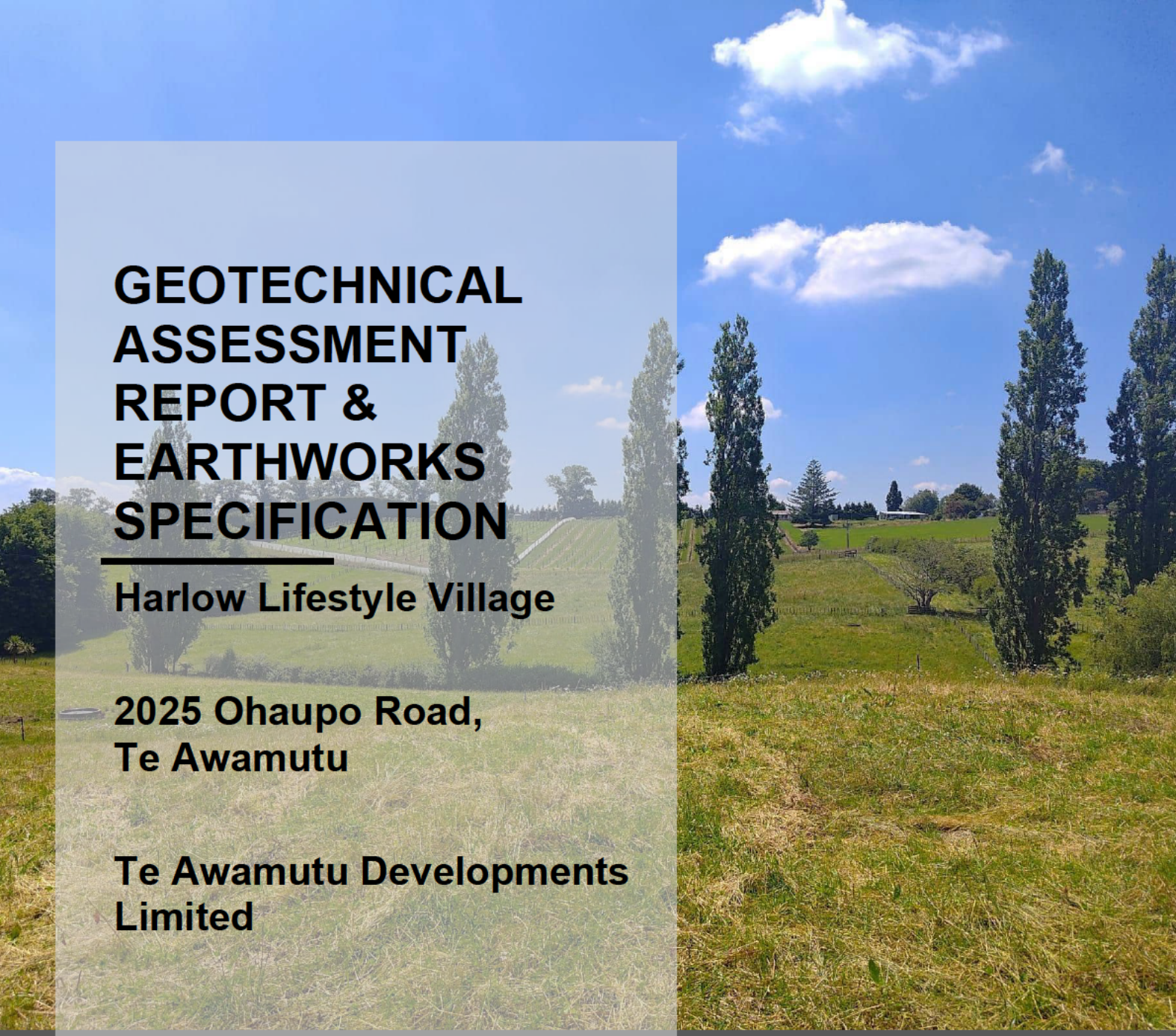
Appendix 11 – Geotechnical Assessment and Earthworks Report



GEOTECHNICAL ASSESSMENT REPORT & EARTHWORKS SPECIFICATION

Harlow Lifestyle Village

2025 Ohaupo Road, Te Awamutu

Te Awamutu Developments Limited

27 OCTOBER 2026

PROJECT NO. C3102

TITUS

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RECORD OF REVISION CHANGES

Revision	Details
A	Draft for review.
B	First issue. Updated based on latest development plans, BBO review, and feedback from Waipa District Council. Changes include: • Section 1.3 – proposed development description updated • Section 1.4 – WGA and Maven report references included • Section 4.3 – As per WGA findings • Section 4.4 – New ground models prepared based on the latest design surface • Section 5.4 – Flood assessment updated as per Maven stormwater reporting • Section 5.7 – Minor update based on latest development plans • Section 5.9.3 – Updated based on new ground models • Section 5.10 – Updated based on new ground models • Section 7.7 – Updated based on new ground models • Section 8 – Updated in accordance with changes to above sections. • Section 9 – Added.

Further Work Required:

- Retaining wall designs
- Plan review on completion of development variations
- Drainage Blanket Civil Drawings
- Construction monitoring as per Section 7.11

Further Information Required:

- Development plans if any major changes are made
- As-built information as per Section 7.12

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STATEMENT OF EXPERIENCE

Author

I am Liam Vick, an Engineering Geologist with Titus Civil Limited. I hold a Bachelor of Science and have 4.5 years of professional experience.

I confirm that I have read and understood the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023, and I agree to comply with it.

I acknowledge that, as an expert, I have an overriding duty to assist the decision maker impartially on matters within my area of expertise. In preparing this report, I have exercised my independent professional judgement, and the opinions I express are my own. I understand that my duty as an expert is not to advocate for the interests of the applicant or any other party.

I confirm that the matters addressed in this report are within my area of expertise, except where I expressly state that I am relying on information or opinions provided by another person. I have not omitted to consider any material facts known to me that might alter or detract from the opinions expressed in this report.

Reviewer and Signee

I am Anthony Richardson, a Geotechnical and Civil Engineer with Titus Civil Limited. I hold a BSc Eng (Civil), MSc (Eng) and Graduate Certificate (Earth Sciences), and have 25 years of professional experience.

I confirm that I have read and understood the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023, and I agree to comply with it.

I acknowledge that, as an expert, I have an overriding duty to assist the decision maker impartially on matters within my area of expertise. In preparing this report, I have exercised my independent professional judgement, and the opinions I express are my own. I understand that my duty as an expert is not to advocate for the interests of the applicant or any other party.

I confirm that the matters addressed in this report are within my area of expertise, except where I expressly state that I am relying on information or opinions provided by another person. I have not omitted to consider any material facts known to me that might alter or detract from the opinions expressed in this report.

1 INTRODUCTION

1.1 General

Titus Consulting Engineers (Titus) has been engaged by Te Awamutu Developments Limited to prepare a Geotechnical Assessment Report & Earthworks Specification for the proposed development at 2025 Ohaupo Road, Te Awamutu.

The purpose of this report is to describe the investigations completed, the ground conditions encountered, and to provide geotechnical recommendations with respect to the proposed development. It is understood this report will be used to support the Fast Track Application to the Fast Track Panel.

The report is subject to revision based on any unsighted planning requirements.

1.2 Scope

This report aims to build on the existing geotechnical reporting in light of the latest development plans and additional ground investigations completed. It assesses the site geohazards with respect to the NPS-NH and RMA (Section 5) including detailed assessments of the following:

- Liquefaction based on additional ground investigations completed across the site.
- Settlement of soils following bulk earthworks and building construction.
- The global stability of proposed retaining walls.

The report will provide a suitability assessment of site-won soils for engineered fill (Section 6), provide specific earthworks requirements in an Earthworks Specification (Section 7), and provide geotechnical recommendations based on the report findings (Section 8).

1.3 Proposed Development

The provided development plans attached in **Appendix A** show a new lifestyle village proposed for the site. The site is legally described as *PT LOT 1 DP 35654 LOT 1 DPS 36696 BLK II PUNIU SD and* comprises an area of approximately 25.78 ha.

The Project is to enable the development of a community for senior living in Te Awamutu, providing approximately 414 dwellings/units (including single-storey and two-storey houses and duplex units, and three-storey apartments), and a 100-bed care facility. The development will also feature communal facilities and a neighbourhood centre, riparian planting, wetland restoration, three waters services infrastructure, and transport infrastructure. The project is designed to cater specifically to an ageing population, addressing a critical need for retirement housing and care capacity.

The development will proceed in alpha-numeric order as set out on the **attached plan**, notwithstanding that some downstream infrastructure that lies within latter stages will need to be constructed at the time that the earlier stages require it.

General construction timing is expected to be as follows:

- Stages 1A and 1B are programmed to commence earthworks and civils in Q1–Q2 2027.

- Subsequent stages are programmed at approximately nine-month intervals between stage commencements.

The development will involve significant earthworks and retaining walls to form level building platforms, roads, stormwater channels, and five stormwater ponds labelled A through E. The provided cut to fill plan indicates cuts of up to 6.5m and fills of up to 7m in places. Cuts are proposed in the far east, along the southern boundary, in the centre north, and in the far west. The head of the southern gully / existing pond (described in Section 2.2) will be filled. Fill is also proposed where retaining walls are to be located. The plans indicate a cut volume of 271,320m^3 and a fill volume of 143,941m^3.

The retaining is proposed to comprise timber pole walls around building platforms, ponds, and stormwater channels, as well as internal block walls where houses are tiered on the slopes. The retaining walls are proposed to heights of up to approximately 5m with the locations shown in **Appendix A** and **B**. The existing gully will be utilised as the stormwater drainage pathway for the development and will include the use of retaining walls. The proposed ponds will be constructed across a combination of cut and fill.

1.4 Previous Reports

BTW Company Limited (BTW) previously prepared a Geotechnical Suitability Report[1] for the site dated 11 November 2022. The report includes the following ground investigations relevant to, and used in preparation of this report with test locations and results attached in **Appendix B** and **C** respectively:

- 29 No. Hand auger boreholes (BH) to a target depth of 2.0m below ground level (bgl).
- 10 No. Cone penetration test (CPT) to a target depth of 20m bgl.

The report includes recommendations on the site suitability, liquefaction, foundation design, groundwater, pavement design, earthworks, and soakage. As mentioned in Section 1.2 above these recommendations and assessments will be used as part of this report, and added to where additional information has become available.

WGA have prepared a Groundwater Effects Substantive Report[2] for the site dated August 2026. The findings are discussed in Section 4.3.

Maven Waikato Limited (Maven) have prepared a Stormwater Management Plan[3] for the site dated 25 September 2026. The findings are discussed in Section 5.4.

1.5 Planning Requirements

It is noted on the supplied plans that the part of the low-lying land in the west of the site is designated as a significant natural area (SNA) comprising kahikatea forest and a wetland. This is shown in Figure 1.

[1] BTW Company, Geotechnical Suitability Report, Proposed Private Plan Change at 2025 Ohaupo Road, Te Awamutu, for Te Awamutu Developments Limited, Rev A – 24/11/2022, ref: 211365
[2] WGA, Harlow Lifestyle Village – Groundwater Effects Substantive Report, Te Awamutu Developments Limited, dated August 2026, ref: WGA260929-RP-HG-0010_A
[3] Maven Waikato Limited, Harlow Lifestyle Village, Stormwater Management Plan, dated 25/09/2026, ref: 411001

2 SITE INVESTIGATION

2.1 Description

The site is located on the northern outskirts of Te Awamutu on State Highway (SH) 3 and is currently pasture covered farmland. A dwelling occupies the site in the east, adjacent to SH3. The site is predominantly bound by farmland, with SH3 to the east and residential properties to the southeast. Access to the site is currently provided via the existing dwelling. An aerial image of the site is shown below in Figure 1 including annotations of the significant site features.

