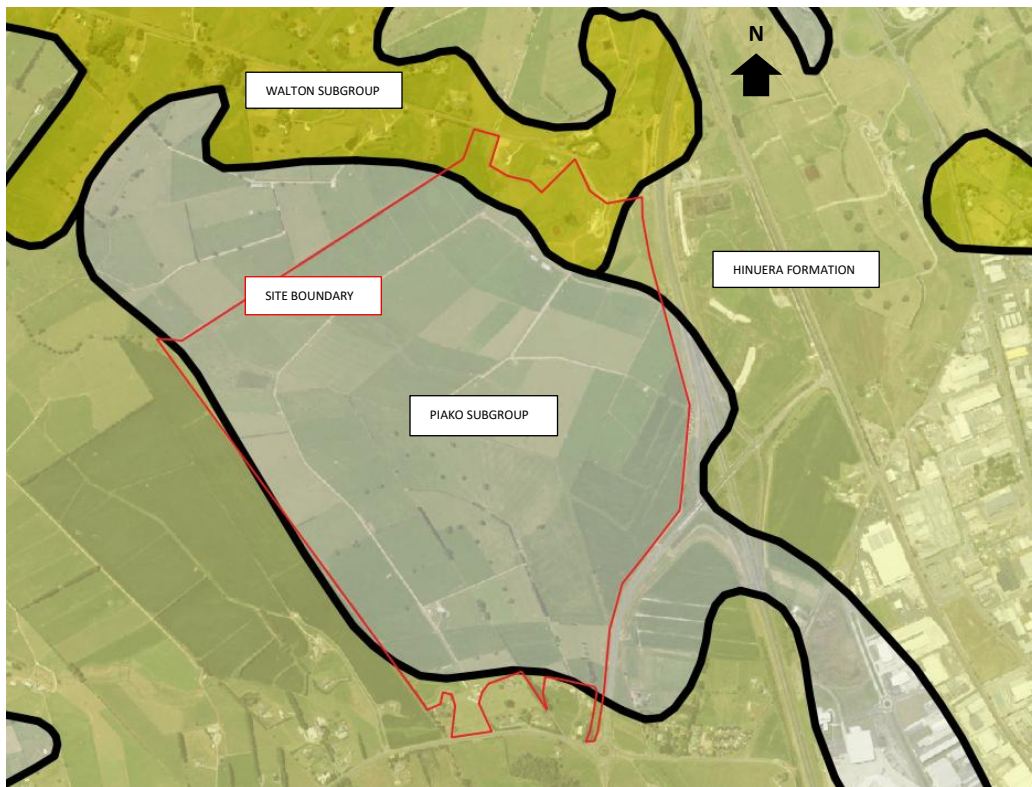
5.1 Published Geology

Published geological maps³ for the area depict the regional geology as comprising of soils belonging to three different geological units, as shown on **Figure 2**, which comprise:

- Piako Subgroup - Holocene aged swamp deposits comprising organic mud, muddy peat and woody peat. These are the most recent, near-surface soils.
- Hinuera Formation - late Pleistocene aged alluvium which comprises cross-bedded pumice sand, silt and gravel with interbedded peats. These soils underlie the Piako Subgroup.
- Walton Subgroup - early Quaternary aged deposits comprising weathered volcanoclastic materials, dominated by primary and fluvially reworked non-welded ignimbrites are overlain by a sequence of weathered volcanic ashes.

³ 1:250 000 Geological Map of New Zealand (QMAP), GNS Science

Figure 2: Geology Map (GNZ New Zealand 1:250k)



5.2 Geomorphology

The ground conditions encountered and inferred from the investigations were partially consistent with the published geology for the area and can be generalised according to the following subsurface sequences.

- Rolling Hill Topography** – The northern rolling hill area is underlain by a substantial sequence of Walton Subgroup deposits. This sequence typically comprises 2–5 m of the Hamilton Ash Formation mantling the surface, underlain by a further 2–5 m of the Kauroa Ash Formation. Both ash formations consist of stiff to very stiff silt and clay soils. Beneath the ashes lies up to approximately 5 m of highly sensitive, stiff to very stiff silt-clay of the Puketoka Formation. This unit is underlain by dense to very dense sands derived from non-welded ignimbrite of the Walton Subgroup.

On some of the steeper slopes there is possible evidence of minor relic slip scarps representing vacated shallow landslides. No recent slope instability features were observed. Ongoing shallow soil creep was observed across the steeper slopes which is exacerbated by farm stock movements.

- Low-Lying Plains Topography** – Regional geological mapping indicates that the low-lying plains are underlain by Piako Formation Holocene swamp deposits, typically comprising weak alluvial silts and peat. However, these materials were typically not encountered during our investigation. Instead, a 1–2 m layer of stiff to very stiff silt was observed consistently across the plains, underlain by 6–10 m of loose to medium-dense sands interbedded with stiff silts of the Hinuera Formation.

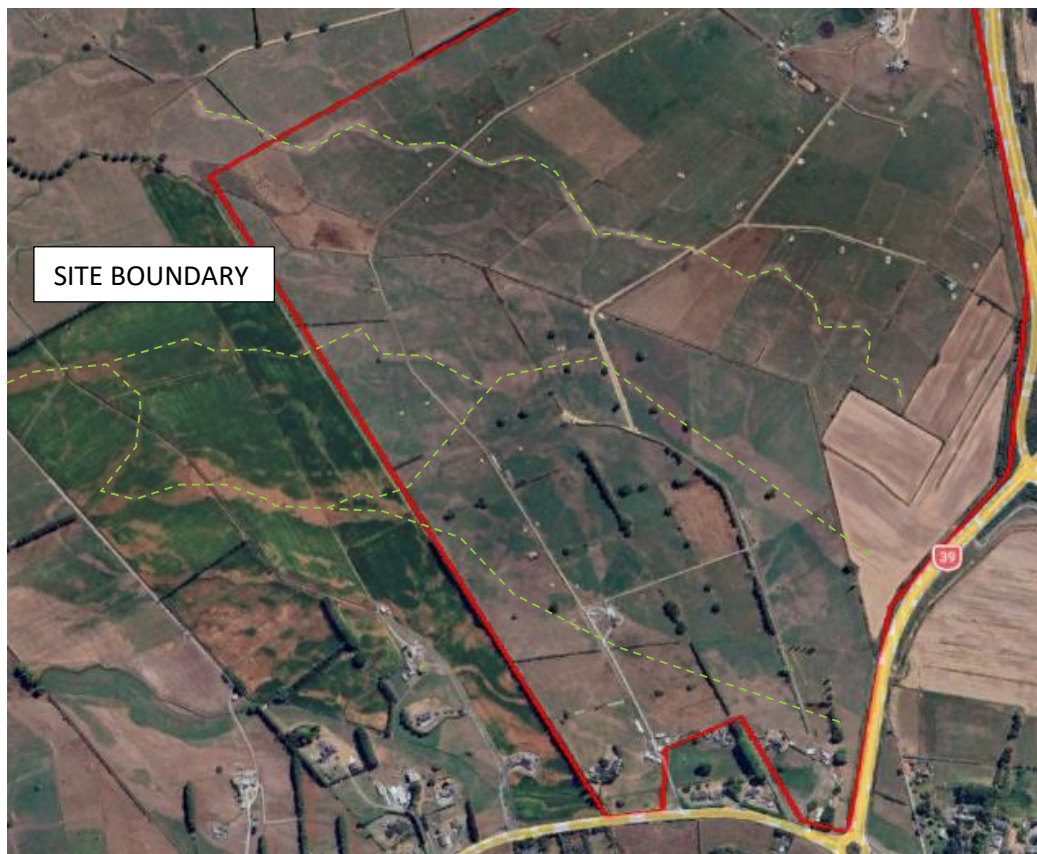
At depths of approximately 8–12 m, the Hinuera Formation is underlain by dense to very dense sands derived from non-welded ignimbrite of the Walton Subgroup.

An isolated pocket of low-strength, compressible Piako Subgroup silt was identified in the northeastern corner of the site, immediately adjacent to the rolling hill terrain. The deposit is considered to represent a localised accumulation formed within a low-energy alluvial setting, likely associated with an eddy or backwater environment during past fluvial activity.

A secondary isolated pocket of organic silt was encountered in the southeastern corner of the site. This deposit is considered to be associated with a larger organic infilled paleochannel which runs outside the southern boundary of site. This area is outside the current proposed earthworks boundaries.

- **Te Kowhai Channel** – A paleochannel of the ancient Waikato River, known as the Te Kowhai Channel, is located in the southwest corner of the site. This feature represents a former natural drainage pathway that once conveyed surface water across the low-lying plains as part of the wider Waikato River system. The channel has been infilled with alluvial gravels to depths of up to approximately 3.5m below surface. An approximate alignment of the paleochannel is shown on the Geomorphology Plan in **Appendix A**, interpreted primarily from aerial imagery, where during summer conditions the coarser-grained channel deposits dry out more readily than the surrounding finer-grained soils, making the channel trace more distinguishable. The shallow gravel backfill is underlain by medium dense sands of the Hinuera Formation.
- **Paleochannels** – As outlined above, a significant paleochannel, known as the Te Kowhai Channel, extends through the southwest corner of the site. In addition to this main feature, numerous smaller tributary paleochannels are visible in aerial imagery (see **Figure 3** below), identifiable from aerial imagery by differential drying patterns associated with coarser-grained channel infill. Targeted hand auger investigations undertaken within these paleochannel traces confirm that they are infilled with coarse sands and gravels, contrasting with the finer-grained soils encountered elsewhere across the site at surface.

Figure 3: Aerial Photo Showing Paleochannels Across Development Area



5.3 Stratigraphic Units

The distribution of the stratigraphic layers at our specific investigation locations are presented in Table 1 and Table 2 below, but variations can be anticipated between these locations. Our findings are illustrated on the appended Geological Section A-A (**Drawings 03 and 04**) presented in **Appendix A**.

Table 1: Stratigraphic Units – Rolling Hill Topography

Geological Unit	Depth to base (m)		Thickness (m)
	Min	Max	
Topsoil	0.15	0.5	0.15 – 0.5
Very stiff to hard Clay / Silt – Hamilton Ash	0.9	2.6	1.9 – 2.3
Stiff to very stiff Clay / Silt – Kauroa Ash	1.45	11.45	1.8 – 2.95
Firm to very stiff Clay / Silt – Puketoka Formation	9.45	14.55	2.05 – 9.45
Dense to very dense Sand – Walton Subgroup	-		-

Table 2: Stratigraphic Units – Low-Lying Plains Topography

Geological Unit	Depth to base (m)		Thickness (m)
	Min	Max	
Topsoil	0.15	0.5	0.15 – 0.5
Firm Silt – Piako Subgroup*	1.4	3.8	1.2 – 3.6
Stiff to very stiff Silt, clayey silt – Hinuera Formation	0.7	2.9	0.5 – 2.6
Loose to medium dense Gravel – Hinuera Formation**	3.6		
Loose to medium dense Sands with silt interbedded – Hinuera Formation	8.0	>10	8 - >10
Firm to very stiff Clay / Silt – Puketoka Formation	9.45	14.55	2.05 – 9.45
Dense to very dense Sand – Walton Subgroup	-		-
Notes: * Only encountered within HA25-11 and HA25-12 **Only encountered within MH25-06			

Although regional geological mapping depicts much of the site as being underlain by Piako Subgroup swamp deposits, the investigation found that these materials are largely absent across the development area. The low-lying plains are instead predominantly underlain by Hinuera Formation cross-bedded sands and silts. Piako Subgroup soils were only locally identified within the southeastern and northeastern corners of the site, where they appear to be associated with the localised low energy environments. This indicates that organic swamp deposition was limited to isolated depressions and former drainage pathways rather than occurring extensively across the site.

