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Project Number 19796.000.002

Geotechnical Investigation – Mt Welcome Development

422 State Highway 59, Pukerua Bay, Porirua

Submitted to:
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Executive Summary

This report presents the findings of a geotechnical assessment of the proposed development site known as Mt Welcome Station, Pukerua Bay. Our assessment included a desktop study, review of existing investigation data and reporting, site walkover, and intrusive ground investigations including machine boreholes and machine test pits observed by an Engineering Geologist experienced in the local geology and geomorphology. Based on our assessment, we consider the site is suitable for the proposed development as we understand it at the time of writing, provided that the recommendations outlined in this report are followed. A brief summary of the key findings and recommendations is provided below:

- Based on our assessment, the site is underlain by surficial soils, including loess, alluvium, colluvium, and residual soil deposits. These typically range in thickness from 1 m – 5 m depending on the local relief and gradient. In some cases, surficial soils extended up to ~16 m below ground level.

- Greywacke rock was observed outcropping at various locations across the site and found throughout most the intrusive investigations.

- Evidence for multiple shallow soil landslides on slopes steeper than ~25° was present across the site and can be observed in historical imagery. Slope stability analysis carried out on existing slopes indicate that failure of the surficial soils, which may continue to the top of highly weathered rock, may occur under raised groundwater, extended periods of rainfall, and seismic events. Mitigation solutions will be required for all natural slopes with a slope angle greater than 25° that have lots or infrastructure above or below them.

- Slope stability assessments of the proposed cut and fill batters indicate that batters may be prone to failure during ULS earthquake events, and that seismic displacement of material may be moderate (<100 mm) to large (~2 m). As such, mitigation solutions will need to be implemented for these batters.

- Large cuts in rock may be prone to rockfall. We therefore recommend that mitigation solutions be implemented to reduce the risk of rockfall causing damage to dwellings located at the base of cut batters at risk of rockfall.

- Localised areas of soft and saturated ground were observed in low lying areas and the base of gullies, these soils are likely to be prone to liquefaction and static settlement. As the proposed development does not include lots in these areas, we consider the liquefaction risk at the site to be low. However, if future development is to occur in these areas, then a detailed liquefaction assessment will be required.

- Most of the surficial soils are suitable for reuse as engineered fill. Rock material will be suitable for reuse as fill, however, consideration will need to be made in regard to the variability of rock weathering and strength characteristics. We recommend additional laboratory compaction testing is carried out during the construction phase as materials change throughout the project lifecycle.

- Site-specific soil strength testing should be carried out on the soil and rock to determine that the soil parameters used within our analyses outlined in this report are appropriate. Should the results differ significantly from those outlined within this report, then slope stability analyses should be reassessed.

- The site has five proposed stormwater retention wetlands. The embankments forming these ponds and the slopes surrounding the ponds will require detailed analysis and design by a suitably qualified and experienced engineer.

- Any cut batters that have loess soils present will require a site-specific erosion design to mitigate against tunnel gullies occurring. If other tunnel gullies are identified which were not identified during our site walkover, then they should either be removed through the bulk earthworks or resolved through specific design.

1 Introduction

ENGEO Ltd was requested by Pukerua Property Group LP (PPG) to undertake a geotechnical investigation of the property at 422 State Highway 59, Pukerua Bay, Porirua, known as Mt Welcome Station (herein referred to as 'the site'). The purpose of the assessment was to support the resource consent application for an approximately 950-lot subdivision development. This work has been carried out in accordance with our signed agreement dated 16 February 2024 and 21 July 2025.

We have previously provided a plan change report titled ENGEO 11/08/2022 - Mt Welcome Station Preliminary Geotechnical Investigation Rev 2. Due to additional investigative information and updated scheme plans, the recommendations in this current report supersedes the 2022 report.