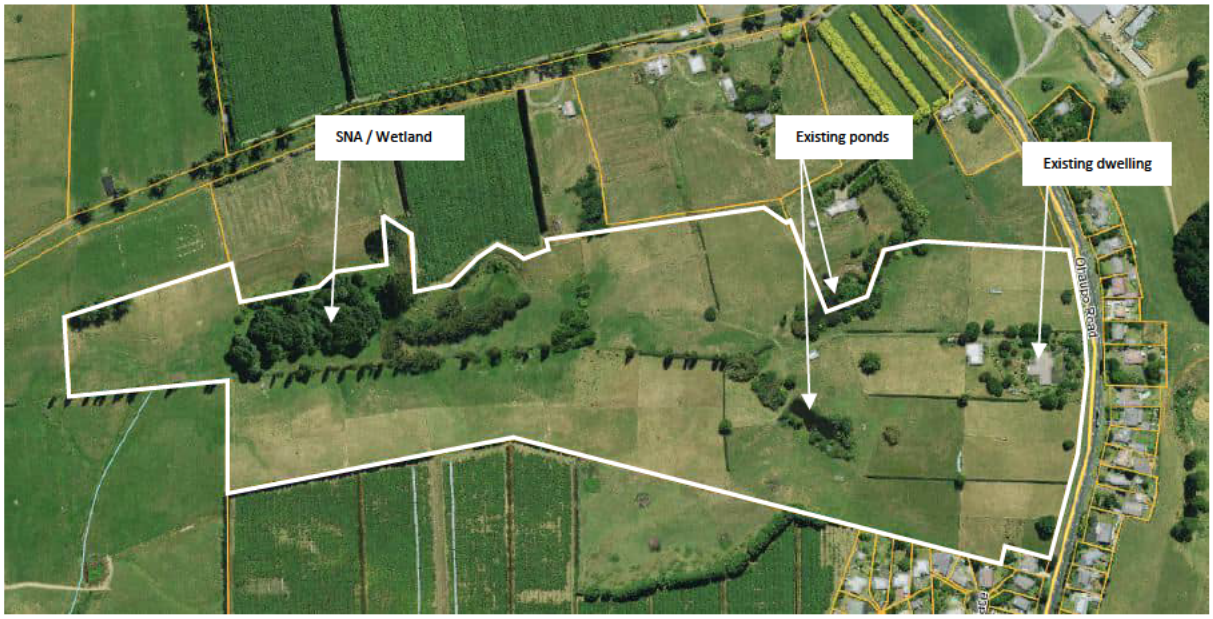


Figure 1: Site location (Waipa District Council GIS maps)

2.2 Landform

The site landform can be classified by three distinct geomorphologies. The rolling low hills (Hamilton Hills) and the flattish alluvial plains[4] comprise the two main landforms. The hills are incised by a gully which runs east to west through the centre of the property and forms the third landform, the gully landform. The alluvial plains and gully landforms have subsequently been considered together for the purposes of this report. The site landforms are presented in Figure 2 below.

At the head of the gully are two existing ponds as shown in Figure 1. Recharging of the ponds is discussed in Section 4.3. Wetlands and a small stream are formed downstream of the ponds eventually dissipating to the alluvial plains. No springs were observed on the site during the site assessment.

[4] Lowe, D. J. (2010). *Introduction to the landscapes and soils of the Hamilton Basin.* In D. J. Lowe, V. E. Neall, M. Hedley, B. Clothier, & A. Mackay (Eds.), *Guidebook for Pre-conference North Island, New Zealand "Volcanoes to Oceans" field tour* (pp. 1.24-1.61). Soil and Earth Sciences Occasional Publication No. 3, Massey University.

Slope gradients within the low hills are gently rolling at average gradients of 1V:10H (5-6 degrees) and steepen to 1V:5H (11 degrees) where the gully has eroded the hills. The base of the gully falls at approximately 1V:50H. The gully slopes steepen in the west of the site on the north gully face to gradients ranging from between 1V:1H (45 degrees) to 1V:3H (18 degrees), henceforth referred to as 'steep slopes'.

The elevation of the low hills ranges from RL 84m in the east to RL 50m in the west near the gully base. The elevation of the alluvial plans / gully base ranges from RL 58m in the east to RL 47m in the west.

Generally, no significant erosional surface features were observed within the development area with exception to the aforementioned north gully face (steep slopes) where soil creep was observed. It is noted that no development is proposed in this area.

A plan of the identified landforms within the site boundary is attached in **Appendix B** with site geotechnical investigations overlaid.



Figure 2: Plan showing alluvial plains and gully landforms in white and the low hills landform in orange

2.3 Historical Imagery

An investigation of historical imagery relating to the site has been conducted using the Retrolens[5] database and Google Earth Pro. The investigation indicates the following key observations:

- The first available image is from 1943 and shows the site covered in pasture. The existing ponds are not present, instead the two heads of the gully appear unincumbered. The base of the gully appears covered in wetland vegetation and the SNA kahikatea forest is juvenile.
- The image from 1958 shows that farm tracks have been formed across the heads of the gully where the present-day tracks are located. Culverts would have been installed as the ponds were not formed for two more decades.

[5] Retrolens website sourced from https://retrolens.co.nz

- In 1963 soil creep is observed along some of the gully slopes and at the steep hills in the west.
- Between 1971 and 1974 the trees on the steep slopes were felled.
- Between 1974 and 1976 the southern gully head culvert blocked which led to the formation of the first pond. By 1979 the northern pond has also formed.
- Between 1979 and 1986 the existing dwelling was constructed. Trees were replanted on the steep slopes. These were partially removed again over time until present-day.

Relevant historical images of the site described above are attached in **Appendix D**.

2.4 New Zealand Geotechnical Database

There are no relevant entries to the site within the New Zealand Geotechnical Database[6] (NZGD).

[6] NZGD website sourced from https://www.nzgd.org.nz/

3 FIELD INVESTIGATIONS

3.1 Scope

The site assessment was carried out on 05 and 06 May 2026 under the direction of Titus and in accordance with NZGS guidelines and relevant standards[7,8,9,10]. The scope of the fieldwork completed was as follows:

- Site walkover to assess the general landform and site conditions.
- 13 No. Hand auger (HA) boreholes to target depths of 4m bgl. All tests achieved the target depth except HA03, 07, 08, 09, 10, and 13 which were refused due to non-retrieval of spoil.
- Dynamic cone (scala) penetrometer (DCP) and shear vane tests were conducted (for the appropriate soil type) in conjunction with the hand augers to determine the relative strength of the soils encountered.
- 5 No. Test pits (TP) to target depths of 6m bgl. Samples were collected from the test pits at various depths for laboratory testing. The test pit target depth was achieved.
- 6 No. Cone penetration test (CPT) to target depths of 20m bgl. All tests achieved the target depth without early termination.

Test locations are shown on the geotechnical investigation plan attached in **Appendix B** and ground investigation logs are presented in **Appendix C**.

3.2 Findings

3.2.1 Ground Conditions

No buried topsoil or fill was encountered on the site during our site investigation, however it is noted that BTW encountered uncontrolled fill to 0.4m bgl in BH09. Topsoil thickness ranged from between 0.1m to 0.3m.

The hand augers conducted on the low hills predominantly show layers of stiff to very stiff brown silt and clay. Tests conducted at lower elevation within or near the gully and alluvial plains showed layers of grey, stiff to very stiff clay and medium dense sands.

The test pits were completed on the low hills and showed layers of brown silt and clay, generally underlain by a thin layer of grey micaceous clay then variable soils comprising clay, and pumiceous clayey sands with gravel.

CPT01-04, and 06 were completed within the low hills landform. The interpreted soil behaviour type (SBT) plots of these tests generally show layers of clay to the test base where a dense to very dense sand (~20MPa) layer is encountered until test termination. In one instance - CPT03 - the sand layer is not found. CPT05 was completed within the alluvial plains / gully landform and showed 5m of clay then sand for the remainder of the test.

[7] New Zealand Geotechnical Society. (2017). NZ Ground Investigation Specification, Volume 1 – Master Specification
[8] NZS 4402:1986 – Methods of Testing Soils for Civil Engineering Purposes – New Zealand Standards, Wellington, New Zealand.
[9] New Zealand Geotechnical Society. (2005). Field description of soil and rock – Guideline for the field classification and description of soil and rock for engineering purposes
[10] New Zealand Geotechnical Society. (2001, August). *Guideline for hand held shear vane test.* NZGS.

A review of the BTW hand augers and CPTs has been carried out with the findings generally being aligned with the above results. It is noted that BTW have anomalously defined some soils as Walton Subgroup which accounts for any discrepancy in unit identification. Simplified geotechnical soil unit descriptions, depths/thicknesses, and strengths are defined in Section 4.

Ground investigations did not encounter significant areas of uncontrolled fill or old fill areas associated with historical farming activities. It is noted however that typically properties of this scale include areas of uncontrolled filling and old rubbish pits which can sometimes be extensive. In all instances where such ground conditions are encountered within proposed development areas these materials shall be undercut to waste and filled as required to achieve the earthworks specifications.

3.2.2 Groundwater

Groundwater was found at the following depths summarised by landform. Testing was carried out during late-autumn. Discussion of the groundwater table across the site is provided in Section 4.3. BTW hand auger and CPT groundwater depths generally align with the below findings.

Perched groundwater was observed above the northern existing pond (pond location shown in Figure 1). No perched groundwater was observed at the southern existing pond. As noted in the table below, artesian flow was recorded within CPT07 (commissioned by BTW). Because no groundwater was recorded in the adjacent BH13 to 2.0m bgl, it is likely that the CPT penetrated an aquifer.

Table 1: Depth to encountered groundwater summary

Low Hills Landform			Alluvial Plains / Gully Landform		
Test ID	**GWT (mbgl)**	**GWT (mRL)***	**Test ID**	**GWT (mbgl)**	**GWT (mRL)**
HA01	Not found	> 80	HA07	0.6	**56.4**
HA02		>74	HA08	0.6	**53.6**
HA03**	2.9	65.1	HA09	1.4	**48.6**
HA04	Not found	>64	HA10	1.5	**47.5**
HA05		>58	HA13	1.2	**45.8**
HA06		>70	CPT05	1.1	**47.9**
HA12		>48	CPT07 BTW	0 Artesian flow	**54.0**
TP01		>76	CPT10 BTW	0.97	**46.0**
TP02		>68			
TP03		>60			
TP04	4.2	**48.8**			
TP05	Not found	>55			

Low Hills Landform			Alluvial Plains / Gully Landform		
Test ID	**GWT (mbgl)**	**GWT (mRL)***	**Test ID**	**GWT (mbgl)**	**GWT (mRL)**
CPT01	Hole collapsed dry 6.3m	>66.7			
CPT02	Hole collapsed dry 1.3m	>59.7			
CPT03	Hole collapsed dry 5.3m	>60.7			
CPT04	Hole collapsed dry 2.5m	>56.5			
CPT06	Hole collapsed dry 2.7m	>55.3			
*symbol > indicates that the GWT is deeper than the depth shown. **HA03 GWT likely perched and discharging to the northern existing pond.					

3.3 Laboratory Testing

All testing was scheduled by Titus and carried out by WSP Laboratory (IANZ registered Testing Authority). Samples of the various soil units encountered across the site were collected from the test pits. The extent of lab testing carried out was as follows:

- 5 No. soil samples were sent to WSP Laboratory for Compaction Curve NZ Standard and Solid Density. All laboratory testing is completed in accordance with NZS 4402.

Lab testing results are attached in **Appendix G** and discussed in Section 6.2.

4 GROUND MODEL

4.1 Geological Setting

Published 1:250,000 geological maps[11] indicate that the site is underlain by the Hinuera Formation and the Walton Subgroup. The Hinuera Formation is broadly mapped as the low-lying alluvial plains and gully landforms, and Walton Subgroup as the low hills. It should be noted that the GNS mapping has low accuracy at local scales.

Walton Subgroup (WSG) comprises Early Pleistocene to Middle Pleistocene river deposits and ignimbrite described as *'Alluvium dominated by primary and reworked, non-welded ignimbrite'* and was deposited between 2.588-0.423 Ma. The alluvial deposits constitute of the Karapiro Formation and ignimbrite of the Puketoka Formation. WSG underlies the low hills landform and is typically overlain by volcanic ashes including Kauroa Ash, Hamilton Ash Formation (HAF), and more recent undifferentiated tephras.

The Hinuera Formation comprises Late Pleistocene river deposits described as '*Cross-bedded pumice sand, silt and gravel with interbedded* peat' and was deposited between 12-27 ka. Hinuera Formation underlies the alluvial plains landform.

Holocene river deposits typically occur within modern drainage systems (gullies) and are likely to be deposited within the base of the site gully and its depositional alluvial fan.

The landform plan in **Appendix B** represents a local-scale geological map of the site.

4.2 Stratigraphic Units

Based on the findings of the field investigation the soils encountered on the low hills are interpreted as HAF underlain by the Karapiro Formation and Puketoka Formation of the WSG with the Rangitawa Tephra separating the HAF and WSG. For simplicity in design and analysis, the Rangitawa Tephra and undifferentiated tephras have been included in the HAF.

The soils encountered in the alluvial plains and within the base of the gully are interpreted as comprising alluvially reworked deposits of the HAF and WSG likely onlapping and interfingering Hinuera Formation sediments, and overlying WSG. The thickness of these deposits is likely to vary across the site being relatively thin in the east and thickening westward.

Table 2: Ground model unit sequence and descriptions

Defined Unit	Depth to Top - Min. to Max (m)	Thickness - Min. to Max (m)*	Consistency / Relative Density
Alluvial deposits	Below topsoil	2.2 – 5.0**	Very loose to medium dense / stiff to very stiff***
Topsoil	Ground surface	0.1 – 0.3	Organic
Hamilton Ash Formation	Below topsoil	2.0 – 6.1	Stiff to very stiff

[11] Edbrooke, S. E. (Compiler). (2005). *Geology of the Waikato area* (1:250 000 geological map 4). Institute of Geological & Nuclear Sciences.

Defined Unit	Depth to Top - Min. to Max (m)	Thickness - Min. to Max (m)*	Consistency / Relative Density
Karapiro Formation**	2.0 – 6.1	2.0 – 13.1	Very stiff
Puketoka Formation**	5.6 – 18.0	-	Dense to very dense

* Only recorded where base of strata has been identified.

** Interpreted from CPT results.

*** Calibrated against BTW ground investigations.

4.3 Groundwater Table

The groundwater table (GWT) across the site is in accordance with the WGA report referenced in Section 1.4. Figure 6 of that report plots piezometric surface contours indicating a GWT that shallows from RL 66m (15m bgl) in the east at high elevation to RL 48m (1.5m bgl) in the west at low elevation and towards the gully parallel with the ground surface.