5.4 Groundwater

During the investigation, which was completed in mid-summer conditions (February 2026), groundwater was encountered within the machine boreholes at the depths provided in Table 3, which also presents the results of an additional groundwater monitoring undertaken following the investigation:

Table 3: Groundwater Monitoring Data

Standpipe	Topography	Screened Depth* (mbgl)	February 2026	
			Depth (mbgl)	Elevation (m RL)
MH25-01	Rolling Hills	12.5 – 15.5	8.77	32.23
MH25-02	Rolling Hills	12.5 – 15.5	11.23	30.77

Standpipe	Topography	Screened Depth* (mbgl)	February 2026	
			Depth (mbgl)	Elevation (m RL)
MH25-03	Low-lying Plains	3.8 – 7.35	1.32	27.68
MH25-04	Low-lying Plains	4.3 – 5.3	1.18	28.82
MH25-05	Low-lying Plains	5.5 – 7.5	1.57	28.43
MH25-06	Low-lying Plains	3.0 – 5.0	1.41	27.59
MH25-07	Low-lying Plains	3.8 – 5.3	3.74	24.26
MH25-08	Low-lying Plains	2.4 – 3.7	1.56	26.44
Notes: *Slotted pipe which allows inflow of groundwater into the piezometer.				

6.0 GEOHAZARD ASSESSMENT

6.1 Context

Section 106 of the Resource Management Act⁴ (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).

The following sections of this report provide an assessment of the geohazards relevant to the site and form the basis of the Natural Hazards Risk Assessment presented in **Appendix H**. Based on the assessment results, the identified geotechnical hazards can be appropriately managed through the implementation of established geotechnical and engineering mitigation measures, resulting in an acceptable residual level of risk for the proposed development.

6.2 Seismicity

A seismic assessment has been carried out in general accordance with NZGS guidance⁵ to calculate the peak horizontal ground acceleration or PGA (amax) as follows:

$$a_{max} = C_{0,1000} \frac{R}{1.3} \times f \times g$$

Where: $C_{0,1000}$ = unweighted PGA coefficient

R = return period factor given in NZS1170.5, Table 3.5

f = site response factor subject to subsoil class

g = acceleration due to gravity

The ULS PGA was calculated based on a 50-year design life in accordance with the New Zealand Building Code⁶ and importance level (IL) 2 structures, which is considered representative of the proposed industrial buildings and general site infrastructure. The PGA for the serviceability limit state (SLS) and ultimate limit state (ULS) earthquake scenarios is as follows:

⁴ Resource Management Act (1991), as at 29 October 2019

⁵ NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (November 2022)

⁶ Ministry of Business, Innovation and Employment (1992) NZ Building Code Handbook, Third Edition, Amendment 13 (effective from 14 February 2014)

Table 4: Design Peak Ground Acceleration (PGA) for Various Limit States

Limit State	AEP	R	PGA (g)	Magnitude
SLS	1/25	0.25	0.06	5.9
ULS	1/500	1.0	0.25	5.9

Note: SLS = serviceability limit state; ULS = ultimate limit state; AEP = annual exceedance probability

Higher Importance Level classifications apply to specific assets such as the proposed wastewater pump station (IL3) and water reservoir (IL4), for which separate site-specific seismic assessments have been undertaken and are discussed in Sections 6.4 and 7.8.

6.3 Fault Rupture

The nearest known active fault⁷ to site is the Kerepehi Fault, located approximately 35km to the east with a recurrence interval of 2,000 to 3,500 years. Accordingly, the site is considered to be low risk with respect to fault rupture.

6.4 Liquefaction

6.4.1 Methodology

Liquefaction occurs in loose saturated cohesionless soils that are subject to cyclic shear loading during an earthquake. This process leads to pore pressure build-up, soil grains moving into suspension and temporary loss of strength causing vertical and lateral ground deformation.

In accordance with MBIE/NZGS guidance⁸ 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 of 0.5m depth below the low-lying plains and 8.0m depth within the rolling hill topography were modelled as being susceptible to liquefaction.
- A site-specific assessment was carried out using the seismic CPT's to investigate the presence of any soil microstructure in accordance with Robertson⁹. Results in **Appendix I** indicate that "no soil microstructure can be justified and therefore no strength gain factor has been applied.
- 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.
- Free-field liquefaction-induced settlements were estimated in accordance with Zhang et al. (2002). To assess the overall liquefaction response, settlement estimates based on the full depth of each CPT sounding were calculated using a depth-weighting factor (e_v), decreasing linearly from 1.0 at the ground surface to 0.0 at 20 m depth, consistent with the approach outlined by Cetin et al. (2009)¹⁰.

⁷ GNS Science (Te Pu Ao) New Zealand Active Faults Database (NZAFD)

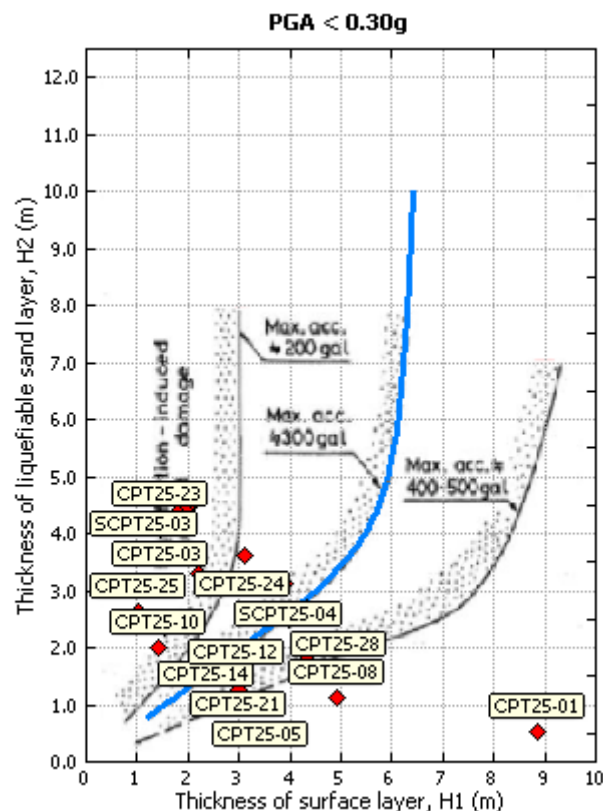
⁸ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

⁹ P.K. Robertson (2015). Comparing CPT and Vs Liquefaction Triggering Methods, Journal of Geotechnical and Geoenvironmental Engineering.

¹⁰ Cetin, K.O., Bilge, H.T., Wu, J., Kammerer, A.M. and Seed, R.B. (2009). Probabilistic Model for the Assessment of Cyclically Induced Reconsolidation (Volumetric) Settlements. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 135(3), 387-398.

- Surface manifestation (ejecta / sand boils) leading to additional settlement through volumetric loss was assessed using the methods outlined in Ishihara (1985)¹², Hutabarat and Bray (2022)¹³. Settlement caused by ejecta is notoriously difficult to estimate, however consideration has been given to the values published in Bray and Olaya (2023) based on the Hutabarat and Bray (2022) ejecta severity of 'Severe'.
- Liquefaction at the proposed wastewater pump station location has been assessed using a 100-year design life and Importance Level 3 (IL3) classification in accordance with the New Zealand Building Code, resulting in a peak ground acceleration (PGA) of 0.32g.
- Liquefaction at the proposed water reservoir location has been assessed using a 100-year design life and Importance Level 4 (IL4) classification in accordance with the New Zealand Building Code, resulting in a peak ground acceleration (PGA) of 0.44g.

Figure 4: Ishihara (1985) Diagram with CPT liquefied layers and surface thickness. Surface manifestation is predicted for CPTs left of the blue line.



6.4.2 Results

Results are presented in **Appendix I** and are summarised as in *Table 5* below:

¹² Ishihara, K. (1985). Stability of Natural Deposits During Earthquakes, Proceedings of the 11th National Conference on Soil Mechanics and Foundation Engineering

¹³ Hutabarat, D., Bray, J.D. (2022). Estimating the Severity of Liquefaction Ejecta Using the Cone Penetrometer Test. Journal of Geotechnical and Geoenvironmental Engineering, 148(3), 04021195.

Table 5: Liquefaction Analysis Results

CPT No.	SLS Settlement (mm)	ULS Settlement IL2 Structures (mm)	ULS Settlement IL3 Structures (mm)	ULS Settlement IL4 Structures (mm)	Depth to Liquefied Layer (m)
CPT25-01	0	25	-	-	3.0
CPT25-02	0	90	-	-	2.0
CPT25-03	0	95	-	-	1.5
CPT25-04	0	90	-	-	1.2
CPT25-05	0	50	-	-	2.2
CPT25-06	0	70	-	-	2.0
CPT25-07	0	75	-	-	1.5
CPT25-08	0	80	-	-	1.5
CPT25-09	0	120	-	-	1.4
CPT25-10	0	80	120	-	1.2
CPT25-11	0	75	-	-	1.4
CPT25-12	0	90	-	-	1.0
CPT25-14	0	45	-	-	2.4
CPT25-15	0	45	-	-	1.3
CPT25-16	0	80	-	-	1.2
CPT25-17	0	<5	-	-	-
CPT25-18	0	<5	-	-	-
CPT25-19	0	<5	-	5	11.0
CPT25-20	0	<5	-	10	10.5
CPT25-21	0	70	-	-	1.5
CPT25-23	0	120	180	-	1.0
CPT25-24	0	85	-	-	1.5
CPT25-25	0	70	-	-	1.0
CPT25-27	0	105	-	-	1.0
CPT25-28	0	65	-	-	1.5
SCPT25-01	0	40	170	-	1.2
SCPT25-02	0	75	250	-	1.0
SCPT25-03	0	100	-	-	1.2
SCPT25-04	0	70	-	-	1.5

Notes: All settlements and depths based on existing ground profile.

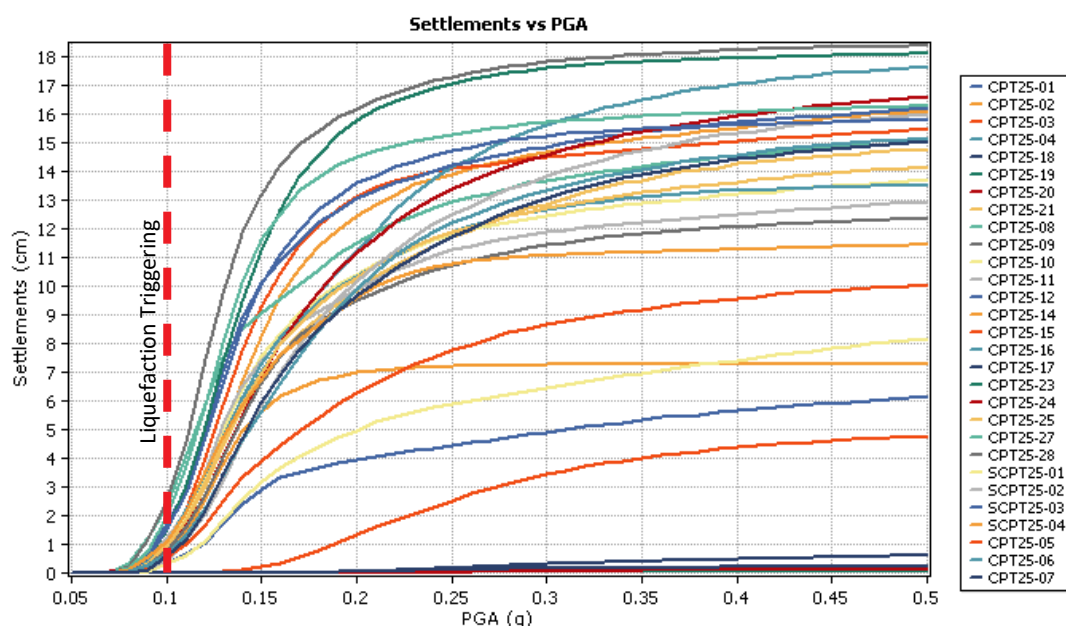
Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied.