Our scope of works includes:

- Review of published geotechnical and geological information relevant to the site.
- Site assessment by an experienced ground engineering professional.
- Coordination of local buried services location contractor.
- Geotechnical investigation, comprising seven machine drilled boreholes (Griffiths Drilling Ltd and Pro-Drill Ltd) and 36 machine excavated test pits (Keith Bullock Contracting Ltd) with associated Scala probe / shear vane tests across the site to assess the subsurface conditions.
- Preliminary investigation at three of the five proposed dam sites – comprising two machine drilled boreholes and three test pits beneath the proposed permanent stormwater ponds.
- Coordination of laboratory testing of representative soil samples for assessment of suitability for reuse as engineered fill.
- Review of the available testing data from the T+T investigation.
- Analysis of field data and production of a conceptual geological site model.
- Computational slope stability analyses of 11 representative cut and fill batters greater than 8 m in height, to inform the feasibility of the proposed subdivision layout.
- Slope stability assessment and recommendations for an unsuitable stockpile area.
- Production of this geotechnical report based on the findings of our enquiries, ground investigation, and analyses.

2 Site Description

The site occupies an 80 ha section partially zoned urban (Medium Density Residential Zone and Neighbourhood Centre Zone) and partially rural (Rural Lifestyle Zone). The site area comprises properties legally described as:

a. Lot 1 DP 608433, Lot 1000 DP 608433 (34 Muri Road);

b. Lot 1 DP 534864 (422 SH59);

c. Lot 2 DP 534864 (422A SH59);

d. Lot 2 DP 89102 (422B SH59);

e. Part Lot 1 DP 89102 (422A SH59); and

f. Road Reserve (SH59 Corridor).

The site is accessed via a shared driveway directly off State Highway 59. The site location can be seen on the plan in Appendix 3.

The property is currently a working farm which predominantly stocks deer as well as a small number of other animals. There are several small dwellings and sheds in the northwest section of the property. The property is mostly vegetated by grass, with mature trees present in localised areas.

The two smaller properties, DP89102 Lot 1 and Lot 2, also have existing dwellings and are covered predominantly by grass. Lot 1 has a dwelling near its southern boundary, two sheds north of the dwelling, and a pond to the west of the dwelling. A two-storey dwelling occupies the southeast corner of Lot 2 and the area surrounding the dwelling is well vegetated with mature trees and shrubs.

Lot 1 DP534864 contains a house and several farm sheds and a helicopter storage facility, all of which are located centrally.

State Highway 59 runs along the western boundary of the site, with the rail corridor to the west of the highway. The area immediately south and east of the site is occupied by farmland, and the area north of the site by forestry. There is a separate property within the northwest corner of the site with an existing dwelling, the area around the dwelling is designated as QE II Covenant Land and is not a part of the proposed development.

3 Proposed Development

ENGEO has been provided with conceptual earthworks plans including proposed layouts and cut / fill volumes by Envelope Engineering Ltd at the time of preparation of this report (Reference 1753-02, Drawing 2200 – 2250, Rev. P10, dated 30/10/2025). The overall conceptual contour plan is presented in Appendix 1 and the earthworks drawings are provided in Appendix 2.

Based on these plans and our discussions with PPG, we understand that the proposed development will comprise an approximately 950 Lot subdivision. There will be significant infrastructure development including, roading, culverts, wastewater, and stormwater. Our understanding is that stormwater control will include the construction of five permanent stormwater retention wetlands. These retention wetlands do not meet the threshold to be considered ‘classifiable dams’ under the New Zealand Building (Dam Safety) Regulations 2022.

The development will require earthworks including significant volumes of cut and fill, and construction of permanent cut and fill batters, some of which will be greater than 17 m in height. At this stage, the total proposed cut volume for the development is approximately 1,905,000 m^3, and total proposed fill volume is approximately 1,806,000 m^3. The conceptual plans also indicate a permanent unsuitable material stockpile with a proposed volume of approximately 135,000 m^3. There is a possibility that the volume of this stockpile may increase as earthworks progresses.