WGA state that groundwater recharges on the upper slopes, then discharges to the wetlands, drains, man-made ponds and the Mangapiko Stream tributary running through the base of the valley. At the eastern edge of the site the groundwater is likely to flow eastward to the Mangapiko Stream to the east of Ohaupo Road.

The GWT has been modelled in general accordance with the WGA and BTW findings and the depths recorded in Table 1.

4.4 Site Models

Five cross-section profiles have been formed across the site through areas of geotechnical significance each assessing one of the proposed ponds. The profiles are based on the finished design contours provided by Maven. LINZ contour data has been overlaid to illustrate the proposed change in topography. Cross-section locations are shown in **Appendix B**.

Based on the above cross-section profiles, geology, stratigraphic units, and groundwater tables, ground models have been created for the site. The ground models are presented in **Appendix E** with each encompassing the following:

- **Cross Section A (CS-A):**
  Stage 4 building platforms, retaining walls, Stormwater Pond E
- **Cross Section B (CS-B):**
  Stage 2C building platforms, retaining walls, Stormwater Pond D
- **Cross Section C (CS-C):**
  Stage 2C building platforms, retaining walls, Stormwater Pond C
- **Cross Section D (CS-D):**
  Stage 2B building platforms, retaining walls, Stormwater Pond B
- **Cross Section E (CS-E):**
  Stage 1C building platforms, retaining walls, Stormwater Pond A

5 GEOHAZARDS ASSESSMENT

5.1 Context

The tables presented in **Appendix K** summarises the preliminary assessment of the site for Natural Hazards per Section 106 of the Resource Management Act 1991 (RMA) and the National Policy Statement for Natural Hazards (NPS-NH) 2025. This includes: flooding, landslips, coastal erosion, coastal inundation, active faults, liquefaction, and tsunamis.

In some cases, the hazards may be separated into multiple hazards (liquefaction), and additional hazards have also been included. The assessment evaluates baseline geohazards in accordance with Section 106 of the RMA and the NPS-NH, and establishes engineered mitigation measures. With the implementation of the specified earthworks, drainage, pre-loading, and engineering designs, the residual geotechnical risk and potential for damage from the identified natural hazards are managed and mitigated across the development.

5.2 Active Faults

The NZ Active Faults database[12] indicates that the nearest active fault is the Kerepehi Fault approximately 50km to the east of the site. This is described as an unknown fault with a recurrence interval between 2,000 and up to and including 3,500 years. As such the probability of surface deformations caused by fault rupture to the site is considered very rare.

5.3 Seismicity and Subsoil Category

Design earthquake magnitude and ground acceleration has been determined in accordance with MBIE Guidelines. The following design parameters have been adopted for the Te Awamutu area:

- Class D – Deep soil site based on NZS 1170.5:2004[13]
- Importance Level (IL) 2 and 3 based on Table 3.2 of AS/NZS 1170.0:2002[14]
- 50-year Building Design Life

Table 3: Seismic parameters (NZGS/MBIE, 2021)

Limit State	AEP	PGA (g)	Magnitude M_{eff}
SLS	1/25	0.06	5.9
ILS	1/100	0.12	
ULS - IL2	1/500	0.25	
ULS - IL3	1/1000	0.32	
Note: SLS = serviceability limit state; ILS = intermediate limit state; ULS = ultimate limit state; AEP = annual exceedance probability			

[12] GNS active faults database sourced from https://data.gns.cri.nz/af/

[13] Standards New Zealand. (2004). Structural design actions - Part 5: Earthquake actions - New Zealand (NZS 1170.5:2004).

[14] Standards Australia, & Standards New Zealand. (2002). Structural design actions - Part 0: General principles (AS/NZS 1170.0:2002).

5.4 Flooding

As mentioned in Section 1.4 Maven have carried out 100yr ARI flood modelling based on the proposed development surface elevations. Section 7.3.3 of that report states that "Floor level requirements in relation to floodplains will be set through rules in the future District Plan.", and Section 10 states that "Further refinement of stormwater infrastructure, hydraulic modelling, wetland design, and flood mitigation measures will be undertaken during detailed design and engineering approval stages."

5.5 Volcanic Hazard

According to the available information provided on Geonet[15], the nearest active volcanos are Rotorua and Mayor Island located approximately 85km and 115km from the site respectively. These are currently at 'Alert Level 0 – No volcanic unrest'. As such it is considered that no volcanic hazard is posed to the site at this time. Alert Levels will need to be monitored and action taken as appropriate to manage any risks.

5.6 Coastal Hazards

The site is situated inland such that land damage due to coastal processes is not expected.

5.7 Static Settlement

Load-induced settlements occur in soils surcharged by fill and/or buildings. The degree of settlement is a function of the soil stiffness and the load applied.

The settlement analysis was carried using CPet-IT to estimate 1-D consolidation vertical settlements based on CPT calculated constrained moduli. The analysis considers settlement for periods over 6 months (primary) and 50 years (secondary / creep). Major differential settlement may arise where buildings span landforms and/or areas of cut and fill. Where these criteria are met, recommendations have been provided in Section 8.4. Additionally, cross-boundary settlement has not been considered as the neighbouring property comprises farmland.

The alluvial plains / gully landform soils are considered to be susceptible to static settlement. The placement of fill and/or construction of buildings on this landform has therefore been assessed for static settlement.

The low hills landform soils have low compressibility and are generally not susceptible to static settlement following the placement of fill or construction of normal residential buildings. The multi-storey apartment buildings proposed as discussed in Section 1.2 constitute high loads and have therefore been assessed for static settlement.

Based on the above, areas of high risk have been identified across the site. These are highlighted and numbered on the figure below then summarised in the following table:

[15] Geonet website sourced from https://www.geonet.org.nz/volcano

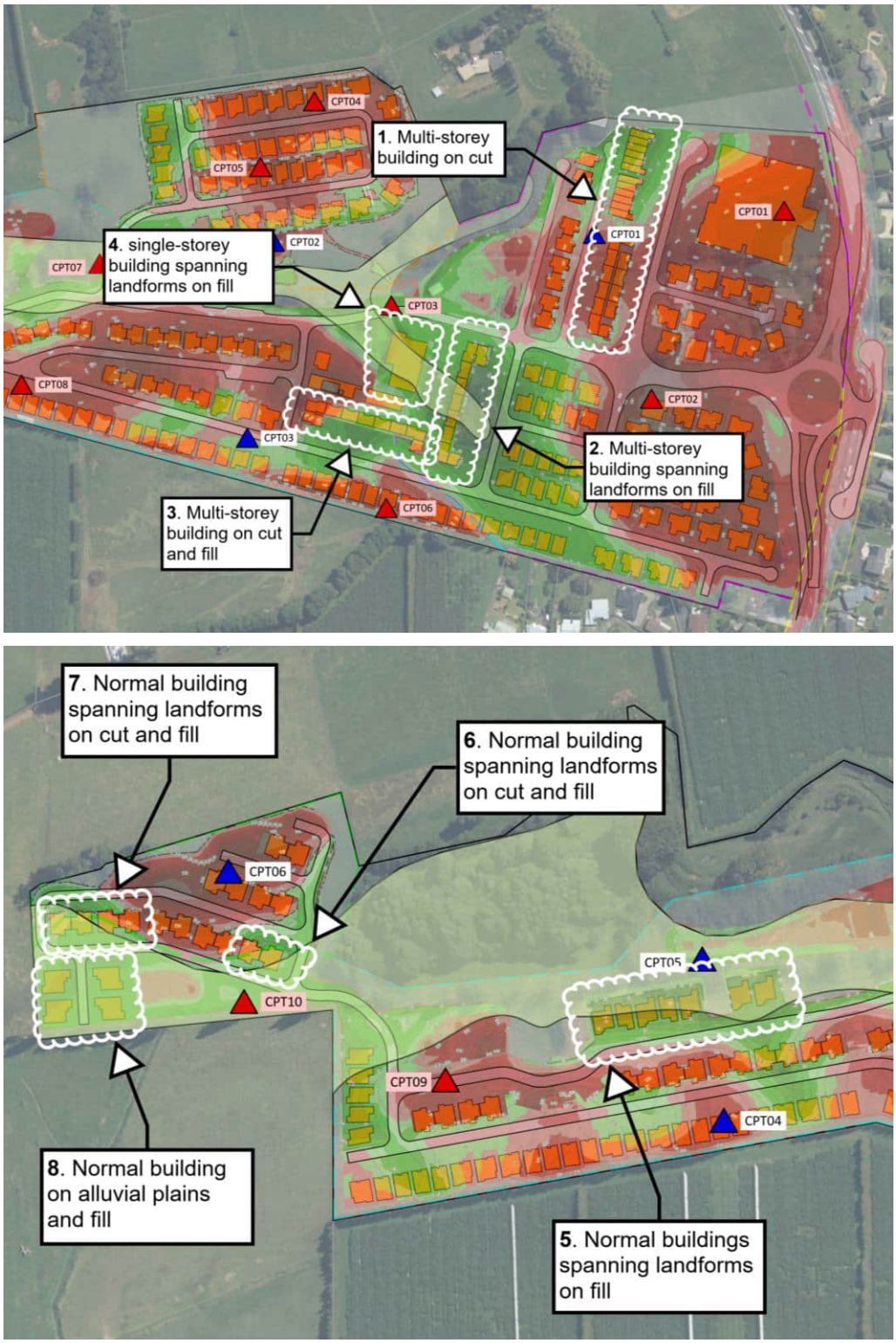


Figure 3: Plan showing cut to fill, landforms, and geotechnical investigations with areas of high settlement risk highlighted and numbered

Table 4: Site settlement assessment summary

Area No.*	Loading	Settlement risk factors	Nearest CPT
1	- Multi-storey buildings	- High building load	01 - Titus
2	- Fill (up to 8m) - Multi-storey building	- High building load - High fill load - Spans landforms (differential)	02 - BTW 06 - BTW
3	- Fill (up to 4m) - Multi-storey building	- High building load - High fill load - Spans cut and fill (differential)	03 - Titus 06 - BTW
4	- Fill (up to 8m) - Single-storey building	- High fill load - Spans cut and fill (differential)	03 - BTW
5	- Fill (up to 4m) - Normal residential building	- High fill load - Spans landforms (differential)	05 - Titus
6**	- Fill (up to 2m) - Normal residential buildings	- High fill load - Spans landforms (differential) - Spans cut and fill (differential)	10 - BTW
7**	- Fill (up to 2m) - Normal residential building	- High fill load - Spans landforms (differential) - Spans cut and fill (differential)	10 - BTW
8**	- Fill (up to 2m) - Normal residential building	- High fill loads - Alluvial plains	10 - BTW

* Area numbers correspond to buildings shown in Figure 3.
** Compiled together for calculation.

Results of the settlement analysis are attached in **Appendix H**.

5.7.1 Areas 1, 2, 3 (Figure 3): Multi-storey Buildings

The proposed multi-storey buildings have been assessed for a nominal building load of 40kPa over an area of 12m by 70m as per the development plans. Fill loads are based on the proposed fill depth multiplied by a soil unit weight of 20kN/m^3. The analysis results are summarised in the table below:

Table 5: Calculated building settlements summary for areas 1, 2, and 3 (Figure 3)

Area No.	CPT No.	Fill depth (m)	Load (kPa)	Settlement (mm)		
				Primary	Secondary	Total
1	01 - Titus	-	40	27	7	34
2	02 - BTW	8	200	194	19	213

Area No.	CPT No.	Fill depth (m)	Load (kPa)	Settlement (mm)		
				Primary	Secondary	Total
	06 – BTW			205	22	227
3	03 - Titus	4	120	200	5	205
	06 - BTW			122	22	144

Area 1. Results indicate settlement in the order of 27mm of primary and 7mm of secondary. The NZBC[16] B1/VM2 requires less than 25mm over a horizontal distance of 6m (1 in 240). As such, the calculated settlement is considered acceptable and no specific mitigation is required.

Area 2. Results indicate settlement in the order of 200mm of primary and 20mm of secondary. These buildings are also shown to span landforms. As such differential settlement may occur.

Area 3. Results indicate the potential for settlement in the order of 120-200mm of primary and 5-20mm of secondary.

5.7.2 Areas 4 (Figure 3): Single-storey building on fill

The proposed single-storey building has been assessed for a nominal building load of 10kPa over an area of 26m by 40m as per the development plans. Fill loads are based on the proposed fill depth multiplied by a soil unit weight of 20kN/m^3. The analysis results are summarised in the table below:

Table 6: Calculated building settlements summary for area 4 (Figure 3)

Area No.	CPT No.	Fill depth (m)	Load (kPa)	Settlement (mm)		
				Primary	Secondary	Total
4	03 - BTW	4	90	215	33	248

Area 4. Results indicate the potential for settlement in the order of 215mm of primary and 30mm of secondary.

5.7.3 Areas 5, 6, 7, 8 (Figure 3): Fill Placement on Alluvial Plains

The fill proposed on the alluvial plains has been assessed for a conservative soil unit weight of 20kN/m^3 over a nominal 50m by 50m area with a nominal two-story residential building load of 20kN/m^2 added. The analysis results are summarised in the table below:

[16] Ministry of Business Innovation & Employment. (2014). The New Zealand Building Code.