CPT25-13, CPT25-22 and CPT25-26 were unable to be undertaken due to site restraints and are hence not included in table above.

Recommendations to mitigate effects of liquefaction settlements on the proposed development are provided below in **Section 7**.

Parametric analyses undertaken using CLIQ software indicate that the threshold for the onset of liquefaction-induced settlement occurs at ground shaking levels of approximately 0.1g, as shown in **Figure 5**.

Figure 5: Parametric Analysis



6.5 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.

Several wetlands, swales, and sections of the Mangaheka Stream are proposed to be excavated below existing ground levels, creating free faces across portions of the development that may be susceptible to lateral spreading following liquefaction. The potential for, and estimated magnitude of, lateral spread beneath the site has been assessed with respect to liquefied shear strengths ($Su_{(liq)}$) within the liquefied soil units using a Newmark Sliding Block approach as discussed in **Section 6.6** below.

6.6 Slope Stability

6.6.1 Design Criteria

The stability of cut batters and fill embankments under a range of design conditions is expressed in terms of a factor of safety, which is defined as the ratio of forces resisting failure to the forces causing failure. The following performance standards have been adopted from the Waikato Regional Infrastructure Technical Specification¹⁴ for slope stability assessment:

¹⁴ Regional Infrastructure Technical Specifications (RITS) V2 2025. Co-Lab. <https://colabsolutions.govt.nz/shared-services/rits-regional-infrastructure-technical-specifications/rits-v2-2025/>

Table 6: 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 pore pressure conditions)	1.3
Ultimate Limit State (ULS) seismic condition	1.0*
Note: *Factor of safety < 1.0 acceptable where displacement-based approach is adopted.	

6.6.2 Shear Strength Parameters

6.6.2.1 Effective Stress Parameters

Drained shear strength parameters for the various geological units that support the existing natural escarpments below the site were inferred from the field investigation and extensive experience with similar soils in the same geological units.

6.6.2.2 Total Stress Parameters

The soils include cohesive silts and clays that will behave in an undrained state during short term seismic loading. Undrained soil shear strengths (S_u), used for assessing the stability of slopes during seismic loading, were taken from the handheld shear vane results and inferred from the CPT data based on the following relationship:

$$S_u = \frac{qc - \sigma}{Nk}$$

Where: qc = CPT cone resistance (kPa)

σ = total overburden pressure (kPa)

Nk = factor, typically between 10 and 20, 20 was conservatively adopted for this site

Undrained shear strength correlations from the CPT and vane shear strength testing gave a range of values across the site. Lower bound values have been used.

The selected effective stress and undrained shear strength parameters used in our analyses are presented in Tables 7 and 8 below:

Table 7: Summary of Geotechnical Parameters – Rolling Hills

Geological Unit	Unit Weight (kN/m ³)	Friction Angle ϕ' (°)	Cohesion, c' (kPa)	Undrained Shear Strength, S_u (kPa)	Vertical Stress Ratio, Minimum Shear Strength (kPa)
Very stiff to hard Clay / Silt – Hamilton Ashes	16	28	4	80	-
Stiff to very stiff Clay / Silt – Kauroa Ashes	16	28	5	100	-
Firm to very stiff Clay / Silt – Puketoka Formation	16	28	3	80	-
Dense to very dense Sand – Walton Subgroup	17	36	0	-	-

Table 8: Summary of Geotechnical Parameters – Low-lying Plains

Geological Unit	Unit Weight (kN/m ³)	Friction Angle ϕ' (°)	Cohesion, c' (kPa)	Undrained Shear Strength, S_u (kPa)	Vertical Stress Ratio, Minimum Shear Strength (kPa)
Very stiff to hard Clay – Engineered Fill	18	28	5	140	-
Firm Silt – Piako Subgroup	16	28	1	30	-
Stiff to very stiff Silt – Hinuera Formation	16	28	2	40	-
Medium dense to dense gravels / sandy gravels – Hinuera Formation	16	36	0	-	-
Loose to medium dense Sands with silt interbedded – Hinuera Formation	16	34	2	-	5
Medium dense to dense Sand – Hinuera Formation	16	36	0	-	-

6.6.3 Slope Stability Analysis

Slope stability analyses were undertaken using the Morgenstern-Price method of slices under both circular and translational failure mechanisms using the proprietary software SLIDE Version 6. Seismic displacements were estimated based on a Newmark Sliding Block approach using 50th percentile correlations published in Bray (2007)¹⁵, Bray et al (2018)¹⁶ and Bray & Macedo (2019)¹⁷ for liquefied soil conditions.

For the Newmark Sliding Block calculation, a range of depths outlined in Table 9 for the critical slip surface and average shear wave of sliding mass of 114m/s was estimated based on CPT investigation data. Bounds on the slip surface search were applied in the seismic case in accordance with CalTrans¹⁸ guidelines.

The analyses represent the unmitigated conceptual design landform and were undertaken to identify areas requiring mitigation. Selected stability printouts are attached in **Appendix J** and summarised as follows:

Table 9: Slope Stability Analyses Results

Location (Proposed Profile)	Slope Stability Factor of Safety			Seismic Yield Acc (pga)	Maximum Height of Sliding Mass (m)	ULS Displacement (mm)
	Prevailing	Transient	Seismic			
Slope Stability Section 01 – Northern Hills	1.04	0.9	1.44	-	-	-
Slope Stability Section 02 – Northern Hills	0.92	0.6	1.18	-	-	-
Slope Stability Section 03 – Northern Hills	2.49	2.78	1.39	-	-	-

¹⁵ Bray, J. D. (2007). Simplified seismic slope displacement procedures. In *Earthquake geotechnical engineering* (pp. 327-353). Springer, Dordrecht.

¹⁶ Bray, J.D., Macedo, J., and Travasarou, T. (2018) "Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes," J. of Geotechnical and Geoenvironmental Engineering, ASCE, V. 144(3): 04017124, DOI: 10.1061/(ASCE)GT.1943-5606.0001833.

¹⁷ Bray, J.D., and Macedo, J. (2019) "Procedure for Estimating Shear-Induced Seismic Slope Displacement for Shallow Crustal Earthquakes," J. of Geotechnical and Geoenvironmental Engineering, ASCE, V. 145(12), doi: 10.1061/(ASCE)GT.1943-5606.0002143.

¹⁸ California Department of Transportation. (2017). Lateral spreading analysis for new and existing bridges (Memo to Designers 20-15). California Department of Transportation

Location (Proposed Profile)	Slope Stability Factor of Safety			Seismic Yield Acc (pga)	Maximum Height of Sliding Mass (m)	ULS Displacement (mm)
	Prevailing	Transient	Seismic			
Slope Stability Section 04 – Wetland 7	2.37	1.74	0.38	0.05	5.2	70
Slope Stability Section 05 – Eastern Swale Neighbouring Road Embankment**	2.87	2.35	0.52	0.12	14.0	50
Slope Stability Section 06 – Wetland 8	2.01	1.41	0.23	Flow failure		
Slope Stability Section 07 – Wetland 4	1.90	1.43	0.26	Flow failure		
Slope Stability Section 08 – 2.0m Deep Typical Boundary Swales	2.19	1.71	0.88	0.13	4.2	15
Slope Stability Section 09 – Wetland 5 / Mangaheka Stream	2.86	2.09	0.32	Flow failure		
Slope Stability Section 10 – Reservoir (L to R)	1.60	1.53	0.74	0.27	15.0	<5
Slope Stability Section 10 – Reservoir (R to L)	1.68	1.56	0.85	0.34	15.0	<5
Slope Stability Section 11 – Northwestern Embankment	2.20	1.79	1.41	-	-	-
Notes: *Bray(2007) model used to calculate displacements as per NZTA guidelines where stability sections neighbour SH1C. ** Results are for the existing slope profile						

Specific earthworks design requirements to achieve appropriate slope stability factors of safety are presented are outlined in **Section 7.3** below.

6.7 Load Induced Settlement

Static settlements were estimated based on the CPT data using the methodology outlined below. Calculations were undertaken using the commercially available software CPeT-IT.

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

Where $\Delta\sigma_v$ = change in total vertical stress, Δz = layer thickness, M_{CPT} = constrained modulus estimated from the CPT data as follows:

$$M = \alpha_M (q_t - \sigma_{vo})$$

Where: α_M = factor, derived according to Robertson (2009)¹⁹; q_t = cone resistance; σ_{vo} = total vertical stress.

Table 10 below shows estimated foundation settlements for anticipated lightweight industrial buildings with 20kPa floor loadings. Fill load has been added to the CPTs based on the cut and fill drawings for each investigation location. Outputs are presented in **Appendix K**.

¹⁹ Robertson, P.K., 2009. Interpretation of cone penetration tests – a unified approach. Canadian Geotechnical Journal, 46:1337-1355.

Table 10: Load Induced Settlement Analyses Results

Test Location	Proposed Fill (m)	Applied Working Pressure (kPa)	Fill Induced Primary Settlement (mm)	Fill Induced Secondary Settlement 12 Months (mm)	Fill Induced Secondary Settlement 100 years (mm)	Foundation Load Primary Settlement (mm)	Foundation Load Secondary Settlement 50 years (mm)
CPT25-03	2.0	56	20	-	<5	10	-
CPT25-06	2.0	56	100	5	20	50	20
CPT25-08	1.0	38	30	5	15	30	15
CPT25-10	1.0	38	10	-	<5	10	-
CPT25-11	1.0	38	10	-	<5	20	-
CPT25-12	2.0	56	25	-	<5	15	5
CPT25-14	4.0	72	70	5	15	No building load anticipated	
CPT25-15	4.0	72	70	-	10	No building load anticipated	
CPT25-16	4.0	72	100	-	20	No building load anticipated	
CPT25-21	1.0	38	60	5	30	60	30
CPT25-24	2.0	56	20	-	<5	5	-
CPT25-25	1.0	38	5	-	<5	10	-
CPT25-27	1.0	38	15	-	<5	10	-
CPT25-28	2.0	56	15	-	<5	5	-
SCPT25-01	2.0	56	25	-	<5	5	10
SCPT25-02	1.0	38	15	-	<5	5	5
SCPT25-03	1.0	38	15	-	<5	10	-
SCPT25-04	3.0	74	70	5	20	-	15

Notes: Applied working pressure assumed 18kN/m³ unit weight for engineered fill and a 20kPa working load spread across a 20.0m length/width from future foundations.