4 Desktop Study

4.1 Topography

The existing topography and relief vary significantly across the site. Low broad hills and small gullies cover most of the north-western side of the site, topography steepens slightly in the southwest of the site, however, based on the provided cross sections and review of previous reports (described below) slope angles are generally shallower than 26°. A large ridge with approximately 10 m – 15 m vertical relief runs along part of the western boundary of the site. The slope to the west of the ridge has been modified and dips steeply west towards State Highway 59.

The central and eastern side of the site is generally steeper and exhibits greater relief with a number of spurs and deeply incised gullies extend towards the west / northwest. These extend from broad north trending ridgelines that run along the central portion and eastern boundary of the site. The smaller spurs are generally narrow and exhibit steep slopes. Slope angles on this side of the site are commonly between 26° – 45° and can be steeper in localised areas.

In multiple areas across the site natural slope angles exceed 45°; these steeper slopes are generally more prevalent on the eastern side of the site. Overall, west facing slopes are generally longer and shallower than east facing slopes across the site. This is likely due to the prevailing westerly winds which resulted in thicker wind-blown sand and silt cover deposits on the western slopes.

4.2 Geology and Geomorphology

The site is located between two active strike slip faults (Pukerua and Ohariu), both faults have a dextral sense of movement.

The geology of the Wellington region is mapped at 1:250,000 scale on GNS Science's online webmap (https://data.gns.cri.nz/geology/). The mapping indicates that the northwest section of the site is underlain by Pleistocene fan, river, and lake deposits consisting of well graded loess-covered gravels, alluvial gravel, and lacustrine silt deposits. The mapping indicates that the eastern and south-western sections of the site are underlain by Triassic sandstone and mudstone of the Rakaia Terrane (referred to locally as Wellington Greywacke) as shown in Figure 1.

Although not shown on the geological map in Figure 1, the Greywacke sandstone and mudstone in this area is generally overlain with cover deposits of loess and colluvium on ridges, slopes, and plateaus, with recent alluvial deposits in the base of gullies and valleys. The cover deposits often vary in thickness up to several metres thick. In some locations, Greywacke is exposed on the ground surface. This is generally confined to small, localised areas (typically in areas exceeding slope angles of 40 degrees) where steep topography does not allow the accumulation of surficial soils.