Table 7: Calculated building settlements summary for areas 4, 5, 6, and 7 (Figure 3)

Area No.	CPT No.	Fill depth (m)	Load (kPa)	Settlement (mm)		
				Primary	Secondary	Total
5	05	4	100	60	16	76
6-8	10 (BTW)	2	60	180	12	192

Area 5. Results indicate settlement in the order of 60mm for primary and 16mm for secondary. This magnitude of non-differential settlement is considered acceptable, however given that the buildings proposed in this area span landforms moderate differential may be expected.

Area 6 and 7. Results indicate settlement in the order of 180mm of primary and 12mm of secondary. These buildings are also shown to span landforms, and cut and fill. As such differential settlement is expected.

Area 8. Results indicate the potential for settlement.

Calculated primary consolidation settlements within Areas 2 through 8 will be effectively managed prior to building construction by implementing the static settlement recommendations provided in Section 8.4. This involves various mitigation measures such as adopting a 1m to 2m overfill pre-load, undercutting compressible organic/soft alluvium from gullies, and carrying out settlement monitoring until ≥90% consolidation is achieved (or alternatively founding structures on piles embedded into competent strata). Provided these recommendations are adopted post-construction total and differential settlements will be reduced to negligible levels and satisfy NZBC B1/VM2 tolerances.

5.8 Soil Expansivity

As per AS 2870[17] clays swell and shrink due to changes in moisture content. Soils dominated by halloysite and allophane, such as those found within the low hills (Hamilton Hills), have a low expansion potential as the clay minerals do not rehydrate[18,19,20,21]. Therefore, these have been considered as Class S surface movement (0-22mm) as per Section 7.5.13.1.2 of NZBC and no mitigation is considered necessary.

5.9 Liquefaction

Liquefaction occurs during an earthquake in loose, saturated cohesionless soils. This process leads to a temporary loss of soil strength and can result in vertical and lateral ground deformation.

[17] Standards Australia. (2011). *AS 2870-2011: Residential slabs and footings*. Standards Australia.
[18] Coduto, 2012, Geotechnical engineering: principles and practices, Upper Saddle River, N.J. PrenticeHall
[19] Joussein, et al 2005, Halloysite clay minerals – A Review, Clay Minerals (2005)
[20] Lowe & Percival, 1993, Clay Mineralogy of Tephras and Associated Paleosols and Soils and Hydrothermal Deposits, North Island, 10th International Clay Conference
[21] Maeda et al, 1977, Advances in Agronomy, Volume 29, American Society of Agronomy, Academic Press, Physical Properties of Allophane Soils (pages 229-264)

No liquefaction is anticipated within the low hills landform as this is generally underlain by greater than 10 meters of unsaturated, cohesive, non-liquefiable soils of the Hamilton Ash Formation and Walton Subgroup. However, the conditions for liquefaction are present within the alluvial plains / gully landforms. The relevant CPT investigations within these landforms and the encountered GWT depths, are shown in Table 1 and have been assessed below.

The liquefaction assessment made in this report is classified as a Level D study (MBIE, MFE, EQC)[22] and will evaluate liquefaction potential at this location. A seismic analysis was undertaken using the software CLiq together with the above-mentioned CPT data and correlated with hand auger testing and the site geology / geomorphology. The following assessment methods were adopted:

- Boulanger and Idriss (2014) calculation method
- Boulanger and Idriss (2014) fines correction
- Zhang et al. (2002) free-field liquefaction induced settlements
- Limited to 10m depth
- Ic=2.6 cutoff

The in-situ GWT was modelled at the depths encountered in accordance with Table 1, and inferred from the ground models where it was not encountered. A GWT elevated by 1m has been adopted for earthquake GWT to represent winter groundwater conditions.

Peak Ground Acceleration (PGA) values have been determined as per Section 5.3.

5.9.1 IL2 Results

The analysis results for the IL2 ULS earthquake events are shown in the table below together with the Performance Level for liquefied deposits as characterised in the MBIE Modules[23]. Results are attached in **Appendix I**.

Table 8: Liquefaction analysis results summary

CPT	Liquefaction Potential Index (LPI)	Liquefaction Severity Number (LSN)	Vertical Settlement (mm)	Performance Level
05 - Titus	5.7*	13.5***	38	L2‡
07 - BTW	6.4*	8.8†	39	L2‡
10 - BTW	2.8**	3.6†	19	L2‡

* High risk

** Low risk

*** Minor expression of liquefaction

† Little to no expression of liquefaction

[22] Earthquake Commission., Ministry of Business Innovation & Employment., Ministry for the Environment. (2017). Planning and engineering guidance for potentially liquefaction prone land

[23] Ministry of Business Innovation & Employment., New Zealand Geotechnical Society. (2021). Earthquake geotechnical engineering practice Modules

CPT	Liquefaction Potential Index (LPI)	Liquefaction Severity Number (LSN)	Vertical Settlement (mm)	Performance Level
‡ Moderate: Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results relatively small in differential settlements (MBIE, NZGS, 2021).				

The liquefaction analysis results for the IL2 ULS earthquake summarised in the table above indicates the potential for moderate liquefaction to occur at this site within the alluvial plains / gully landforms. Predicted vertical settlements are in the order of 19mm to 39mm. Liquefaction recommendations are provided in Section 8.3.

A parametric liquefaction graph is presented in **Appendix I** which plots vertical settlements against PGA values and highlights the magnitude of ground shaking required to induce liquefaction ground settlement. This graph shows that no settlement is expected following the SLS seismic event (PGA of 0.06g) and that settlements tend to plateau beyond the ULS event PGA values.

5.9.2 IL3 Results

The analysis results for the IL3 ULS earthquake indicate vertical settlements within the alluvial plains / gully show up to 43mm, and within the low hills range from 0mm to 21mm. Liquefaction recommendations are provided in Section 8.3.

5.9.3 Lateral Spread

Lateral spread is defined as lateral displacement of a soil layer riding on liquefied soil either down a gentle slope or toward a free face, such as an incised river channel. Following the onset of liquefaction, lateral spread may be expected where a site is located within 100m of a free face less than 2m high, or within 200m of a face greater than 2m high (MBIE).

The gully and proposed stormwater channel may be susceptible to lateral spread and have therefore been assessed below based on the relevant CPT data. According to Youd[24] liquefiable layers must be within twice the height (2H) in order to generate lateral spread. Based on the results of the liquefaction assessment the following liquefiable layers have been assessed for lateral spread:

Table 9: Liquefiable layers summary

CPT No.	Liquefiable Layer No.	Depth to top of layer (m)	Layer Thickness (m)
10 - BTW	1	5.1	0.8

[24] Youd, T. L. (2018). Application of MLR procedure for prediction of liquefaction-induced lateral spread displacement. Journal of Geotechnical and Geoenvironmental Engineering, 144(6). https://doi.org/10.1061/(asce)gt.1943-5606.0001860

CPT No.	Liquefiable Layer No.	Depth to top of layer (m)	Layer Thickness (m)
05	1	2.7	0.3
	2	3.2	1.3
	3	4.8	1.1
07	1	1.9	0.5
	2	4.6	1.2

As shown in **Appendix E**, the liquefiable layers within CS-A and CS-B begin at a depth greater than 2H. Additionally, no liquefiable layers are present in CS-E; therefore, these cross sections have not been assessed for lateral spread.

However, CS-B and CS-C meet the conditions for the development of lateral spread. The liquefiable layers identified above have been assessed as part of the slope stability analysis in the section below to quantitatively determine the lateral spread risk to the affected lots in accordance with the design safety factor.

5.10 Slope Stability

The cross sections as per Section 4.4 have been assessed for global stability below. These represent areas that are of the greatest global instability risk particularly with respect to the proposed retaining walls and stormwater ponds.

Slope stability modelling has been undertaken in accordance with the MBIE Guidance Units[25] using the proprietary software programme *Slide2* published by RocScience. The Morgenstern-Price method was adopted and checked for circular and non-circular failure mechanisms. The Slide2 analyses determines the theoretical failure surfaces of deep-seated (global) slope failure arcs based on the slope modelled.

The stability of slopes under a range of design conditions is expressed in terms of a factor of safety (FoS) which is defined as the ratio of forces resisting failure to the forces causing failure. The following scenarios and recommended minimum required FoS for the assessment have been considered.

Table 10: Minimum required Factors of Safety

Modelled Loading Condition	Required FoS
Static Condition (normal GWT)	1.5
Static Condition (intense rainfall event)	1.3
Seismic SLS (Serviceability Limit State)	1.3

[25] Ministry of Business Innovation & Employment., New Zealand Geotechnical Society. (2025). Slope Stability Geotechnical Guidance Series, Unit 3, Slope Stability Analysis

Modelled Loading Condition	Required FoS
Seismic ULS (Ultimate Limit State)	1.0
Post-seismic ULS (liquefied soils)	1.0

5.10.1 Design Parameters

Peak Ground Acceleration (PGA) values have been determined as per Section 5.3.

The proposed buildings, roads, and ponds have been incorporated into the slope model in the locations shown on the development plans as the following surcharges:

- Single-storey building: 5 kN/m^2
- Two-storey buildings: 15 kN/m^2
- Three-storey (apartment) buildings: 30 kN/m^2
- Roads: 5 kN/m^2
- Stormwater ponds: 10-15 kN/m^2

The proposed timber pole retaining walls were modelled with a nominal 100kN/m shear strength and an embedment equal to the height of the wall. Internal block walls were modelled with a nominal 300mm of embedment.

Transient short-term analyses were based on elevated pore water pressures to simulate wetting fronts moving through the profile under intense rainfall events. Porewater pressure ratios (Ru) of 0.3 and 0.2 were applied to all layers above the water table.

Liquefied layers have been included within the lateral spreading scenarios for the relevant cross sections based on the liquefiable layers described in Table 9.

Drained and undrained shear strength parameters for the various geological units that underlie the site were inferred from the field investigation data. The geotechnical properties for the soils within the slope stability modelling are presented in the table below.

Table 11: Soil geotechnical parameters

Unit	γ (kN/m^3)	c' (kPa)	ϕ' (deg)	Su (kPa)	K
Engineered fill	17	7	32	200	-
Alluvium	16	3	28	65	-
Hamilton Ash Formation	17	5	30	100	-
Karapiro Formation	17	7	32	150	-
Puketoka Formation	20	0	40	-	-
Liquefied layer	17	-	-	5	0.07

Note: γ = soil unit weight; c' = effective cohesion; ϕ' = effective friction angle; Su = undrained shear strength; K = vertical stress ratio

5.10.2 Results

Slope stability results are summarised in the table below and presented in **Appendix J**.

Table 12: Assessed slope stability FoS for modelled load cases

Modelled Loading Case	Assessed Factor of Safety (FoS)						
	CS-A	CS-B	CS-C		CS-D		CS-E
			L-R	R-L	L-R	R-L	
Static Condition (normal GWT)	1.89	2.51	1.62	2.27	1.87	1.72	2.87
Static Condition (intense rainfall event)	1.53	2.19	1.30	1.92	1.53	1.39	2.13
Seismic SLS (Serviceability Limit State)	4.13	5.17	5.21	8.74	4.72	3.75	4.14
Seismic ULS (Ultimate Limit State)	1.94	2.00	2.16	3.33	2.00	1.76	1.66
Post-seismic ULS (liquefied soils)	N/A	N/A	4.28	2.21	3.96	4.76	N/A

The results of the modelling indicates that the proposed slope conditions meet the requisite FoS for the range of modelled scenarios. Slope instability recommendations are provided in Section 8.2.

6 FILL SUITABILITY

6.1 Overview

Hamilton Ash Formation (HAF) soil samples and one sample of the Rangitawa Tephra were tested as per Section 3.3 to determine the suitability of these soils for engineered fill and to provide soil specifications for bulk earthworks quality control. The Walton Subgroup soils were determined in the field to be unsuitable for use as engineered fill due to having low undisturbed and disturbed shear strengths and were therefore not tested.

The order of samples listed in the table below broadly corresponds to the changing consistency / fines content / mineralogy of the Hamilton Ash Formation (HAF) with depth encountered within the test pits. The lab results for the soil samples are summarised in the table below:

Table 13: Summary of laboratory tested soil sample specifications

Soil Unit	Source and depth	Material type	MDD (t/m^3)	NMC (%)	OMC (%)	SD (t/m^3)	Corrected shear vane (kPa) at OMC & NMC
Hamilton Ash Formation	TP02 3.2m	Brown Silt	1.05	58.5	56	2.73	112 & 63
Hamilton Ash Formation	TP03 2.2m	Light brown Clay with silt	1.07	56.5	52	2.78	166 & 86
Hamilton Ash Formation	TP01 4.5m	Brownish grey Clay	0.99	69.2	60	2.76	123 & 60
Hamilton Ash Formation	TP05 5.0m	Light brownish grey silty Clay	0.91	89.3	66	2.80	112 & 37
Rangitawa Tephra	TP01 5.5m	Grey silty Clay	1.29	38.7	38	2.70	~66

6.2 Suitability

The table above shows that the Rangitawa Tephra is not suitable for use as engineered fill due to it not being able to achieve the required 150kPa shear strength as per NZS 4431.