Settlement estimates are based on CPTs at discrete locations and should be updated as part of detailed design / Building Consent stage for individual structures.

New Zealand Building Code, Clause B.1.0.2 of B1/VM4 provides the following differential settlement criteria for design of shallow foundations: "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 1170 Part 0, unless the structure is specifically designed to prevent damage under a greater settlement."

For the foundation size and working load combinations considered in Table 10 above, settlement across the site is expected to be variable. However, it is also expected, based on the low-lying plains soil conditions, for most settlement to occur immediately and generally within the construction period and cease 3 to 6 months following construction. Residual vertical settlement and differential settlement will need to be accounted for in the design of future structures.

In areas of cut underlain by clays, some ongoing consolidation settlement is still anticipated as the exposed clay benches adjust to reduced overburden and new fill loads, with movements likely to occur gradually over several months and requiring allowance for residual and differential settlement in foundation design.

6.7.1 Reservoir Fill Placement

To facilitate construction of the proposed water reservoir, in the northeastern portion of the site, up to 9.5m of engineered fill will be required to establish a suitable level building platform for the reservoir tanks. The proposed fill footprint extends across the low-lying plains adjoining the rolling hill terrain in the northeastern portion of the site. Two hand-auger boreholes were advanced within this area to verify the underlying ground conditions. The investigations encountered low-strength, compressible silt deposits to depths of approximately 4.0m below existing ground level, which are notably different from the predominantly sandy soils present throughout most of the adjacent low-lying plains within the development area.

A preliminary settlement assessment was undertaken using Settle3 software, which indicates that total settlements of up to 700mm may occur beneath the proposed fill loading.

6.8 Groundwater Drawdown Induced Settlement

As part of the consenting process, Wallbridge Gilbert Aztec (WGA) prepared an assessment of the potential groundwater drawdown associated with the construction of the proposed wetlands and swales (Te Kowhai East Industrial Development Hydrogeological Effects Assessment, dated August 2026).

Analytical modelling undertaken by WGA indicates that the greatest off-site permanent groundwater drawdown is expected to occur following construction of the stormwater swales and deepening of the Mangaheka Stream. The modelling predicts that groundwater levels beneath Koura Drive could be lowered by up to 0.75m below the typical summer groundwater minimum.

Groundwater drawdown induced settlement was assessed considering effective stress change (increase) due to lowering of groundwater levels and resulting soil load increase on soils below the groundwater table. A conservative groundwater drawdown of 1.0m and a representative soil elastic modulus of 5MPa have been adopted. The resulting estimated settlements are in the order of 5–10mm (see assessment in **Appendix K**). Based on our experience, settlements of this magnitude would generally be regarded as minor and are not anticipated to adversely affect existing infrastructure or neighbouring land. As such, cross-boundary effects are considered negligible and no specific mitigation measures are recommended at this stage.

The assessment is intentionally conservative and represents a screening-level estimate suitable for consenting purposes.

6.9 Sensitive Soils

6.9.1 Rolling Hill Topography

As part of the proposed bulk earthworks, a significant proportion of the material excavated from the northern hill cut is expected to comprise highly sensitive Puketoka Formation silts. This unit is encountered from approximately 4.0 m depth and will be exposed throughout the lower 6.0 m of the planned cut. Due to the very sensitive nature of these silts, their excavation and handling will be challenging and will require careful management of plant movements wherever they are exposed. Further recommendations are provided in **Section 7**.

6.9.2 Low Lying Plains Topography

The Hinuera Formation silt unit present across the site, and encountered within the upper 1.0m, is typically considered moderately sensitive to sensitive. These characteristics may make the silt unit challenging to earthwork and will require special consideration of plant movements during the construction period where exposed. Further recommendations are provided in **Section 7** below.

6.10 Acid Sulphate Soils

A Waikato Region Acid Sulphate Soils Preliminary Risk Assessment report was prepared by GHD (Ref. 12571919, dated 17 May 2024). The GHD report included assessment of different geological environments within the Waikato Region that may be susceptible to acid sulphate soil occurrence and to provide a regional-scale assessment of associated risk. Risk maps included in the report were based on published geology mapping indicating the low lying plains at this site to be underlain by Piako Subgroup (Holocene Aged) alluvial deposits which often include organic soils / peat. Accordingly, the GHD report considered this area being of high probability of occurrence.

However, the GHD report identifies Late Pleistocene fluvial deposits comprising cross-bedded pumice sands, silts, and gravels with minor interbedded peat as having a low probability of acid sulphate soil occurrence. Based on the findings of the site investigation across the low-lying plains, the subsurface profile was found to comprise predominantly Hinuera Formation soils consisting of cross-bedded sands and silts. Accordingly the risk of encountering acid sulphate soils within the development area is considered very low.

Given the consistency of the investigation findings with the published geological interpretation and the absence of extensive estuarine or marine-derived sediments, acid sulphate soils are not considered a significant geotechnical or environmental constraint to the proposed development.

Only limited localised organic deposits were found to be associated with the southern paleochannel system. The southern road link off Te Kowhai Road will require a culvert over this feature that will likely be interacting with localised shallow peat soils. Management of potential for acid sulphate soils in this area will be via onsite soil testing (pH) and isolation of material if required. This is consistent with acid sulphate risk management undertaken for the Rotokauri Greenway project, which lies within the same drainage catchment as this development.

7.0 GEOTECHNICAL RECOMMENDATIONS

7.1 Seismic Site Subsoil Category

The geological units encountered beneath the development areas comprise soil strength materials, which, with respect to the seismic site subsoil category defined in Section 3.1.3 of NZS1170.5, is defined as having a UCS < 1MPa. Therefore, the seismic site subsoil category is assessed as being Class D (deep soil site).

7.2 Liquefaction Mitigation

Based on the analysis results presented in **Section 6.5** above, we consider the risk of liquefaction and liquefaction induced settlements to be moderate to severe under the ULS case.

For industrial and commercial buildings liquefaction effects can be mitigated readily with specific engineering design options such as:

- Structural design of the proposed superstructures to sustain the calculated amount of liquefaction induced ground deformation and lateral spreading;

- Ground remediation measures, including but not limited to the densification, strengthening, and stabilisation of underlying liquefiable soils through deep soil mixing, stone columns, compaction grouting, dynamic compaction, drainage enhancement, or other appropriate ground improvement methods to reduce liquefaction susceptibility;
- Foundation options such as (but not limited to) waffle slab foundations, designed to sustain the calculated liquefaction induced ground deformation and lateral spreading, further details are included in **Section 7.7**.

Calculated liquefaction magnitudes can possibly be reduced by performing further laboratory testing to assess the fines content and plastic nature of the fine-grained soils at the site, and to account for the pumice content of the coarse-grained soils at the site.

7.3 Slope Stability Management – Northern Hills

The proposed development requires substantial regrading of the northern hill landform to create the planned development platforms. To achieve the proposed geometry while maintaining appropriate long-term stability, engineered retaining structures will be required along portions of the northern boundary. Geotechnical stability assessments were undertaken using a series of representative cross-sections through the retained landform to evaluate performance under long-term static, transient groundwater, and seismic loading conditions. Additional analyses were completed to assess the stability of temporary (short term) excavation faces during construction and to confirm the feasibility of staged earthworks and retaining wall installation.

Results of these analyses are presented in Table 11 below:

Table 11: Rolling Hills- Slope Stability Analyses Results

Remedial Works / Batter Gradient	Slope Stability Factor of Safety			
	Prevailing (Long-term)	Transient	Seismic	Undrained Short Term
Slope Stability Section 01 – Cantilever Retaining Wall	1.73	1.63	1.55	-
Slope Stability Section 02 – Cantilever Retaining Wall	1.50	1.36	1.44	-
Slope Stability Section 02 – Temporary Works Case	-	-	-	2.04

At the completion of the development, **Specific Design Zones (slope) or Specific Design Zones (retaining walls)** are expected to be applied in the Geotechnical Completion Report to protect future building development from long term creep effects on or adjacent to steep slopes and to protect the slopes / retaining walls from inappropriate loading or undermining. In this geology these are typically applied where gradients steeper than 1v:3h are present. Where required at the toe of a slope, limitations will also be required relating to location, depth and timing of any future excavations.

7.4 Lateral Spread Mitigation

7.4.1 Results

Results of the slope stability analyses discussed in **Section 6.6** above demonstrate that design landform gradients through the proposed development will not achieve the requisite slope stability factors of safety for the swales, wetlands and Mangaheka Stream widening in terms of liquefaction displacement under a ULS seismic event.

Several representative worst-case scenarios were modelled in SLIDE to evaluate potential mitigation measures and establish preliminary design requirements for managing lateral spread displacements. The results indicate that the predicted displacements can be reduced to negligible levels through appropriate engineering design. Further assessment and optimisation of the preferred mitigation measures will be required during detailed design once the final development layout and infrastructure design have been confirmed.

Table 12: Wetlands & Swales - Slope Stability Analyses Results

Remedial Works / Batter Gradient	Slope Stability Factor of Safety			Seismic Yield Acc (pga)	ULS Displacement (mm)
	Prevailing	Transient	Seismic		
Slope Stability Section 04 – Wetland 7 Mitigation with 5 Rows of 0.6m Dia CFA Piles (~12.0m Deep)	6.57	6.03	1.04	-	-
Slope Stability Section 06 – Wetland 8 Mitigation with 5 Rows of 0.6m Dia CFA Piles (~15.0m Deep)	4.93	4.55	1.06	-	-
Slope Stability Section 07 – Wetland 4 Mitigation with with 4 Rows of 0.6m Dia CFA Piles (~11.0m Deep)	5.55	2.76	1.29	-	-
Slope Stability Section 05 – Eastern Swale Neighbouring Road Embankment	2.38	2.38	0.60	0.13	30
Slope Stability Section 08 – Typical 2.0m Swale Mitigation with 1.0m Thick Engineered Clay Cap ¹	2.63	2.11	1.13	-	-
Slope Stability Section 09 – Geogrid Mitigation below Engineered Fill / Pavements adjacent to Mangaheka Stream	2.75	2.09	0.32	0.08	30 ²
Notes: 1. 2.0m deep swale is worst case so applicable for all swales along site boundary. 2. Additional geogrid layers or increased geogrid tensile strength may further reduce or eliminate potential lateral displacements. 3. CFA Piles will be required to be installed in triangular lattice pattern – see Appendix J for preliminary drawing					

7.4.2 Wetlands Bordering External Boundaries

The representative mitigation analyses presented in Table 12 for the wetlands were undertaken using deep CFA piles as a preliminary design scenario to demonstrate that liquefaction-induced lateral displacements adjacent to the proposed wetlands and swales can be reduced to negligible levels through appropriate engineering mitigation. The results confirm that satisfactory seismic performance can be achieved and that any potential effects beyond the site boundaries can be reduced to negligible levels.

CFA piles represent only one of several potential mitigation options available for the site. Alternative measures may include, but are not limited to, sheet pile walls, shear keys, rock buttresses, engineered clay liners, geogrid reinforcement beneath pavements and fills, or combinations of these techniques. The preferred mitigation approach should be selected during detailed design following confirmation of the final development layout, asset locations, and performance requirements.