Figure 1: Geological Setting

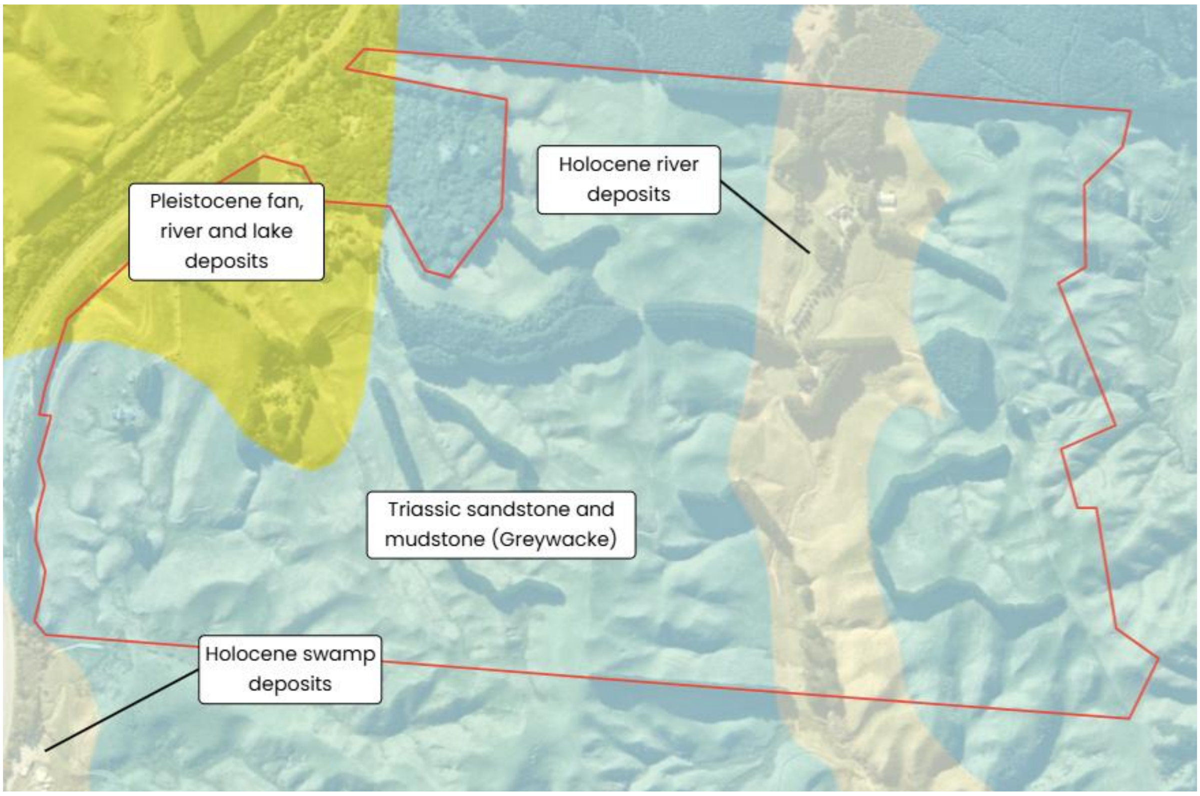


Photo 1: Geological map sourced from GNS Science website, basemap image sourced from Nearmap.

4.3 Seismicity

The GNS New Zealand Fault Database website indicates that the site is located within 20 km of the onshore trace of several active faults.

The closest active fault to the site is the Pukerua Fault, located approximately 400 m – 500 m northwest of the site. Other active faults within 20 km of the site include the Ohariu Fault (2.5 km), Moonshine Fault and Ōtaki Fault (8.5 km), Akatarawa Fault (13.5 km), Wellington Fault (13.7 km), and the Whiteman's Valley Fault (17 km); these faults are all located to the east and southeast of the site. The offshore trace of the Wairau Fault lies approximately 20 km to the northwest of the site.

Of the active faults within 20 km, two are listed amongst the major faults in Table 3.6 of NZS1170.5, as follows:

- Wellington Fault – approximately 17.7 km from the site.
- Wairau Fault – approximately 20 km from the site.

Near fault factors, as per NZS1170.5, will therefore need to be considered by a Structural Engineer.

4.4 Regional Seismic Hazard

Greater Wellington Regional Council hazard maps for Porirua only cover a small section along the northern boundary of the entire site, the parts of the site that are mapped indicate a combined earthquake hazard rating of Low-Moderate to Moderate. This part of the site is mapped as having a Low to Low-Moderate ground shaking potential. The site is mapped as having a Slope Failure hazard of Low to Low-Moderate. The maps do not indicate the potential liquefaction hazard at the site.

It should be noted that these maps are regional in nature and the hazard potential indicated on the maps does not necessarily apply to any specific site.

4.5 Historical Aerial Photography Review

We have reviewed 16 historical aerial photographs of the site available through Retrolens, Porirua City Council online maps, Google Earth, and Nearmap. These photographs were viewed to assist with understanding the geotechnical history of the site, including placement of fill and changes in landform. Our key observations are described below and a selection of photographs provided in Figure 2:

- 1942 – The earliest available photograph of the site shows the site appears to be farmland, mostly cleared of vegetation, and covered by grass. Stream channels are shown throughout the site. Landslide scarps can be seen on slopes in the eastern section of the site. There is a structure near the location of the current woolshed.