The table also indicates that the Hamilton Ash Formation soils are generally suitable for use an engineered fill and that the 95% MDD compaction target may be achieved with minimal moisture conditioning. However, the table highlights that this will not be true of deeper cuts as the difference between the natural moisture content (NMC) and the optimum moisture content (OMC) becomes significant near the base of the HAF. This means that that OMC and therefore the compaction target may not be achievable for some of the soils within the HAF.

As such it is recommended that the deeper cuts are mixed with the surface HAF soils. Provided these recommendations are followed, the HAF soils are considered suitable for use as engineered fill. As per the proposed cut to fill plan the site has an excess of cut materials meaning the best HAF soils can and should be selectively used for use as fill. Earthworks recommendations are provided in Section 8.1.

7 EARTHWORKS SPECIFICATION

7.1 General

This earthworks specification is intended for the site bulk earthworks and does not cover any road subgrade requirements or specifications.

The proposed cut to fill earthworks are outlined in Section 1.3. This generally involves cut up to 6m and fill up to 8m.

All earthwork activities must be carried out in accordance with the requirements of NZS 4431[26], Waikato Regional Council, RITS[27], Specification for Earthworks Construction TNZ F/1[28], WorkSafe New Zealand, and any other relevant documentation under the guidance of a suitably qualified geoprofessional.

7.2 Engineered Fill Requirements

Imported fill shall be discussed and approved by the project geotechnical engineer. Appropriate testing or material certificates shall be provided to verify material suitability prior to importing. It is noted that no imported fill (other than roading and drainage) has been proposed at this stage.

The following requirements shall be adhered during the placement of fill:

- All engineered fill shall be free of topsoil, organic matter, rubbish, and other unsuitable materials.
- Fill lifts shall be no greater than 250mm loose depth. The contractor must ensure even and adequate compaction throughout each layer/lift.

7.3 Subgrade preparation

All vegetation including roots with a diameter greater than 25mm shall be removed. Any tree-stumps are to be over-excavated. Cleared areas shall then be stripped and topsoil stockpiled.

The prepared subgrade shall be observed by Titus prior to fill placement to confirm the strength of the underlying soils. Where any weak materials are encountered, they should be undercut.

Fill shall be keyed into the underlying natural ground where slopes exceed 1V:5H and should be carried out in accordance with the normal requirements of NZS 4431.

Given the variable soils of the alluvial plains / gully landforms, weak material should be undercut at the direction of the project geotechnical engineer during subgrade inspections prior to the placement of fill.

[26] NZS 4431:2022 – Code of Practice for Earth Fill for Residential Development – New Zealand Standards, Wellington, New Zealand.
[27] Local Authority Asset Management (LAAM). (2018). Regional infrastructure technical specifications (Version 2.0)
[28] Transit New Zealand. (1997). *Specification for earthworks construction* (TNZ F/1:1997)

7.4 Quality Control

The table below outlines the compaction control criteria for the bulk earthworks in accordance with NZS 4431. Fill soil properties for compaction control are provided in Table 13.

Table 14: Compaction control criteria

	Test	Test apparatus and method/s	Minimum test frequency	Acceptance criteria
Fine-grained / cohesive soils **E.g. clays & silts**	Field water content and density	Nuclear densometer NZS 4402 test 2.1 and NZS 4407 4.1, 4.2, or 4.3	2 per 1000 m^3 (min 2 per lift)	≥ 95% maximum dry density ≤ 10% air voids
	Shear strength	Shear vane NZGS Guideline for hand held shear vane	2 per 1000 m^3 (min 2 per lift)	Lowest value ≥ 150kPa
Coarse-grained / granular soils with a particle size <65mm **E.g. sand**	Field water content and density	Nuclear densometer NZS 4402 test 2.1 and NZS 4407 4.1, 4.2, or 4.3	2 per 1000 m^3 (min 2 per lift)	≥ 95% maximum dry density MDD shall be provided by lab testing.
	Relative density	Dynamic cone penetrometer NZS4402 test 6.5	2 per 1000 m^3 for full depth of lift (min 2 per lift)	≥ 5 blows / 100mm
General notes: • Testing frequency may vary at the discretion of the geotechnical engineer. • Laboratory moisture correction testing must be carried out for all NDM tests in cohesive soils.				

7.5 Slope Stability of Temporary and Permanent Cut/Fill Batters

7.5.1 Temporary Batters

Temporary stability of the works is the responsibility of the earthworks contractor. To help mitigate the risk of slope instability/side wall collapse, it is recommended that the contractor:

- Limits excavation batters to 1V:2H and 3m in height.
- Diverts surface water runoff away from excavation batters.
- Avoids loading the crests of slopes and excavations with working plant and/or stockpiles. Temporary stockpiles shall be setback at least 15m from the crest of any slope or drain and should be limited to flat to gently sloping ground.
- Has in place risk identification and management procedures, and work methodologies for temporary excavation stability.
- Seeks advice from the project geotechnical engineer when questioning the stability of a slope of excavation.

7.5.2 Permanent Batters

Cut and fill batters are to be formed at the angles specified on the approved civil drawings and shall be topsoiled and stabilised as soon as reasonably practicable following completion to minimise the risk of erosion.

7.6 Stormwater

The permanent stormwater ponds are proposed to be constructed in a combination of natural ground and engineered fill.

Pond excavations shall be observed by Titus during construction to advise on any remediation of unsuitable soils that might be encountered to avoid slumping/erosion and scouring over the lifetime of the pond. This may involve re-compaction of the natural soils, undercut and replacement with engineered fill, or erosion protection matting.

7.7 Groundwater

Subsurface drainage, such as the use of a drainage blanket (and subsoil drains including drainage metal), should be implemented below areas of deep fill proposed over natural overland flow paths or in places where springs or groundwater is encountered. The locations of these shall be in accordance with the final civil design drawings. Recommended locations are provided in Section 8.1. The location of subsurface drainage shall be confirmed and varied as needed on site by Titus at the time of construction in response to ground conditions encountered.

Groundwater may be encountered during the excavation of proposed Stormwater Pond C. Should groundwater be encountered during earthworks, the Contractor shall dewater as necessary to control the water level in the excavation and provide their construction and dewatering methodology to the Titus prior to construction. The Contractor shall obtain any consent required for dewatering and conform to the relevant consent conditions.

7.8 Erosion and Sediment Controls

The specific design of the erosion and sediment control plan is outside the scope of this report and should be submitted and approved prior to the commencement of earthworks in accordance with Waikato Regional Council erosion & sediment control guidelines.

The following requirements are noted with respect to temporary sediment retention ponds:

- The construction of temporary sediment retention ponds shall be observed by Titus.
- All water softened material in the bases and sides of the ponds shall be removed to the satisfactions of Titus following pond decommissioning.
- The backfill standard of temporary ponds shall be to that of engineered fill unless otherwise specified.

7.9 Settlement Monitoring

Settlement monitoring should be undertaken during and following earthworks as per Section 8.4.

Settlement markers should be installed at the locations shown in **Appendix F**. The location and number of settlement markers may be varied on site by Titus at the time of construction in response to ground conditions encountered.

Equipment shall consist of 600mm x 600mm base plate (typically 20mm thickness), 150mm PVC duct / tube, 32mm diameter threaded bar with couplers for extensions at 1m intervals and waratahs and coloured flagging for protection following installation.

Inner bars will be attached to the plate and must be capable of extensions up through the fill as the height of the fill increases. The extensions shall be coupled to the previous bar using a screw head.

Compaction of the fill surrounding the settlement plate and ducts/ tubes shall be by hand to prevent damage. Any damage that occurs must be communicated to the project geotechnical engineer and must be reinstated immediately.

Settlement markers shall be surveyed fortnightly or every 500mm lift, whichever is more frequent, during construction within an accuracy of 2mm. Once the design ground level is achieved the markers shall be surveyed on a fortnightly basis for a minimum of 6 months or as directed by the project geotechnical engineer. Results must be provided to the engineer within 24 hours of each reading.

7.10 Finishing Works and Topsoil Spread

All areas cut to below finished level should be reinstated with engineering filling to the satisfaction of the project geotechnical engineer.

Topsoil respread thickness should be between 100mm and 300mm, or as directed by the Engineer to Contract.

Testing and formation of road subgrades will be carried out as part of the subdivision civil works package.

7.11 Construction Monitoring Requirements

The following items require observation, testing and approval by Titus. It is the contractor's responsibility to ensure the project geotechnical engineer is given acceptable notice and opportunity to observe the below works and that they do not proceed until approval has been gained from the engineer.

- Prepared subgrade prior to fill placement including any undercuts.
- Pond excavations.
- Decommissioning of temporary sediment retention ponds.
- Laboratory test results any imported soil fill materials prior to placement on site.
- Fill testing at regular intervals in compliance with Table 14.
- Retaining walls including pile holes, drainage, and poles/laggings.

- Subsoil drainage.
- Surveyed settlement monitoring data.
- Any unforeseen ground conditions that may impact the construction works or future land use.
- Post construction hand augers on completion of bulk earthworks.

7.12 As-built Information Requirements

The following as-built information is required from the contractor for Titus to complete the Geotechnical Completion Report (GCR). It is the contractor's responsibility to ensure that all the following items are surveyed.

- Cut and fill plan (including undercuts)
- Final surface contour plan
- Details of any defined overland flow paths
- Locations of temporary ponds which have been backfilled including depth of fill
- Material data for imported products and fill material
- As-built details of any retaining walls and ponds
- Settlement monitoring data

8 GEOTECHNICAL RECOMMENDATIONS

These recommendations are considered suitable for the proposed development as per Section 1.3. However, any changes to the proposed development will trigger the need for a plan review by the project geoprofessional engineer to assess the suitability of our recommendations.

8.1 Earthworks

All earthwork activities must be carried out in accordance with the Earthworks Specification outlined in Section 7 of this report.

As per the findings of the fill suitability assessment in Section 6, it is recommended that the deeper cuts (generally below 4-5m) should be wasted where possible and where not possible should be mixed with the overlying cut soils. Provided this is complied with, the HAF soils are considered suitable for use as engineered fill. Rangitawa Tephra and Walton Subgroup soils are not suitable for engineered fill. The HAF generally extends to the depth of proposed cut. Unforeseen cut to waste of the lower HAF may be expected which shall be confirmed by the project geoprofessional engineer during quality control.

It is recommended that the design of permanent cut and fill batters be limited to a maximum gradient of 1V:2.5H and limited to 3m high. Batters greater than 3m should adopt a maximum gradient of 1V:3H. The civil design drawings shall be assessed and approved by a suitably qualified geoprofessional prior to being finalised.

Subsurface drainage, such as the use of a drainage blanket (and subsoil drains), should be implemented below areas of deep fill proposed over natural overland flow paths and where groundwater is encountered during earthworks. The locations of these shall be in accordance with the final civil design drawings with the figure below highlighting areas of greatest importance. These are to be reviewed by the project geotechnical engineer.

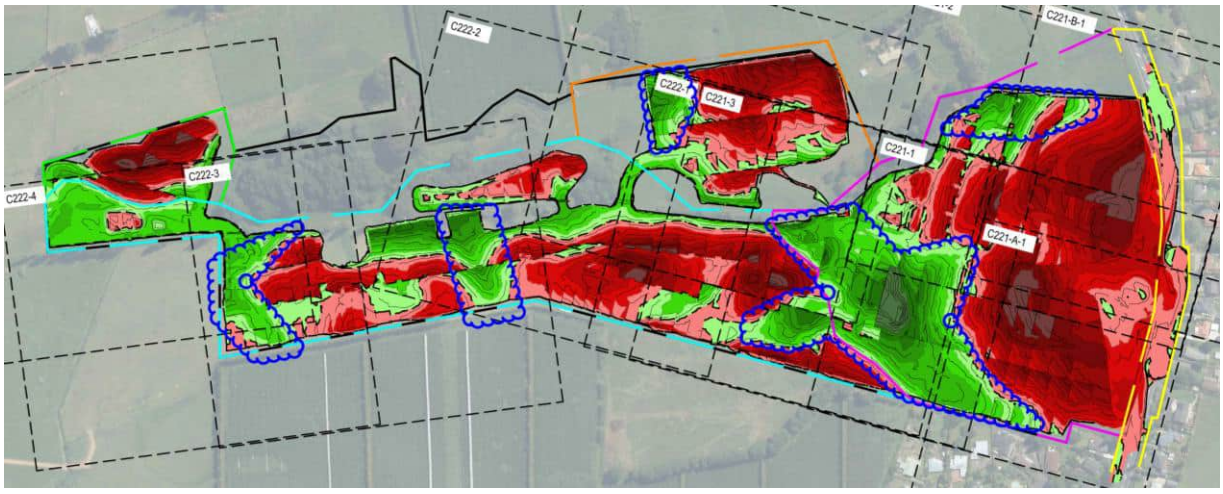


Figure 4: Cut to fill plan - highlighted areas in blue show where subsoil drainage blankets are recommended

8.2 Retaining Walls

The results of the slope stability assessment are detailed in Section 5.10 and demonstrate that the proposed retaining walls are globally stable for the development.