7.4.3 Swales Bordering External Boundaries

The proposed development includes swales along portions of the site boundaries, typically ranging between 1.0m to 2.0m deep. Stability analysis results indicate that, in order to reduce predicted displacements to

negligible levels in the worst-case scenarios, lining these swales with low-permeability engineered clay fill significantly reduces liquefaction-induced lateral spreading.

7.4.4 Swale Neighbouring Road Embankment

The proposed development includes a swale along the eastern boundary adjacent to the existing road embankment. Stability analyses indicate that lining the swale with low-permeability engineered clay fill provides a beneficial reduction in liquefaction-induced lateral spreading, resulting in lower predicted lateral displacements than those currently predicted for the existing site conditions.

7.4.5 Wetlands and Swales Within the Development Site

For wetlands and swales internal to the development, the predicted lateral displacements can be managed through a combination of appropriate geotechnical and structural design measures. In areas where lateral spreading is anticipated, adjacent structures, buried services, pavements, and other infrastructure should be designed to accommodate the calculated ground deformations associated with the ULS seismic event.

Where required, geogrid reinforcement may be incorporated beneath engineered fills, road and lot pavements, and structures to reduce liquefaction-induced lateral spreading and maintain the serviceability of supported infrastructure. While geogrid reinforcement can significantly reduce deformations within the reinforced zone, it does not prevent ground movements from occurring in surrounding unreinforced areas. Consequently, wetland margins, swale batters, and other landscape features may still experience localised displacement or instability during a major seismic event. These features may therefore be treated as unmitigated elements, with post-event inspection and minor remediation undertaken as necessary to restore functionality. Provided appropriate mitigation measures are implemented, the anticipated ground movements are not expected to adversely affect the performance or ongoing operation of adjacent engineered infrastructure.

Calculated lateral spread magnitude can potentially be reduced by performing further laboratory testing to assess the liquefaction susceptibility by testing fines content and plasticity of the fine-grained soils, and pumice content testing of the coarse-grained soils at the site.

The areas described above are illustrated on the indicative plan presented as Drawing 06 in **Appendix A**.

7.5 Static Settlement

7.5.1 Buildings

In our experience, preliminary settlement estimates from CPT testing can be conservative and reviewing settlement monitoring data is required to validate / correct estimated static settlement values. Hence, we recommend an instrumented fill-induced settlement monitoring approach during bulk earthworks. Adequate monitoring duration is required to enable a full assessment of settlement trends. Settlement will be considered adequately mitigated once 90 percent of primary consolidation has been reached (known as T_{90}), and the resulting creep settlements are predicted to be below 25mm over the design life of proposed structures (assuming widespread 20kPa uniformly distributed floor loads).

For buildings, refinement of the settlement analysis may be carried out at a building design / consent stage.

Alternatively, the foundations and superstructures may be designed to accommodate the post construction settlements outlined within Table 10 in **Section 6.7** above.

7.5.2 Buried Infrastructure

Primary settlement induced by the proposed engineered fills is estimated to be in the order of 5–100 mm, with movement expected to occur within 3–6 months of placement. Buried services will need to be detailed to accommodate differential settlement along their alignments or installed after primary settlement has occurred. Settlement monitoring will be required to measure actual fill induced settlement magnitudes.

Residual settlements occurring following completion of primary consolidation are estimated to range between 5mm and 30mm across the development area over a 100-year period. Where settlement-sensitive assets are proposed, monitoring data should be used to confirm that post-construction ground movements have reduced to acceptable levels prior to construction and will remain within the tolerances adopted for the asset's design life. If this cannot be demonstrated, mitigation measures such as preload surcharging, ground improvement to reduce differential settlements may be adopted to reduce settlements to acceptable levels.

7.5.3 Cross Boundary Effects

Cut earthworks are generally proposed along the site boundaries, with only minor filling required in limited boundary areas to tie into existing ground profiles. The scale of filling proposed adjacent to neighbouring properties is small relative to the overall development and is not expected to generate significant additional loading on adjoining land. The proposed earthworks have been configured such that finished ground levels generally tie into existing topography without the need for substantial perimeter embankments. Subject to construction in accordance with the recommended geotechnical controls and NZS 4431:2022, no specific mitigation measures are considered necessary to manage cross-boundary effects associated with fill placement.

7.6 Wastewater Pump Station

For the proposed wastewater pump station (WWPS) located in the western part of the site, seismic assessment indicates that total vertical liquefaction-induced settlements across the WWPS footprint could range from approximately 120mm to 250mm during the ULS seismic event. The design of the wastewater pump station should account for both total and differential settlements between individual pump station components. Where differential settlements cannot be adequately accommodated through structural design, deep ground improvement methods should be considered.

Wastewater pump station structures should also be designed to resist liquefaction-induced buoyancy effects. During seismic events, excess pore water pressures may develop within liquefiable soils, increasing the buoyancy forces acting on buried structures such as wet wells, deep manholes, and storage tanks. These pressures can be mitigated through the installation of free-draining, gap-graded aggregate (e.g. 20/40mm drainage metal) around the structures, with suitable outlets connected to the stormwater network or another approved discharge location.

Static settlement is typically not an issue for WWPS inground, and light-weight above-ground, structures, but will require assessment as part of detailed design.

The recommendations outlined above are preliminary and subject to confirmation and refinement during the detailed design phase at the Building Consent stage for the wastewater pump station. Based on the assessments completed to date, the identified geotechnical risks are considered manageable through established engineering measures, including appropriate structural design, drainage provision, and, where required, ground improvement techniques.

7.7 Foundations

On this site, our provisional expectation is that, provided earthworks are completed in accordance with the relevant standards, the following will apply:

- A preliminary ultimate bearing pressure of 300kPa should be available in the static case for shallow strip and pad foundations measuring at minimum 1.0m width constructed within both the natural cut ground across and engineered fill.

Geotechnical ultimate bearing pressure in the ULS seismic case will be >200kPa based on the shallow liquefiable layers and assuming a minimum non-liquifiable crust is present below foundations as encountered within the investigation.

- There may be areas where localised variations in shear strength within the natural 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 and will be presented in a Geotechnical Completion Report.
- Specific structural design of proposed buildings is required to sustain the calculated amount of liquefaction induced ground deformation and lateral spreading where adjacent to stormwater swales / basins.
- As required by Section B1/VM4²⁰ of the New Zealand Building Code Handbook, the following strength reduction factors must be applied to all recommended geotechnical ultimate soil capacities in conjunction with their use in factored design load cases:
 - 0.8 for load combinations involving earthquake overstrength;
 - 0.5 for all other load combinations.

7.8 Reservoir

7.8.1 Slope Stability

Results of the slope stability analyses discussed in **Section 6.6** indicate that the proposed reservoir landform achieves the required factors of safety under both long-term and elevated groundwater conditions. While the calculated seismic factor of safety is less than 1.0, displacement-based analysis indicates that seismic movements are expected to be negligible.

7.8.2 Fill Induced Settlement

The preliminary settlement assessment presented in **Section 6.7** indicates that vertical settlements of up to 700mm may occur following the placement of the proposed engineered fill beneath the reservoir platform. Should this site be selected as the preferred location for the reservoir tanks, appropriate measures will need to be considered during the detailed design phase to mitigate and manage predicted settlements, ensuring they remain within acceptable limits for the proposed structure.

To minimise post construction static ground settlements, a range of options may be considered, including the following:

Construction of a temporary surcharge or pre-load fill embankment above design finished level, with or without wick drains, to over-consolidate the compressible soils and minimise post construction embankment settlements to acceptable limits;

²⁰ Ministry of Business, Innovation and Employment (2019) *Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure, B1/VM4, Amendment 19*.

Undercutting compressible soils and replacement with engineered fill;

Undertake ground improvement beneath the embankment and/structure footprints, such as stone columns, soil mixed columns, CFA piles, Rammed Aggregate Piers (RAP's) or similar rigid inclusions to transfer loads from the embankment / tanks to more competent underlying soils at depth.

7.8.3 Liquefaction

Preliminary liquefaction assessment undertaken for an Importance Level 4 (IL4) reservoir indicates that liquefaction-induced settlements are expected to be negligible. Accordingly, no specific liquefaction mitigation measures are anticipated to be required.

7.8.4 Further Work

The conceptual reservoir configuration presented on the preliminary Maven drawings is considered technically feasible to construct in its current form and is not expected to adversely affect neighbouring properties through cross-boundary geotechnical effects.

Selection of this site as the preferred reservoir location will require detailed geotechnical investigation and engineering design. Investigations may include, but are not limited to, deep machine borehole drilling, deep CPT testing, and site-specific seismic investigation and assessment.

7.9 Earthworks

7.9.1 Excavatability

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431:2022 and under the guidance of a Chartered Professional Geotechnical Engineer.

The Hinuera Formation silts and Puketoka Formation silts and clays are sensitive to remoulding during earthworks. These soils rapidly lose strength if overworked, over trafficked or allowed to get excessively wet. Mitigation options can include avoiding directly trafficking over these soils, limiting the area exposed to water infiltration at any one time, shaping and compacting the cut surface to allow water to runoff rather than pond. Disturbed soils may regain strength if left to settle for a period of days in fine weather, or they may need to be undercut and replaced with appropriately conditioned materials.

7.9.2 Stockpiling

Careful consideration must be given to the location of temporary topsoil / unsuitable stockpiles to ensure that they are not located immediately above steep or unstable slopes or immediately adjacent to proposed stormwater pond excavations.

The location of all temporary stockpiles must be approved by the Geotechnical Engineer prior to placement. Where stockpiles cannot be avoided above potentially unstable ground, they should be placed over a wide area with the height restricted under the direction of the Geotechnical Engineer.

7.9.3 Compaction

As part of the proposed bulk earthworks, the material excavated from the rolling hill topography will consist of an approximately 4m thick mantle of volcanic ashes in turn underlain by up to 6m of Puketoka Formation sensitive silts and clays. Excavation of the soils that will be encountered within the proposed earthworks cuts are expected to be readily achieved with normal earthworks plant, such as scrapers and bulldozers/tractors with scoops.

It is expected that Puketoka Formation sensitive silt will require blending with more suitable material with careful placement and compaction. Experience on similar projects throughout the Waikato indicates blends of 40% Puketoka Formation with 60% non-sensitive volcanic ashes can provide good handling characteristics, enabling placement and compaction without prolonged drying periods. Trials are required to confirm the effectiveness and practicality of soil blends.

All earthfill must be placed to ensure adequate knitting of successive fill lifts by ripping any natural subgrade or fill surfaces that have become dry prior to placing the following fill lift.

A project specific geotechnical works specification including compaction control criteria will be required prior to undertaking bulk earthworks. This specification should form part of the earthworks tender package for clarity of performance requirement.

7.9.4 Uncontrolled Fill

During the site investigation, several isolated areas of shallow uncontrolled fill, typically less than 0.5m thick, were encountered within the boreholes. These deposits are considered to be associated with historic farm activities, such as the construction of farm races.

During site preparation works, following the stripping of topsoil, the extent and depth of any uncontrolled fill should be exposed and inspected by a geotechnical engineer. Under geotechnical supervision, all uncontrolled fill and other materials considered unsuitable, or unsuitable to remain beneath proposed fills, should be removed. Excavation depths should be sufficient to ensure the complete removal of such materials.