- 1962 – A second structure can be seen near the location of the existing farmhouse at the eastern side of the site. A small section of vegetation is present just south of the woolshed.

- 1966 – Construction of a farm track is visible which runs from the old State Highway 1 through to the eastern edge of the site and beyond.

- 1979 – Several smaller structures have been constructed near the woolshed. Several smaller farm tracks have been constructed in the eastern parts of the site, two of these appear to cross the main stream that runs through the site.

- 1986 – The trees south of the woolshed appear to have been cleared and a second farmhouse has been constructed in this area.

- 2000 – A farm track has been cut along the eastern ridge at the edge of the site.

- 2005 – The dwelling at Lot 2 DP89102 has been constructed and the surrounding area has been planted. A driveway has been made to this dwelling as well as a second driveway to the existing house site on Lot 1 DP89102. The main driveway has been planted with trees, the existing pines at the entrance to the site and along the driveway have been planted, and the wind breaks along several of the ridges have been planted. There is evidence of shallow seated soil failures on several slopes on the eastern side of the site and in multiple gullies on the western side of the site.

- 2009 – The main dwelling at Lot 1 DP89102 has been constructed as well as one of the small sheds northwest of the dwelling.

- 2017 – The second shed at Lot 1 DP89102 has been constructed.

Figure 2: Aerial Photographs

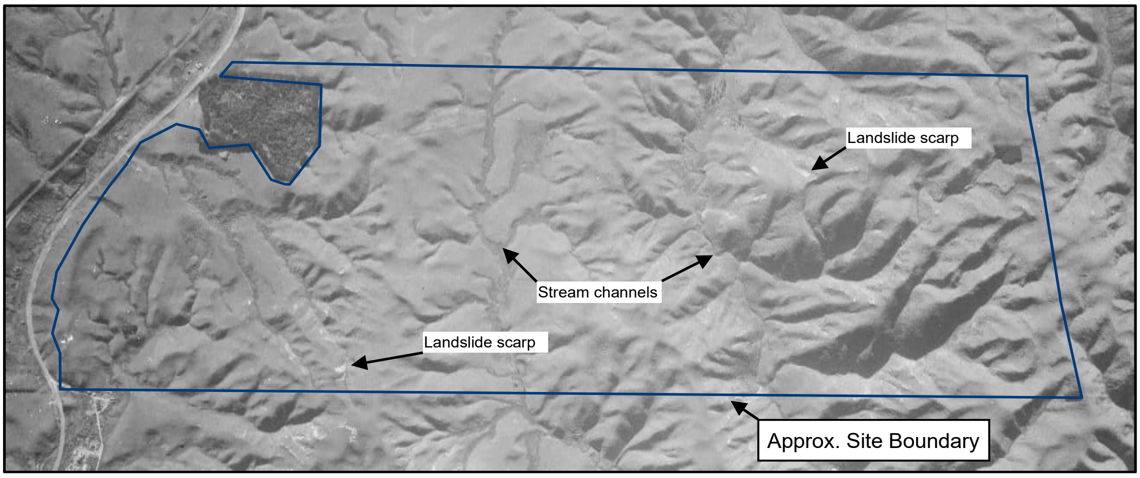


Photo 1: 1942 Imagery: This imagery is the oldest available on Retrolens that covers the site. The imagery shows the site with very little vegetation cover. A number of stream channels are visible through the valleys with smaller tributaries feeding into the streams. Landslide scarps can be identified on slopes to the east as well as on slopes that fall toward gullies and water courses.