It is recommended that all development retaining walls be subject to specific engineering design (SED) to accommodate the proposed surcharges. The walls shall be designed based on the soil parameters provided in Table 11 in accordance with the layer depths provided in Table 2 and as shown in **Appendix E**.

8.3 Liquefaction

The results of the liquefaction analysis are discussed in Section 5.9 and summarised below:

- The low hills landforms have been classified as TC1-like for IL2 and IL3 buildings.
- The alluvial plains landform in the far west of the site has been assessed as being TC1-like.
- The gully landform has been assessed as being TC2-like, however it is noted that the only buildings proposed within this landform (Stage 2C as shown by CS-C in **Appendix E**) are founded on up to 4-5m of engineered fill which is sufficient to mitigate the surface expression of liquefaction. Therefore, TC1 foundations may be adopted.
- The buildings proposed over the existing pond (Stage 2A) are recommended, as per Table 15, to have all alluvium removed thereby removing potentially liquefiable soils. As such TC1 foundations may be adopted.

Based on the above, no liquefication mitigation is required for all buildings proposed as per the attached development plans. Should the development change, a plan review must be undertaken by the project geoprofessional to assess the suitability of our recommendations.

8.4 Static Settlement

The results of the settlement assessment and recommended mitigation method are summarised in the table below for the assessed areas as per Figure 3.

Table 15: Site settlement assessment summary

Area No.	Settlement result	Mitigation
1	- Low	- None
2	- High primary - Low secondary - Differential	- Settlement monitoring - Overfill by 2m - Remove all alluvium / softened soil from gully prior to fill placement
3	- High primary - Low secondary - Differential	- Settlement monitoring - Overfill by 2m

Area No.	Settlement result	Mitigation
4	- High primary - Low secondary - Differential	- Settlement monitoring - Overfill by 1m - Remove all alluvium / softened soil from gully prior to fill placement
5	- Low to moderate primary - Low secondary - Differential	- Settlement monitoring - Overfill by 1m
6	- High primary - Low secondary - Differential	- Settlement monitoring - Overfill by 1m
7	- High primary - Low secondary - Differential	- Settlement monitoring - Overfill by 1m
8	- High primary - Low secondary	- Settlement monitoring - Overfill by 1m
* Area numbers correspond to buildings shown in Figure 3.		

It is recommended that settlement monitoring be carried out during and following the placement of fill within the above areas. These areas should be pre-loaded by overfilling to allow for future building loads and to expedite settlement. Given the low secondary settlement results, a monitoring duration of 6 months is considered appropriate. Monitoring should be carried out until such time as greater than 90% of settlement has occurred. Settlement markers should be installed at the locations shown in **Appendix F** and in accordance with Section 7.9.

Alternative to the above recommendations, buildings located within a high-risk area may be founded on piles bearing on a competent soil layer below all fill.

Provided these recommendations are followed, all settlement including differential is considered fully mitigated and no specific engineer designed foundations for settlement mitigation are required.

8.5 Preliminary Foundations

The bearing capacity of the alluvial plains / gully landforms is highly variable. However, all buildings proposed in this landform are shown on engineered fill which by its nature will provide a geotechnical ultimate bearing capacity (GUBC) of at least 300kPa. The low hills landform HAF soils are capable of providing a GUBC of at least 300kPa.

Provided the recommendations in this report are followed no specific geohazard foundation mitigation is required. Therefore, based on the information supplied at this time the proposed single and two storeyed buildings may adopt standard NZS 3604:2011/proprietary raft foundations provided the specific criteria of those foundation types are met. Foundations for the multi-storey apartment buildings shall be engineer designed as they are outside the scope of NZS 3604. This assessment is subject to the findings of the Geotechnical Completion Report.

9 SUMMARY OF KEY FINDINGS AND CONCLUSIONS

The key findings and recommendations of the report are summarised below:

- Hamilton Ash Formation soils are considered suitable for use as engineered fill. Walton Subgroup soils are not suitable for engineered fill.
- Subsurface drainage should be implemented below areas of deep fill, where fill is proposed over natural overland flow paths (shown in Figure 4) and wherever groundwater is encountered during earthworks construction.
- The design of permanent cut and fill batters are limited as per Section 8.1.
- The civil design drawings shall be assessed and approved by a suitably qualified geoprofessional prior to being finalised.
- Proposed retaining walls are globally stable. All retaining walls shall be specific engineer designed.
- No liquefaction mitigation is required for the proposed buildings.
- Settlement monitoring (expected to be completed within 6 months of completion of earthworks) is to be carried out during and following the placement of fill including pre-loading.
- Provided the recommendations in this report are followed no additional specific geohazard mitigations are required for foundation designs.
- Post construction hand augers are required on completion of bulk earthworks.
- Construction monitoring is required as per Section 7.11 and as-built information is required as per Section 7.12.

This report has identified and assessed the principal geotechnical risks affecting the proposed development including liquefaction susceptibility, settlement risks, elevated groundwater levels, retaining structure requirements, and slope stability considerations. The report findings demonstrate that the main risks requiring mitigation include the temporary stability of excavations during earthworks, settlement induced by the placement of fill on alluvial and colluvial soils, and the management of groundwater flows below fill. It has been shown that these can be appropriately managed and mitigated by adopting the recommendations provided in this report.

Based on the investigations undertaken, modelling completed, and mitigation measures proposed, we consider that the construction and operation of the Harlow Lifestyle Village can be appropriately managed.

10 LIMITATIONS

This report does not assess risk of contamination of soils. This report does not provide, a foundation design, an assessment of flood risk or a FFL recommendation. This report has not assessed buoyancy of underground structures or stormwater ponds / wetlands. This shall be addressed as part of detailed engineering design.

Testing portrays a limited percentage of ground conditions at 2025 Ohaupo Road, Te Awamutu and may not be representative of all soils present on site.

Assessment of the water table depth and moisture content is subject to seasonal variation.

During excavation and construction, the site should be examined by a suitably qualified engineer in order to assess whether the exposed subsoils are compatible with the inferred soil conditions on which the recommendations have been based and potentially further investigation and design rationalisation may be required.

This report has been prepared solely for Te Awamutu Developments Limited, its professional advisors and local authorities in relation to 2025 Ohaupo Road, Te Awamutu. No liability is accepted for its use for any other purpose or by any other entity. Reliance by other parties or future owners of the property on the information or opinions contained in the report shall be verified with Titus Consulting Engineers. This report should be read and understood in its entirety, including Appendices, prior to construction or development.

Should you be in any doubt as to the recommendations of this report it is essential that you discuss these issues with Titus Consulting Engineers prior to proceeding with any work based on this report.

The recommendations in this report do not supersede recommendations of other engineering reports and shall be considered in conjunction with all other information available for the site.

APPENDICES

APPENDIX A – Select Development Plans



1 Overall Proposed Level 1 1:1500

STAGE 1		
KEY	Typology	Yield
	1A	1
	2B	1
	2C	1
	2D	7
	PUD (Lower)	6
	PUD (Upper)	6
	3A	2
	3B	15
	3C	12
	3D	2
	4B	1
	3F2	4
	3G2	6
	Apartment - 1B	4
	Apartment - 2A	6
	Apartment - 2B	6
	Apartment - 2C	2
	Apartment - 3A	4
	Serviced Apartment 1A	16
	Serviced Apartment 1B	6
	Serviced Apartment 2A	6
		134

STAGE 2		
KEY	Typology	Yield
	1A	5
	2B	30
	2C	15
	2D	7
	PUD (Lower)	16
	PUD (Upper)	16
	3A	2
	3B	1
	3F2	2
	3G2	5
	Apartment - 1A	4
	Apartment - 1B	4
	Apartment - 1C	2
	Apartment - 2A	20
	Apartment - 2B	16
	Apartment - 3A	16
		200

STAGE 3		
KEY	Typology	Yield
	1A	10
	2B	6
	2C	4
	2D	7
	3B	7
	3C	7
		44

STAGE 4		
KEY	Typology	Yield
	PUD (Lower)	16
	3A	4
		20

Overall Yield		
	Typology	Key Yield
	1A	16
	2B	43
	2C	28
	2D	31
	PUD (Lower)	16
	PUD (Upper)	16
	PUD (Lower)	44
	PUD (Upper)	47
	3A	8
	3B	23
	3C	35
	3D	2
	4B	1
	3F2	6
	3G2	11
	Apartment - 1A	4
	Apartment - 1B	8
	Apartment - 1C	2
	Apartment - 2A	26
	Apartment - 2B	74
	Apartment - 2C	2
	Apartment - 3A	20
	Serviced Apartment 1A	16
	Serviced Apartment 1B	6
	Serviced Apartment 2A	6
		484

KEY:
- Kahikatea Forest, Wetland & Watercourse Restoration Area
- Proposed Stormwater Ponds
- Wastewater Pump Station
- Dates Shed
- Restoration Areas - refer Ecologist report
- Revised Wetland Areas - refer Ecologist report
- Watercourse - refer Ecologist report
- Site Boundary
- Aged Care

N

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2025 Ohaupo Road, Te Awamutu 3800

PACIFIC ENVIRONMENTS ARCHITECTS

Concept Design ISSUE

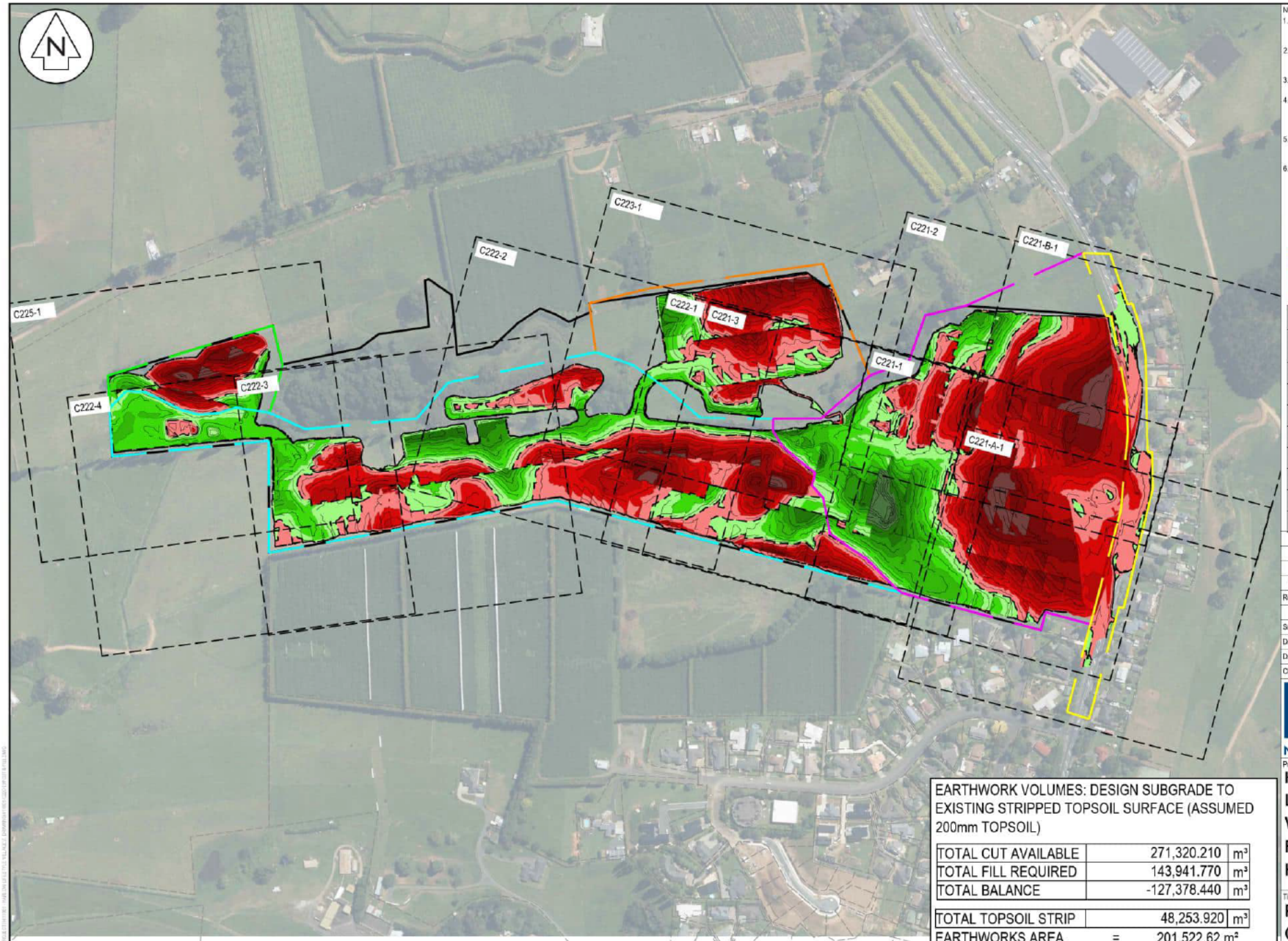
Title: Overall Proposed Site Plan - Level 1

Date: 14/09/2026 Scale: 1:1500, 1:1@A1 Client: Te Awamutu Developments Limited

P.O. Box 9827 Symonds St, Auckland, NZ
Ph. 09 969-0071 Email: mail@pearc.co.nz

Job No. 25020

Drawing No. A0100 Issue



NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL GUIDELINES, SPECIFICATIONS, AND REQUIREMENTS.
2. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
3. THE CONTRACTOR SHALL COMPLY WITH ALL RELEVANT HEALTH AND SAFETY REQUIREMENTS.
4. THE CONTRACTOR SHALL OBTAIN ALL NECESSARY APPROVAL FROM UTILITY OPERATORS BEFORE COMMENCING WORK UNDER OR NEAR THEIR SERVICES.
5. SEDIMENT CONTROL SHALL BE INSTALLED AND OPERATIONAL BEFORE EARTHWORKS START ONSITE IN ACCORDANCE WITH COUNCIL STANDARDS.
6. CONTRACTOR SHALL PROVIDE ASBUILT OF WORKING SEDIMENT CONTROL DEVICES AND CONFIRMATION OF PONDIDECENT VOLUMES TO ENGINEER.