7.9.5 Erosion

Proposed excavation of the Mangaheka Stream alignment, swale formation and associated wetlands is anticipated to encounter saturated sandy soils that may be susceptible to “running sand” conditions during construction. Temporary groundwater management measures, including localised dewatering, are expected to be required to facilitate excavation and maintain stable working conditions. With appropriate dewatering and earthworks management practices in place, the risks associated with running sands can be effectively mitigated.

In the long term, the swales are expected to function as a low-energy drainage corridor (Maven Associates Stormwater Drawings ref. 180010-C400 Set), and therefore erosion is not considered a significant geotechnical risk. Accordingly, the potential for geotechnically significant erosion or scour is considered low under normal operating conditions. Establishment of topsoil and robust vegetation cover across exposed soil surfaces is expected to provide adequate long-term erosion protection. Where detailed civil design identifies localised areas of elevated flow velocity, concentrated discharge, or steeper channel gradients, additional erosion control measures such as riprap armouring, reno mattresses, rock buttressing, geotextile reinforcement, or similar engineered protection systems may be incorporated to mitigate scour and erosion risks.

7.10 Road Subgrades

The proposed roading is shown to be in a combination of both cut and fill areas. Based on DCP results in in-situ soils, a lower bound subgrade equivalent CBR value of 2 to 3% is recommended for preliminary design within the surface silts present across the low-lying plains. For the underlying sandy soils, a higher equivalent CBR value of 3 to 4% is recommended for preliminary design. A higher CBR will likely be available for areas with engineered fill, subject to further testing.

7.11 Stormwater Soakage

The soils underlying the low-lying plains comprise variable silts and sands that exhibit hydraulic conductivities in the order of 9.8×10^{-6} to 1.56×10^{-4} m/s, based on the CIRIA-113 falling-head permeability results. These materials are considered suitable to provide a seepage pathway for rain-garden or attenuation-pond design. However, the effectiveness of soakage is expected to be constrained by the shallow groundwater table that persists across the plains.

Underlying the rolling hill topography, the soils are clayey in nature and are expected to have very low coefficients of permeability. Accordingly, soakage systems are not expected to provide any significant ground soakage function.

7.11.1 Trench Excavations

All of the materials to be exposed during the excavation of service trenches should be readily removed using an excavator.

Trench collapse is expected to pose problems across the low-lying plains as excavations extend below the water table at approximately 1m depth and here, trench support is likely to be required. Temporary dewatering, in the form of regularly spaced sump pumps or well point dewatering spears may also be required.

8.0 SUMMARY OF RECOMMENDATIONS

- Liquefaction within Hinuera Formation soils is expected in a major earthquake, with moderate ground deformation.
 - This can be managed through appropriate structural and foundation design.
 - Further lab testing may refine the liquefaction assessment.
 - Localised lateral displacements may occur adjacent to the main swale and wetland features during a ULS seismic event. Where required, these displacements can be managed through appropriate engineering mitigation measures as outlined in Section 7.4.
- Static settlement may occur under new fills.
 - Settlement monitoring during earthworks is recommended to confirm when ground movements have stabilised prior to civil and building construction commencing.
 - Foundations can alternatively be designed to tolerate the predicted settlements.
- Stormwater soakage is feasible, with soil permeability varying across the site.
 - Final stormwater design must consider soakage rates, groundwater levels, and soil layering.
- Foundations are expected to achieve suitable bearing capacity once earthworks are completed.
- Earthworks must follow NZS 4431 and Waikato Regional Council requirements under geotechnical supervision and subject to a project specific specification.
 - Some site soils (Hinuera Formation and Puketoka Formation silts) are sensitive and lose strength when wet or over-worked.
 - Blending sensitive silts with volcanic ash may be required to achieve suitable fill quality.
 - Groundwater may be encountered at shallow depth, requiring drainage measures.
- Roadway subgrade should be designed using a preliminary CBR value of 3% for natural soils, with higher values expected in engineered fill.

- The proposed reservoir location is considered generally suitable from a geotechnical perspective, subject to appropriate geotechnical design and mitigation measures.
- The proposed wastewater pump station location is considered generally suitable from a geotechnical perspective. Detailed design should account for liquefaction-induced settlement, differential movement and seismic buoyancy effects, which can be managed through appropriate structural design, drainage measures and, where required, ground improvement.
- Acid sulphate soil risk is considered low across the site, and any localised risks can be readily managed through typical construction and environmental controls.

9.0 FUTURE WORK

No further geotechnical work is required to support the FTAA substantive application for the proposed development. However, the following future work items need to be considered as part of the detailed design and construction:

- Preparation of an earthwork's specification prior to undertaking earthworks.
- DCP testing, laboratory CBR testing and pavement design will be required to support design of roading infrastructure.
- Additional targeted geotechnical investigation and laboratory testing may be undertaken during detailed design to refine the assessment of liquefaction and lateral spreading and optimise the associated foundation and mitigation requirements.
- Detailed design of the northern hill slope retaining wall to support a Building Consent application.
- Specific investigation and detailed design for the proposed wastewater pump station.
- Specific investigation and detailed design for the proposed water reservoir.
- During earthworks site observations, testing, certification and preparation of a Geotechnical Completion Report will be required.
- Geotechnical investigations, analysis and reporting for industrial / commercial buildings suitable to support Building Consent applications.

10.0 RITS V2 UPDATE

In accordance with Waikato Regional Infrastructure Technical Standard (RITS) version 2, the MBIE/NZGS Earthquake Geotechnical Engineering Module 2 is referenced for determining the number of investigation points. However, based on engineering judgement and the specific characteristics of the site, strict application of the Module 2 guidance is not considered appropriate at this stage in the development cycle.

The prescribed investigation density is not considered to yield proportionate benefit for the current level of design. For this specific site, ground conditions are well constrained and understood as can be seen on the Geological Section A-A (**Drawings 03 and 04** presented in **Appendix A**). Soil material parameters have been selected using extensive local knowledge in the geological units present on site with verification of parameters from extensive laboratory testing in the same geologies from nearby projects including the Waikato Expressway Hamilton section.

Additional, targeted investigations will be undertaken during detailed design for all structures / industrial buildings once structural loads and foundation requirements are defined. As this will be required for all structures across this development, this approach aligns with the RITS clause permitting departures from best

practice guidelines where supported by a rigorous and verifiable assessment process and ensures that investigation effort is appropriately focused to achieve a technically robust and efficient outcome.

11.0 SUITABILITY STATEMENT

The site investigations carried out are considered suitable for the assessment of geotechnical constraints and associated requirements in support of a FTAA substantive application.

The post-development qualitative assessment of natural risk hazard (residual risk) for the site is low to medium for all hazards considered.

Based on the findings of the geotechnical investigations, we consider that the site is suitable for the proposed development provided our recommendations are followed. With the outlined mitigations implemented, we consider that ongoing effects from geohazards would be suitably managed.

The further investigation, analysis and design identified in Section 9 are detailed design and construction-stage requirements and do not represent unresolved geotechnical constraints to the feasibility or suitability of the proposed development.

12.0 SAFETY IN DESIGN

The design landform requires site excavations that are likely to include geotechnical works such as undercuts, temporary excavations, retaining walls and fill batters. Exposure to these works forms a significant safety risk for contractors and inspectors / testers.

In conducting our scope of work, we have considered and addressed Safety in Design (SiD) aspects relevant to our understanding of the proposed design and construction work. SiD must consider the construction, operation, maintenance, and ultimate demolition phases of the relevant works.

It is noted that CMW are focussed on design aspects, and whilst we have attempted to be comprehensive in our assessment, it is the Contractors responsibility to cover construction related risks in a more comprehensive manner (being the competent party in that respect).

Our SiD risk assessment is presented in **Appendix L**. This risk assessment must be communicated with all affected parties involved with the project and dealt with through specific on-site risk assessment plans.

13.0 CLOSURE

This report has been prepared for use by Te Kowhai East Limited Partnership in relation to the Te Kowhai East Industrial Development, 270 Te Kowhai Road and 28 Mathers Road, Hamilton project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of this report please do not hesitate to contact us.

Where a party other than Te Kowhai East Limited Partnership seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site. A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

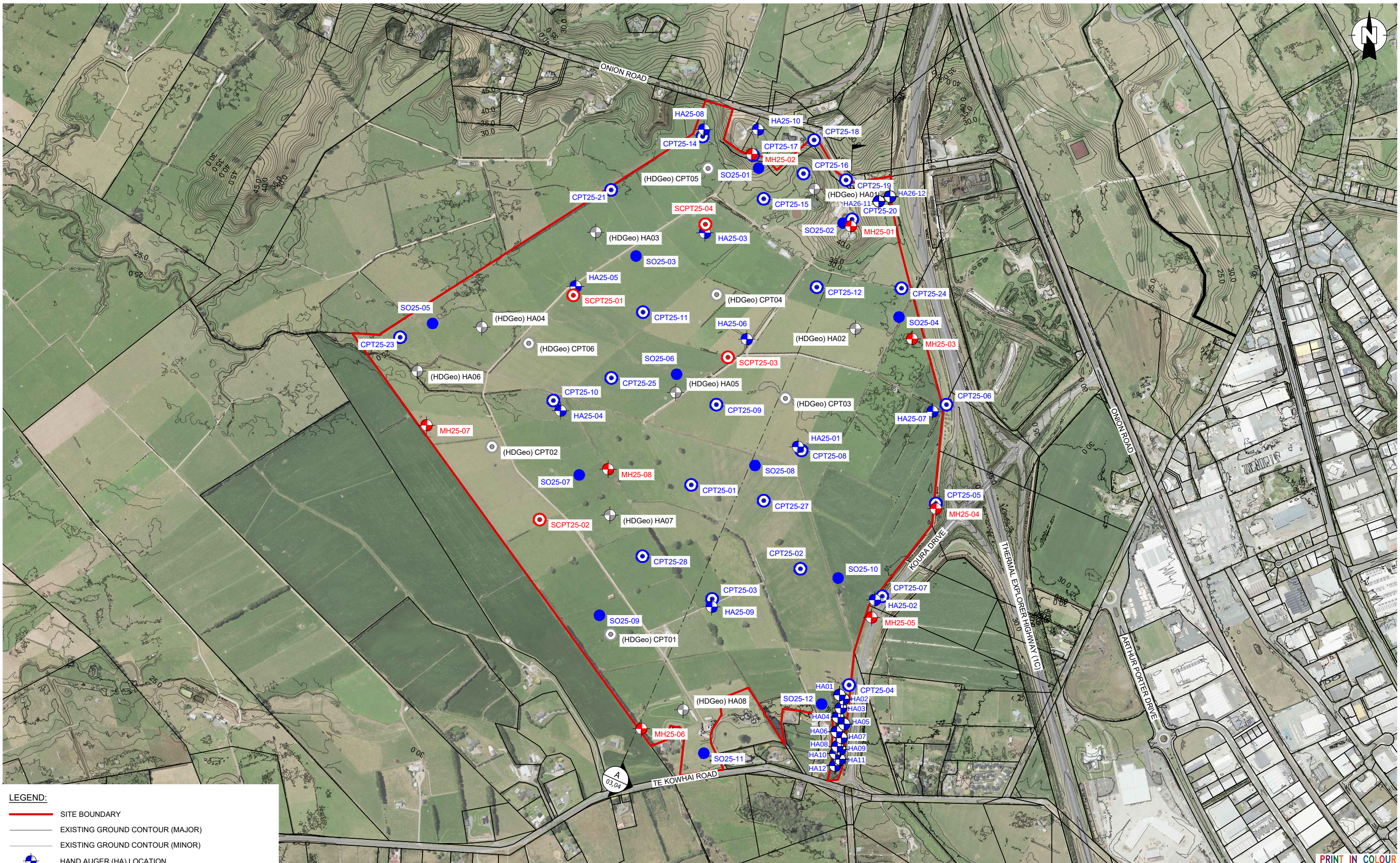
Environmental matters are not covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

APPENDIX A

Drawings



LEGEND:

SITE BOUNDARY

EXISTING GROUND CONTOUR (MAJOR)

EXISTING GROUND CONTOUR (MINOR)

HAND AUGER (HA) LOCATION

CONE PENETROMETER TEST (CPT) LOCATION

SOAKAGE TEST (SO) LOCATION

MACHINE BOREHOLE (BH) LOCATION

SEISMIC CONE PENETROMETER TEST (SCPT) LOCATION

HAND AUGER (HA) LOCATION [HD GEO]

CONE PENETROMETER TEST (SCPT) LOCATION [HD GEO]

- NOTES:
1.