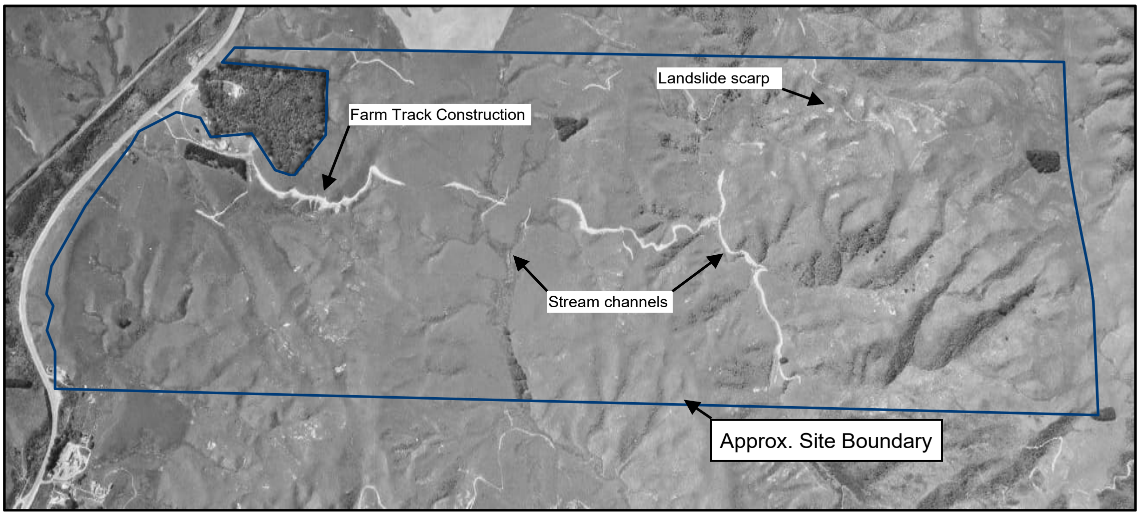


Photo 2: 1966 Imagery: This imagery shows minor growth of vegetation in some areas and the construction of a farm track is visible. New landslide scarps that weren't visible in the 1942 imagery can be identified. The scarps appear to be similar size to the garage buildings present in the western area of the site.

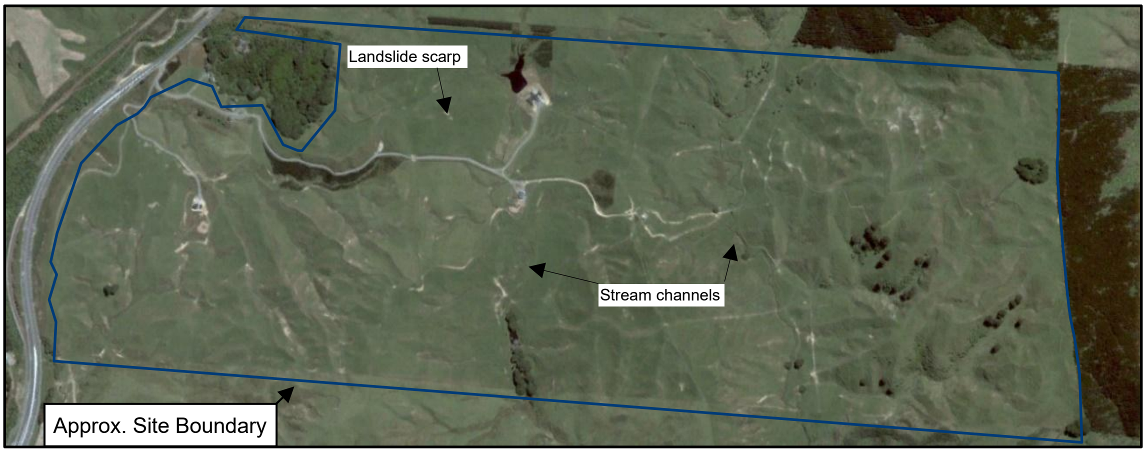


Photo 3: 2002 Imagery: The above imagery from Google Earth shows the continued growth of vegetation in some areas and the plantation of new vegetation. A new building along with a water reservoir has been constructed near the northern site boundary. New landslide scarps can be identified across the site.