LEGEND

- EX BDY
- PROP BDY
- PROP EXTENT WORK
- EW STAGE 1
- EW STAGE 2
- EW STAGE 3
- EW STAGE 4
- EW STAGE NZTA

Cut/Fill Table

Number #	Minimum Elevation	Maximum Elevation	Color
1	-8.000	-6.000	
2	-6.000	-4.000	
3	-4.000	-2.000	
4	-2.000	0.000	
5	0.000	2.000	
6	2.000	4.000	
7	4.000	6.000	
8	6.000	8.000	

EARTHWORK VOLUMES: DESIGN SUBGRADE TO EXISTING STRIPPED TOPSOIL SURFACE (ASSUMED 200mm TOPSOIL)

TOTAL CUT AVAILABLE	271,320.210	m³
TOTAL FILL REQUIRED	143,941.770	m³
TOTAL BALANCE	-127,378.440	m³
TOTAL TOPSOIL STRIP	48,253.920	m³
EARTHWORKS AREA	= 201,522.62 m²	

NOTE: NO ALLOWANCE FOR DRAINAGE AND SERVICE TRENCHES, VOLUMES ARE UNFACTORED AND IN SITU

RESOURCE CONSENT

Rev	Description	By	Date
A	RESOURCE CONSENT	SM	10/2026

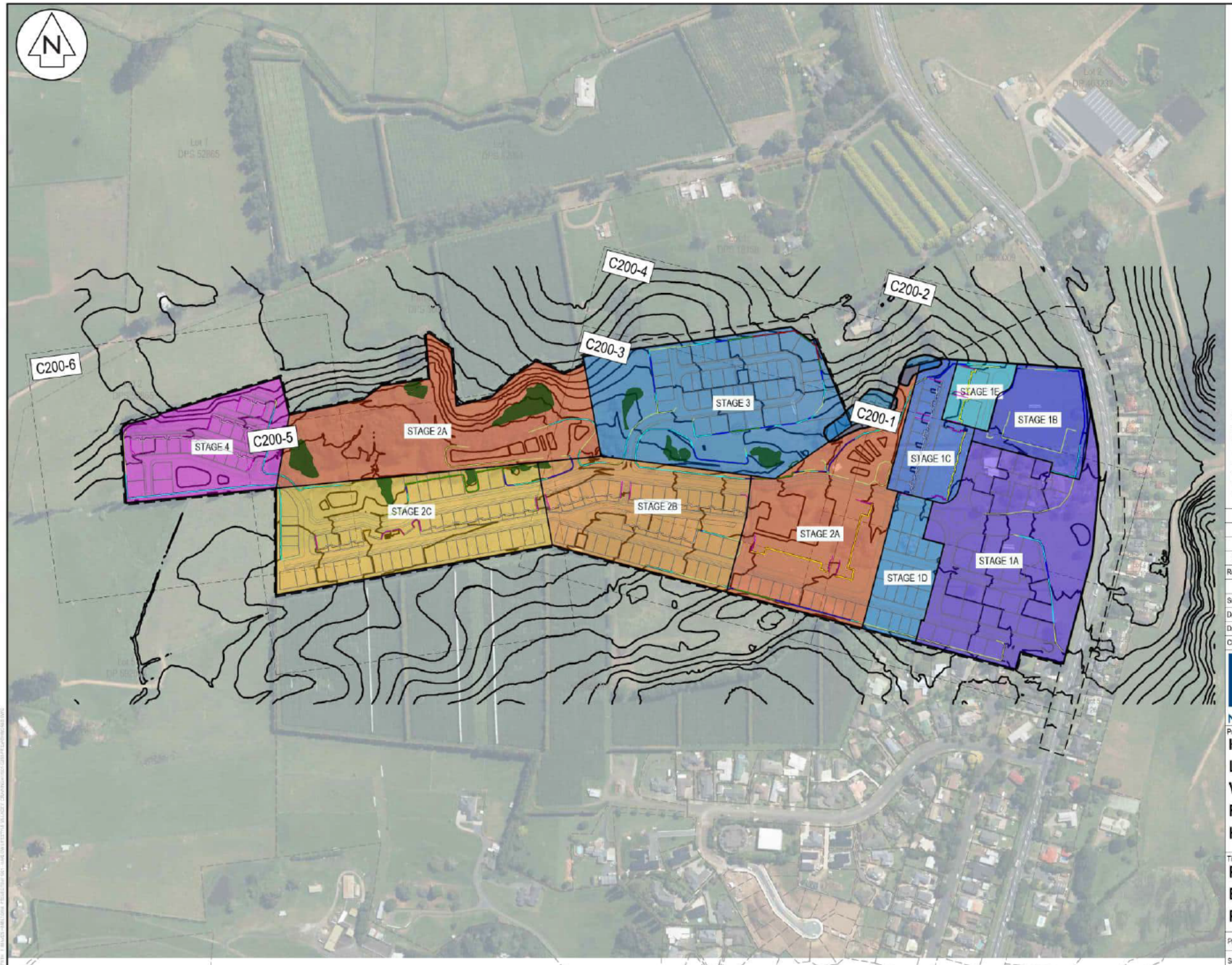
	By	Date
Survey	MAVEN	01/2026
Design	SM	03/2026
Drawn	NDL	03/2026
Checked	SM	10/2026

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Project:
HARLOW LIFESTYLE VILLAGE FOR HARLOW LIFESTYLE LTD

Title:
PROPOSED CUT/FILL (SITE) PLAN

Project no.	411001
Scale	1:4000 @ A3
Cad file	411001-C220-EW CUT & FILL.DWG
Drawing no.	C220-1 Rev A



RESOURCE CONSENT

LEGEND

- EX BDY
- PROP BDY
- EX MAJOR CONTOUR
- EX MINOR CONTOUR
- PR MAJOR CONTOUR
- PR MINOR CONTOUR
- PROP EXTENT WORK

Rev	Description	By	Date
A	RESOURCE CONSENT	AJP	10/2026

	By	Date
Survey	MAVEN	01/2026
Design	SM	03/2026
Drawn	NDL	03/2026
Checked	DJM	04/2026

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Project:
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Title:
PROPOSED EARTHWORKS PLAN

Project no.	411001
Scale	1:4000 @ A3
Cad file	411001-C200-PR EARTHWORKS.DWG
Drawing no.	C200 Rev A



Legend.

- Site outline
- (1) Clubhouse, lawn bowls, vegetable plots, petanque, exercise equipment, play elements, pergolas, plantings and seating areas - Refer Architecture Package
- (2) Existing Significant Natural Area (WP312) Vegetation - Kahikatea remnant
- (3) Revegetation area including connected pathways (refer to pedestrian connectivity sheet), seating, fitness stations and wayfinding signage. Pathways include various treatments
- (4) Pedestrian connections including lighting and plantings
- (5) Pocket parks including pergolas, picnic tables, raised vegetable plots, play elements and fitness stations
- (6) Main entrance including paving, signage, security gates (vehicle and pedestrian access), 2m high local rock walls, 2m wide vertical timber sleepers with 100mm gaps, and low plantings
- (7) Security gates including paving, security gates (vehicle and pedestrian access), 2m high local rock wall columns, 2m wide vertical timber sleepers with100mm gaps as boundary fencing, and low plantings
- (8) Stormwater devices - Refer Engineering Package
- (9) Existing water way naturalised with planting, log and rock placements
- (10) Existing pond enhanced with plantings, rock and log placements and potential sculpture
- (11) Relaxation moments with seating, natural rock retaining, gravel and natural paving mix, potential lookout structures and plantings
- (12) Pedestrian bridge across water ways
- (13) Care Centre and landscaped grounds to include seating, plantings, vegetable plots, lawn and pergolas - Refer to Architecture Package
- (14) Proposed roundabouts. Potential for sculptural elements
- (15) Traffic calming and pedestrian network alignment in the form of surface treatments including exposed aggregate and 600mm strip of paving setts either side. Located throughout the site.
- (16) "Men's Shed" and glass house - Refer to Architecture Package

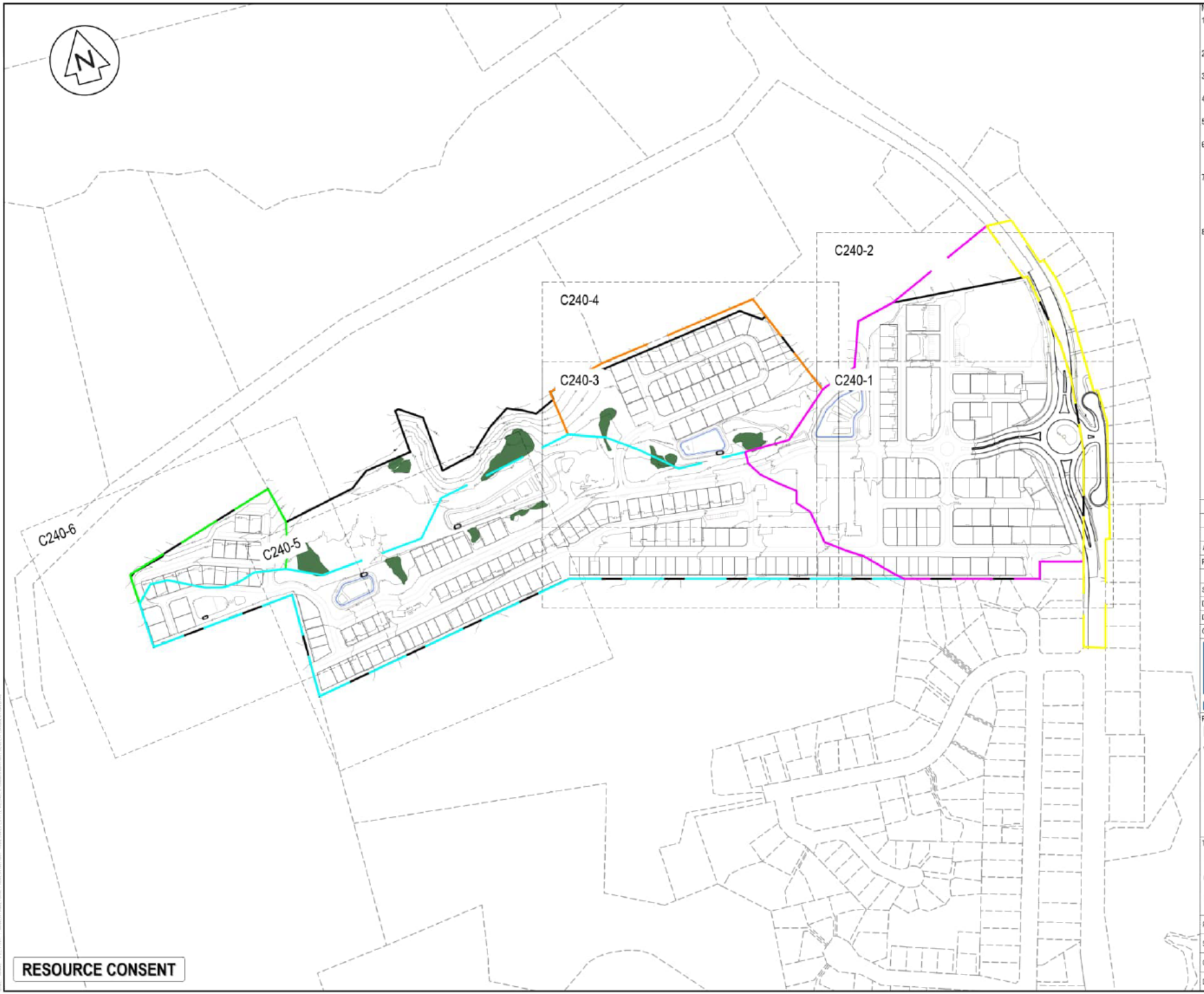
EARL DESIGN
URBAN DESIGN + LANDSCAPE ARCHITECTURE

Notes:
1. *Dwellings' outdoor spaces includes the provision of space for adequate bin storage, letter boxes and clotheslines*
2. *Stage 1A and 1B comprise the publicly accessible portion of the site. The remainder is private and gated*

Date. 21.09.2026
Scale. 1:3000 at A3
Sheet. LA02

Revision. A

Harlow Lifestyle Village
Landscape Architecture Package
Landscape Masterplan



Notes

1. All works to be in accordance with Waipa District Council and Waikato Regional Council Standards including RITS.
2. Co-ordinates in terms of NZ Geodetic Datum Mt Eden 2000.
3. Levels in terms of the New Zealand Vertical Datum 2016.
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6. The contractor shall obtain all necessary approval from utility operators before commencing work under or near their services.
7. Sediment control shall be installed and operational before earthworks start onsite in accordance with council standards, Erosion and Sediment Control: Guidelines for Soil Disturbing Activities TR2009.
8. Contractor shall provide asbuilt of working sediment control devices and confirmation of pond/decent volumes to engineer.