2.

3.

4.
- AERIAL IMAGE COURTESY OF LINZ - HAMILTON 0.05m URBAN AERIAL PHOTOS (2024-2025).

EXISTING GROUND CONTOURS CREATED USING LIDAR DATA OBTAINED FROM LINZ - NEW ZEALAND LIDAR 1m DEM. CONTOURS SHOWN IN 1m INTERVALS.

CRS: NZGD2000 / NZTM. VERTICAL DATUM IN TERMS OF NZVD2016.

TEST LOCATIONS ARE INDICATIVE ONLY.



CMW

Geosciences

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CLIENT:

TE KOWHAI EAST LIMITED PARTNERSHIP

PROJECT:

TE KOWHAI EAST INDUSTRIAL DEVELOPMENT

TITLE:

SITE INVESTIGATION PLAN

DRAWN:

HV

CHECKED:

AS

REVISION:

0

DATE:

24.04.2026

PROJECT:

HAM2025-0116

DRAWING:

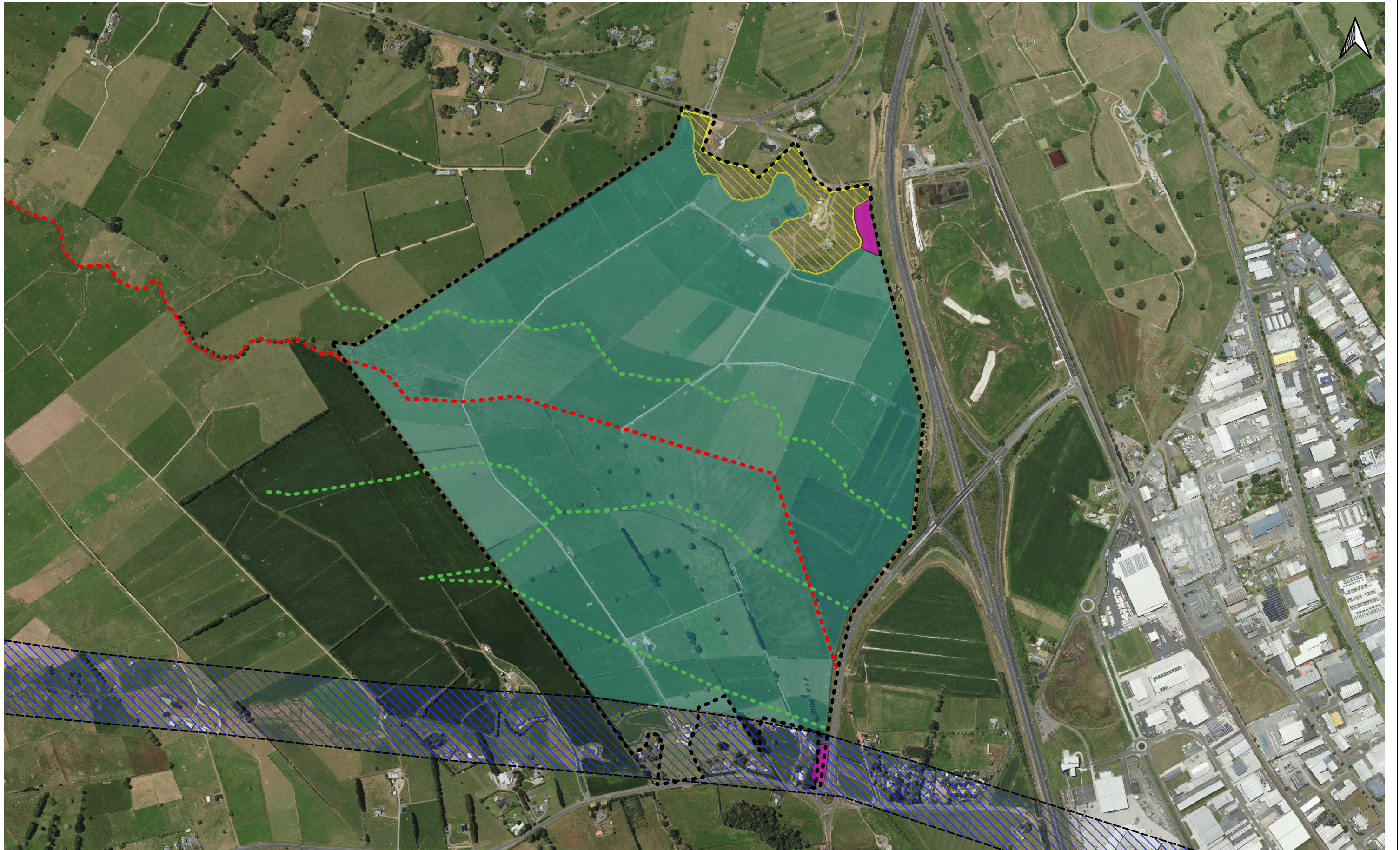
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SCALE:

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SHEET:

A3 L

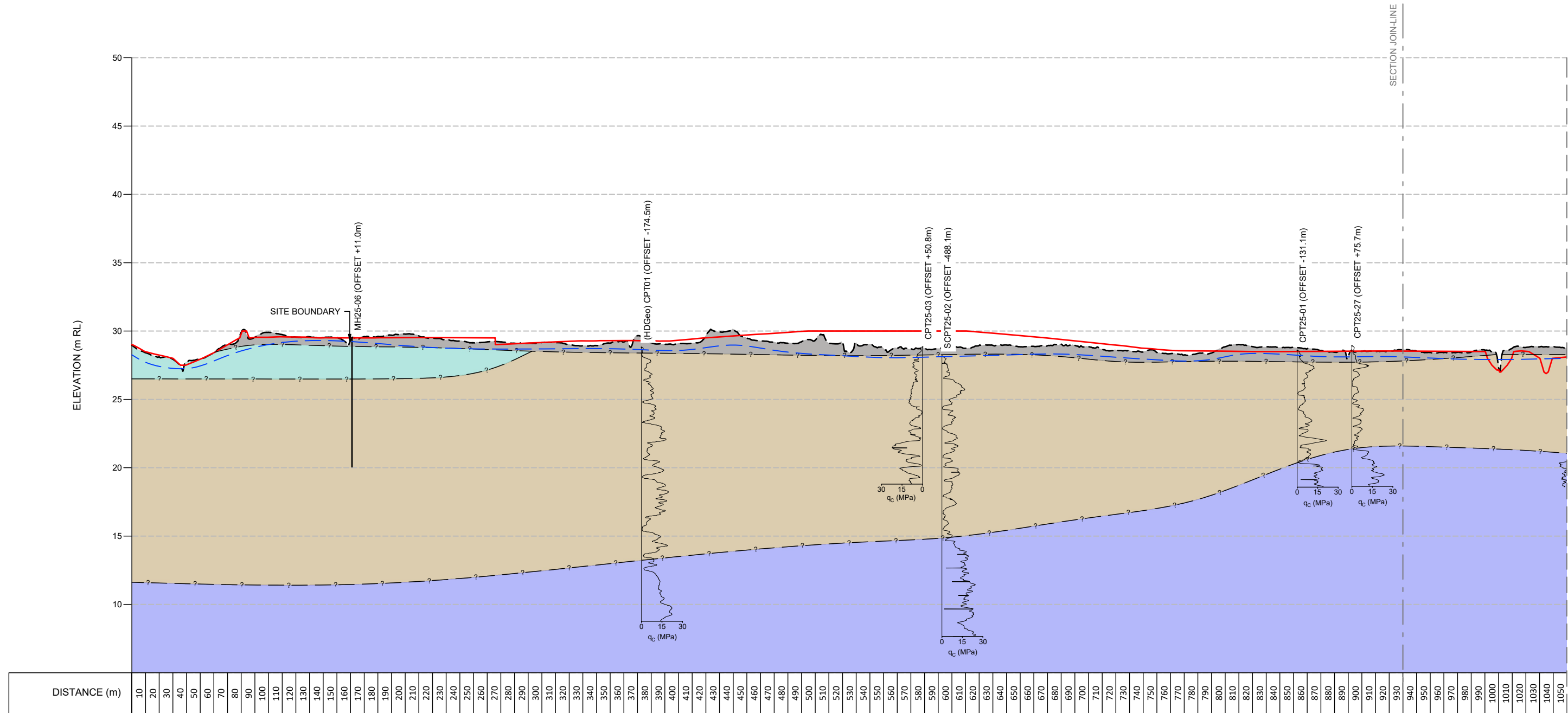


LEGEND:

-  Approximate Historic Te Kowhai Channel
-  Approximate Paleochannel Alignments
-  Approximately Site Boundary
-  Low-lying Plains
-  Low-lying Plains (Piako Subgroup)
-  Mangaheka Stream
-  Rolling Hill Topography



CLIENT:	TE KOWHAI EAST LIMITED PARTNERSHIP		DRAWN:	AS	PROJECT:	HAM2025-0116
PROJECT:	TE KOWHAI EAST INDUSTRIAL		CHECKED:	KL	DRAWING:	02
TITLE:	GEOMORPHOLOGY PLAN		REVISION:	0	SCALE:	1:10,000
			DATE:	28/08/2026	SHEET:	A3L

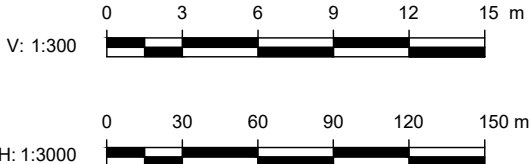


SECTION-A
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1:300 V

LEGEND:

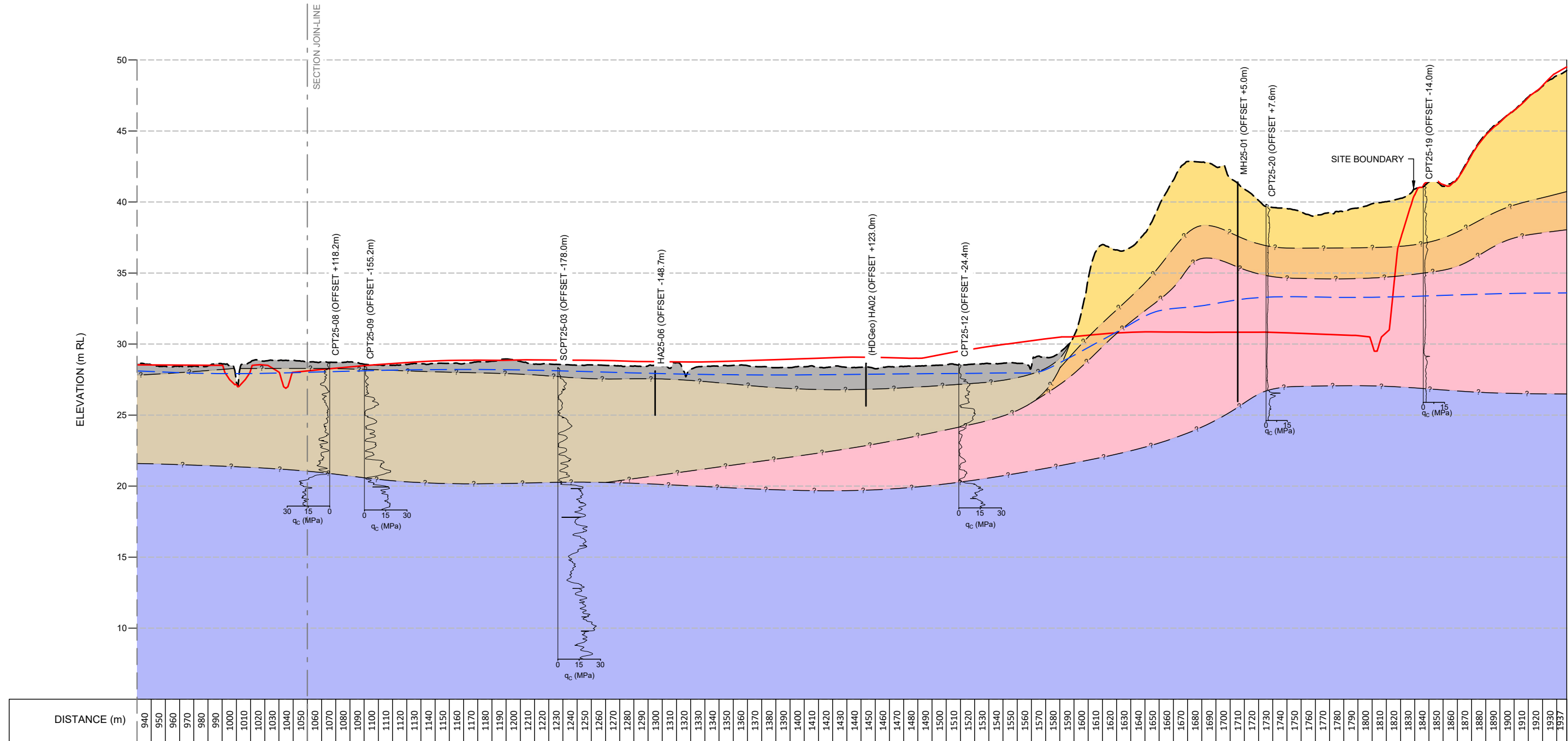
- EXISTING GROUND PROFILE
- DESIGN GROUND PROFILE
- INFERRED GEOLOGY BOUNDARY
- GROUND WATER LEVEL
- FIRM TO STIFF SILT (HINUERA FORMATION)
- LOOSE TO MEDIUM DENSE GRAVEL (HINUERA FORMATION)
- LOOSE TO MEDIUM DENSE SANDS W/ INTERBEDDED SILTS (HINUERA FORMATION)
- STIFF TO VERY STIFF CLAY (HAMILTON ASHES)
- VERY STIFF TO HARD CLAY (KUROA ASHES)
- STIFF CLAY/SILT (PUKETOKA FORMATION)
- DENSE TO VERY DENSE SAND (NON-WELDED IGNI MBRIE - WALTON SUBGROUP)

- NOTES:**
- EXISTING GROUND PROFILE CREATED FROM NZ LIDAR DATA OBTAINED FROM LINZ.
 - DESIGN PROFILE CREATED FROM CONTOURS PROVIDED BY MAVEN ASSOCIATES.
 - VERTICAL DATUM IN TERMS OF NZVD2016.
 - TEST LOCATIONS ARE INDICATIVE ONLY.



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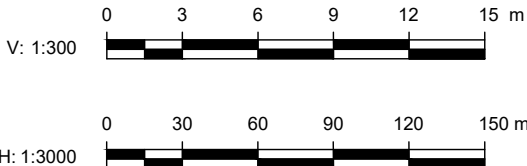
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PROJECT: TE KOWHAI EAST INDUSTRIAL DEVELOPMENT	CHECKED: AS	DRAWING: 03
TITLE: GEOLOGICAL CROSS SECTION-A (SHEET 1 OF 2)	REVISION: 0	SCALE: AS SHOWN
	DATE: 24.04.2026	SHEET: A3 L



SECTION-A
SCALE: 1:3000 H
1:300 V

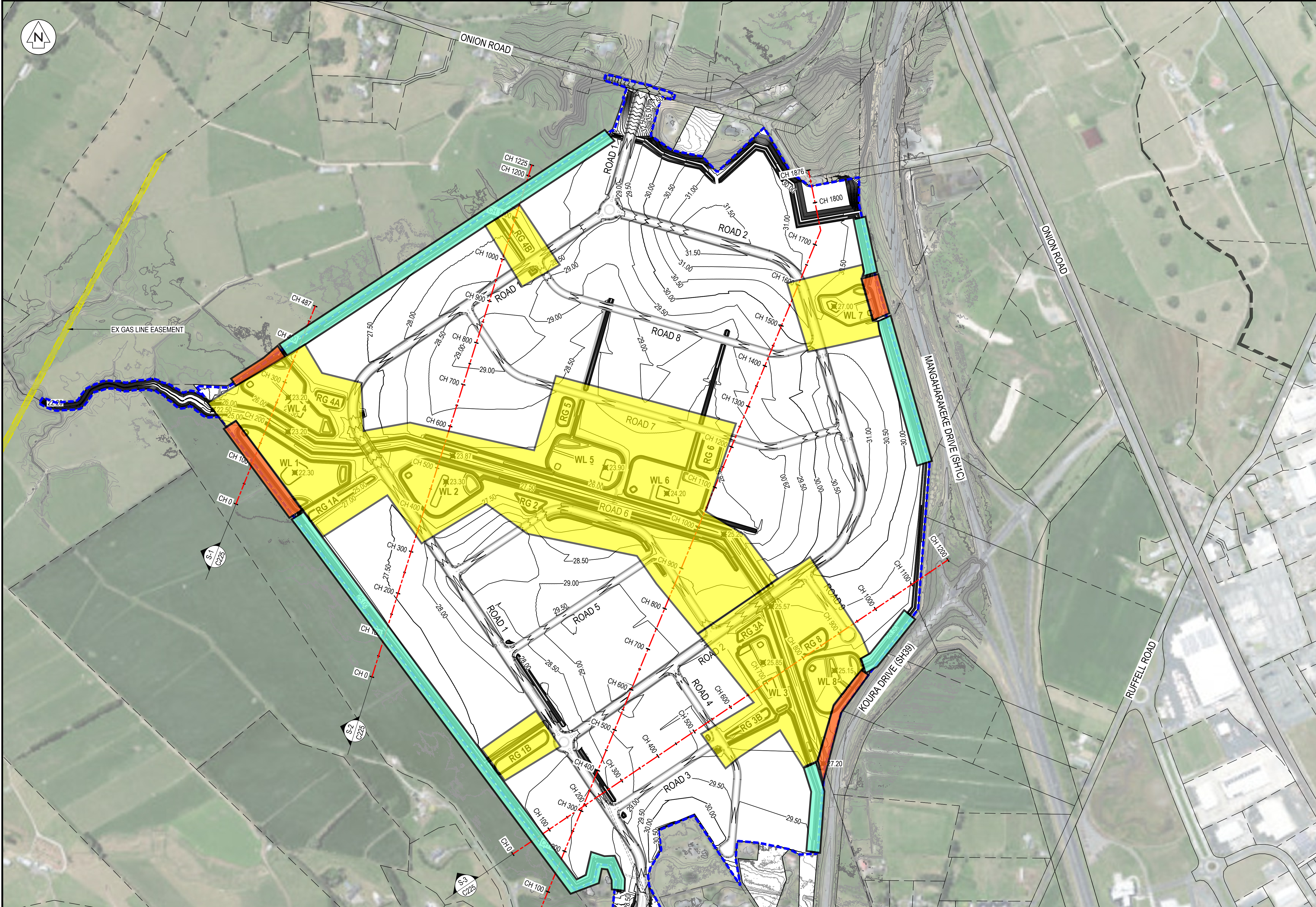
- LEGEND:**
- EXISTING GROUND PROFILE
 - DESIGN GROUND PROFILE
 - INFERRED GEOLOGY BOUNDARY
 - GROUND WATER LEVEL
 - FIRM TO STIFF SILT (HINUERA FORMATION)
 - LOOSE TO MEDIUM DENSE GRAVEL (HINUERA FORMATION)
 - LOOSE TO MEDIUM DENSE SANDS W/ INTERBEDDED SILTS (HINUERA FORMATION)
 - STIFF TO VERY STIFF CLAY (HAMILTON ASHES)
 - VERY STIFF TO HARD CLAY (KUROA ASHES)
 - STIFF CLAY/SILT (PUKETOKA FORMATION)
 - DENSE TO VERY DENSE SAND (NON-WELDED IGNI MBRIE - WALTON SUBGROUP)

- NOTES:**
- EXISTING GROUND PROFILE CREATED FROM NZ LIDAR DATA OBTAINED FROM LINZ.
 - DESIGN PROFILE CREATED FROM CONTOURS PROVIDED BY MAVEN ASSOCIATES.
 - VERTICAL DATUM IN TERMS OF NZVD2016.
 - TEST LOCATIONS ARE INDICATIVE ONLY.



CLIENT: TE KOWHAI EAST LIMITED PARTNERSHIP	DRAWN: HV	PROJECT: HAM2025-0116
PROJECT: TE KOWHAI EAST INDUSTRIAL DEVELOPMENT	CHECKED: AS	DRAWING: 04
TITLE: GEOLOGICAL CROSS SECTION-A (SHEET 2 OF 2)	REVISION: 0	SCALE: AS SHOWN
	DATE: 24.04.2026	SHEET: A3 L

PRINT IN COLOUR



LEGEND:

-  0.5m ENGINEERED FILL LINER IN SWALES (SEE REPORT SECTION 7.4.3)
-  DEEP SOIL MIXING (DSM) PILES OR SIMILAR (SEE REPORT SECTION 7.4.2)
-  APPROXIMATE AREA WITHIN SITE BOUNDARIES THAT WILL REQUIRE REMEDIATION (SEE REPORT SECTION 7.4.4)

NOTES:

1. BASE PLAN ADAPTED FROM MAVEN PROPOSED FINISHED CONTOUR PLAN
2. EXTENTS ARE INDICATIVE ONLY AND WILL REQUIRE DETAILED DESIGN



CLIENT:	TE KOWHAI EAST LIMITED PARTNERSHIP	DRAWN:	AS	PROJECT:	HAM2025-0116
PROJECT:	TE KOWHAI EAST INDUSTRIAL DEVELOPMENT	CHECKED:	KL	DRAWING:	06
TITLE:	SWALE / WETLAND REMEDIATION PLAN	REVISION:	0	SCALE:	NTS
		DATE:	14/08/2026	SHEET:	A3