4.6 Flood Hazard

The Porirua City Council hazard maps indicate several flood hazards are present at the site. The main valley / gully features of the site are mapped as Stream Corridors which may be prone to flooding following heavy or prolonged rain events. Several low-lying areas along the western section of the main gully through the site are mapped as potential ponding areas which may become inundated with water following flooding events.

4.7 Previous Geotechnical Reports

4.7.1 2022 ENGEO Preliminary Geotechnical Investigation

ENGEO completed a preliminary geotechnical investigation for the proposed development area in 2022 to assess potential geohazards present at the site and to support a plan change for the site. This investigation included preliminary geomorphic mapping across the site. The key findings from ENGEO's 2022 investigation are provided below:

- The site is underlain by a variety of surficial soils that typically range in thickness from 1 to 3 m depending on the local relief and gradient of the land.
- Greywacke rock was observed outcropping at various locations across the site, primarily on steep slopes where thinner soil mantles are present.
- Evidence for multiple shallow soil landslides on slopes steeper than 26 degrees is present across the site and was observed in historical imagery. Water appears to be the primary driver of failure for these slopes.
- There was no evidence to suggest the site has a history of significant, deep seated, rock mass failures.
- Localised areas of soft and saturated ground are anticipated in low lying areas and the base of gullies.
- Most of the surficial soils are considered likely to be suitable for reuse as engineered fill following lab testing and compaction trials. Rock material will be suitable for reuse as fill but consideration will need to be made in regards to the variability of rock weathering and strength characteristics.
- The mitigation and management of natural hazards is considered to be feasible from a geotechnical point of view.
- Future intrusive testing should be completed as part of the next design stage for the project once site layouts are finalised. Testing will be used to verify the ground model and as part of natural hazard mitigation methods.

  Slopes steeper than 15 degrees commonly exhibit extensive soil creep in the form of terracettes, small step like features that run horizontally along slopes as a result of surface soil creep. They are often caused by devegetation of steep slopes and can be exacerbated by livestock. These features may increase the risk of shallow soil instabilities by affecting overland flow of water and causing ponding.

4.7.2 2019 Tonkin + Taylor Preliminary Geotechnical Investigation

PPG have provided ENGEO with a preliminary geotechnical investigation and natural hazard assessment report prepared by Tonkin + Taylor Ltd (T+T) for the portion of the site legally described as Lot 1 DP89102. As part of this investigation, 25 test pits and nine Scala penetrometer tests were completed across the site. This work was carried out in May 2019.

Results from the T+T investigation indicated that the site is overlain by a mixture of dune sand, loess, colluvium, and alluvium. Rock was encountered in all but six of the test pits, the rock was described as predominantly siltstone with some sandstone and mudstone, the degree of weathering of the rock varied from completely to moderately weathered.

4.8 Previous Laboratory Testing

As part of the 2019 preliminary completed by T+T, several soil samples were collected from the test pit investigations for laboratory testing. The laboratory testing included four Particle Size Distribution (PSD) tests, and three Atterberg and moisture content tests. The tests were undertaken in accordance with NZS 4402. A summary of the laboratory test results is presented in Table 1.

Table 1: Previous Geotechnical Laboratory Testing Results Summary

Location ID	Sample Depth (m)	Moisture Content (%)	Atterberg Limits			Particle Size Distribution (%)		
			LL	PL	PI	Gravel	Sand	Fines
TT_TP01	1.0	N/A	N/A	N/A	N/A	0	62	38
TT_TP13	3.0	N/A	N/A	N/A	N/A	0	58	42
TT_TP14	0.75	23	30	16	14	N/A	N/A	N/A
TT_TP15	1.0	22	28	20	8	N/A	N/A	N/A
TT_TP15	3.0	N/A	N/A	N/A	N/A	17	47	36
TT_TP18	1.0	N/A	N/A	N/A	N/A	0	83	17
TT_TP22	0.8	20	38	20	18	N/A	N/A	N/A

LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index.