LEGEND

- EX BDY
- PR BDY
- 0 TO 1.5m HIGH RW
- 1.5 TO 2.5m HIGH RW
- 2.5 TO 3.5m HIGH RW
- 3.5 TO 4.5m HIGH RW
- 4.5 TO 5.0m HIGH RW
- 3.2m HIGH RW
- 3.4m HIGH RW
- 0 TO 3.4m HIGH RW
- STAGE 0 - NZTA WORKS
- STAGE - 1 EW EXTENT
- STAGE - 2 EW EXTENT
- STAGE - 3 EW EXTENT
- STAGE - 4 EW EXTENT

Rev	Description	By	Date
A	RESOURCE CONSENT	SM	06/2026

	By	Date
Survey	MAVEN	01/2026
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Drawn	AJP	06/2026
Checked	SM	10/2026

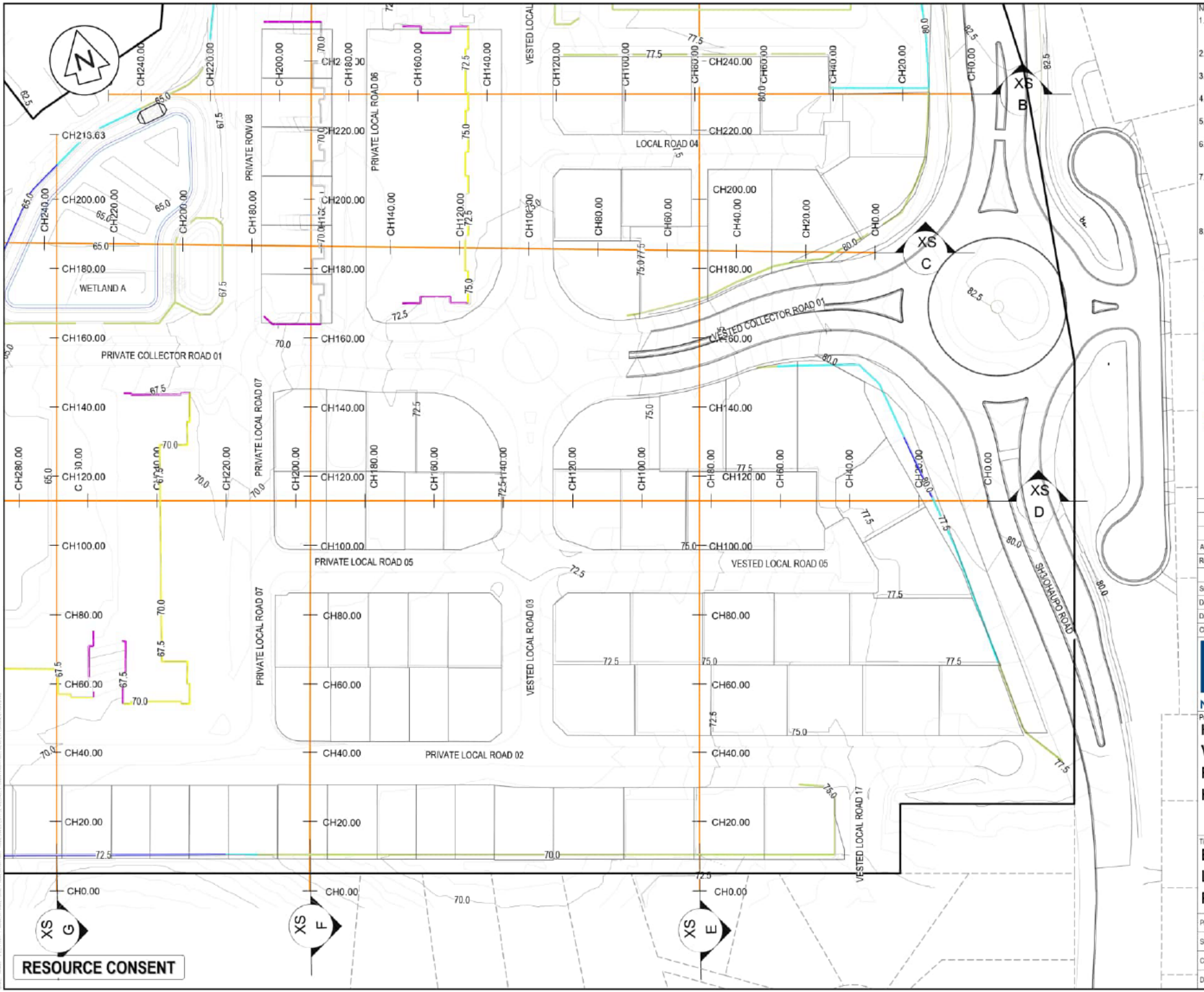
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Project:
HARLOW LIFESTYLE VILLAGE FOR HARRLOW LIFESTYLE LTD

Title
EARTHWORKS OVERVIEW PLAN

Project no.	411001
Scale	1:4000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240 Rev A

RESOURCE CONSENT



Notes

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- EX BDY
- PR BDY
- 0 TO 1.5m HIGH RW
- 1.5 TO 2.5m HIGH RW
- 2.5 TO 3.5m HIGH RW
- 3.5 TO 4.5m HIGH RW
- 4.5 TO 5.0m HIGH RW
- 3.2m ARCHITECTURAL WALL
- 3.4m ARCHITECTURAL WALL
- 0m - 3.4m ARCHITECTURAL WALL

Rev	Description	By	Date
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	By	Date
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Checked	SM	10/2026

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Title:
EARTHWORKS LONGSECTION PLAN 1 OF 6

Project no.	411001
Scale	1:1000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240-1 Rev A



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Drawn	AJP	06/2026
Checked	SM	10/2026

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Project:
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Title
EARTHWORKS LONGSECTION PLAN 3 OF 6

Project no.	411001
Scale	1:1000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240-3
Rev	A

RESOURCE CONSENT



Notes

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- 3.2m ARCHITECTURAL WALL
- 3.4m ARCHITECTURAL WALL
- 0m - 3.4m ARCHITECTURAL WALL

Rev	Description	By	Date
A	RESOURCE CONSENT	SM	06/2026

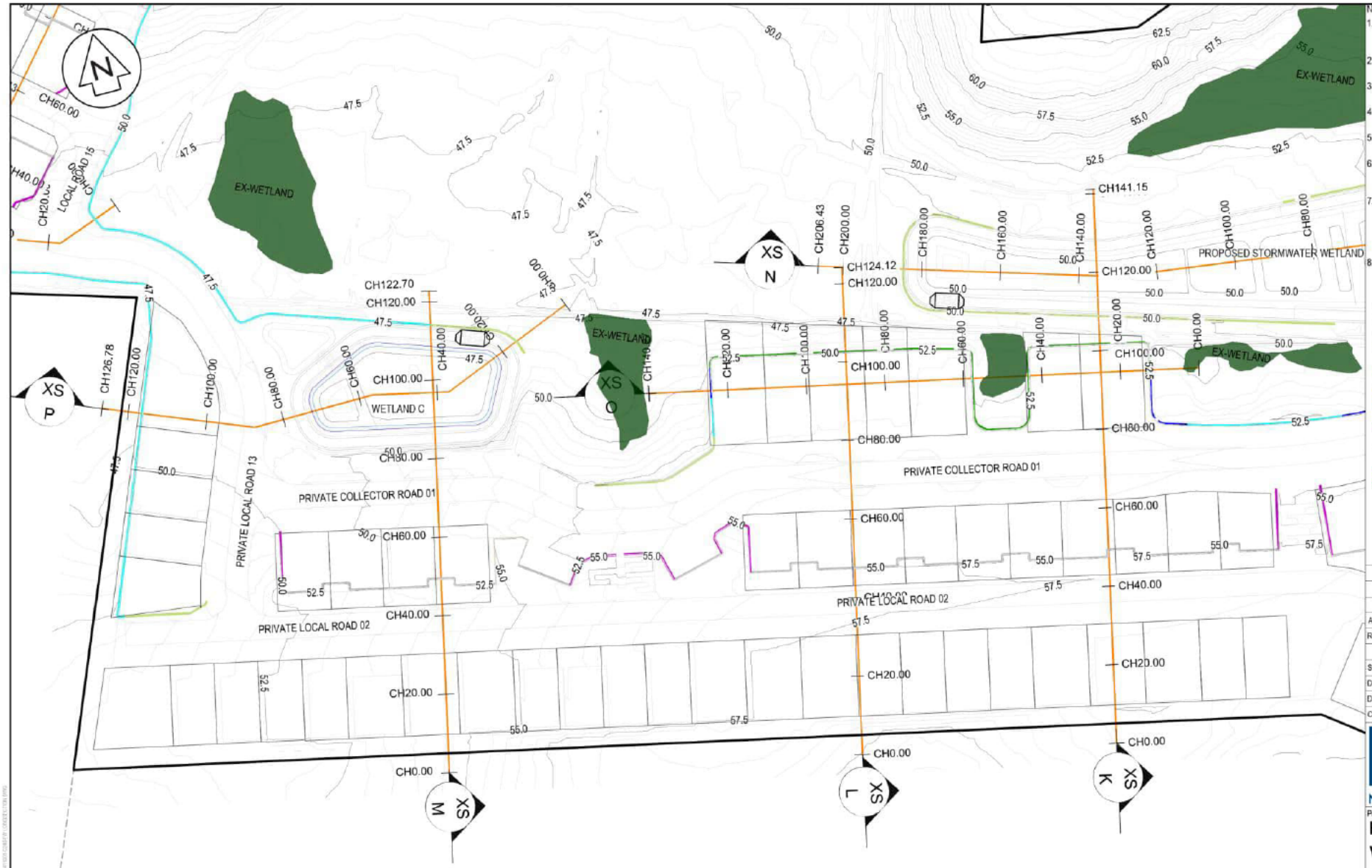
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Survey	MAVEN	01/2026
Design	SM	06/2026
Drawn	AJP	06/2026
Checked	SM	10/2026

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Project:
HARLOW LIFESTYLE VILLAGE FOR HARRLOW LIFESTYLE LTD

Title
EARTHWORKS LONGSECTION PLAN 4 OF 6

Project no.	411001
Scale	1:1000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240-4
Rev	A



Notes

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LEGEND

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- 3.5 TO 4.5m HIGH RW
- 4.5 TO 5.0m HIGH RW
- 3.2m ARCHITECTURAL WALL
- 3.4m ARCHITECTURAL WALL
- 0m - 3.4m ARCHITECTURAL WALL

Rev	Description	By	Date
A	RESOURCE CONSENT	SM	06/2026

	By	Date
Survey	MAVEN	01/2026
Design	SM	06/2026
Drawn	AJP	06/2026
Checked	SM	10/2026

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Project

HARLOW LIFESTYLE VILLAGE FOR HARRLOW LIFESTYLE LTD

Title

EARTHWORKS LONGSECTION PLAN 5 OF 6

Project no.	411001
Scale	1:1000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240-5 Rev A

RESOURCE CONSENT



Notes

1. All works to be in accordance with Waipa District Council and Waikato Regional Council Standards, including RITS.
2. Co-ordinates in terms of NZ Geodetic Datum Mt Eden 2000.
3. Levels in terms of the New Zealand Vertical Datum 2016.
4. It is the contractors responsibility to locate all services that may be affected by his operations.
5. The contractor shall comply with all relevant Health and Safety requirements.
6. The contractor shall obtain all necessary approval from utility operators before commencing work under or near their services.
7. Sediment control shall be installed and operational before earthworks start onsite in accordance with council standards, Erosion and Sediment Control: Guidelines for Soil Disturbing Activities TR2009.
8. Contractor shall provide asbuilt of working sediment control devices and confirmation of pond/decant volumes to engineer.

LEGEND

- EX BDY
- PR BDY
- 0 TO 1.5m HIGH RW
- 1.5 TO 2.5m HIGH RW
- 2.5 TO 3.5m HIGH RW
- 3.5 TO 4.5m HIGH RW
- 4.5 TO 5.0m HIGH RW
- 3.2m ARCHITECTURAL WALL
- 3.4m ARCHITECTURAL WALL
- 0m - 3.4m ARCHITECTURAL WALL

Rev	Description	By	Date
A	RESOURCE CONSENT	SM	06/2026

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Project:
HARLOW LIFESTYLE VILLAGE FOR HARRLOW LIFESTYLE LTD

Title:
EARTHWORKS LONGSECTION PLAN 6 OF 6

Project no.	411001
Scale	1:1000 @A3
Cad file	411001-C240-EW LONGSECTION.DWG
Drawing no.	C240-6
Rev	A

RESOURCE CONSENT