5 Site Investigation

5.1 Site Observations

Overall, the geological and geomorphological observations made during our site visits corroborate well with those outlined in Section 4. The western area of the site is comprised of low, broad hills with mostly moderate gradients (< 26°). Topography is steeper and relief larger in the eastern sections of the site, with slopes commonly exceeding 26°. Wetlands and soft ground, sometimes with running streams, were observed within the gullies and valleys across the site. The slopes consist primarily of loess / colluvium deposits overlying Greywacke sandstone. The loess deposits range from a thin veneer to greater than 5 m deep. The loess tends to be thicker on the western and northern facing valley sides, which is likely due to the aeolian loess deposits on the leeward side from the prevailing wind direction. Within the loess deposits, landslides, soil creep, and tunnel gullies are prevalent.

Detailed geological information, including our conceptual ground model is presented in Section 6.

5.2 Investigations Completed

ENGEO attended site between 18 and 30 April 2024, and 10 to 15 September 2025 to complete the following intrusive testing:

- Nine machine boreholes, named BH01 through BH09, to depths of up to 19.65 m.
- Eighteen test pits with associated Scala penetrometer tests in the 18 – 30 April Investigations, named TP01 through TP19 (excluding TP16), to depths of up to 5.1 m depth
- Seventeen test pit investigations with associated Scala penetrometer tests in the 10 to 15 September investigations, named TP20 to TP41.
- Test pits TP16, 27, 30, 31, 33, and 34 could not be completed as the excavator could not practically reach the proposed test locations. In the case of TPs 27, 30, 31, 33, and 34, Scala penetrometer tests were undertaken in their place. We do not consider that these investigations will be required for resource consent.

Previous investigations completed and the development plans informed the number, type, and location of testing. A geotechnical investigation location plan is presented in Appendix 3. Machine borehole and test pit logs are presented in Appendix 4 and 5 respectively.

All test pits were backfilled after logging of the recovered material was completed. Piezometers were installed in four of the nine boreholes. These boreholes were left with PVC standpipes protruding from the ground to enable ongoing groundwater monitoring. All boreholes were backfilled with gravel and sealed with bentonite.

6 Engineering Geological Model

6.1 Model Development

Based on findings from the desktop study, as well as ENGEO's 2022 and Tonkin + Taylor's 2019 geotechnical investigations, a conceptual ground model was developed, which suggested that loess or dune deposits cover the majority of the slopes, ridges, and plateaus across the site, and that alluvium and lacustrine deposits are likely present in the gullies and low-lying areas of the site. The conceptual ground model suggested that these surficial deposits likely overlie Greywacke basement rock with varying degrees of weathering, and that these deposits are likely thicker on the more gently sloping hills and gullies on the western side of the site, possibly several metres thick.

This model was largely confirmed during the investigations which indicated thick deposits of loess, alluvium, dune sand, and lacustrine deposits were present across the western side of the site. Investigations from the eastern side of the site indicated generally thinner surficial deposits, predominantly loess and colluvium, overlying Greywacke bedrock that showed a variable weathering profile.

Due to difficulties distinguishing loess from loess derived colluvium in the field and owing to the two soil types displaying generally similar field strength characteristics, we have modelled the loess and colluvium as a single unit during slope stability analysis.

6.2 Generalised Geological Profile

Test pits and machine boreholes indicate that surface soils vary in composition across the site. The soil below a thin layer of topsoil is generally comprised of sand and silt dominant loess and colluvium which varies in thickness across the site from 0.8 m to greater than 3.8 m. These cover deposits were generally thicker on the west facing slopes of the site. Fill was encountered in TP39 to 0.8 m depth and is considered to be isolated in that area.

Investigations located in gully or valley features commonly encountered lacustrine or alluvial deposits comprised of sand, silt, and gravel, below the loess / colluvium (where present) or directly below topsoil. Lacustrine / alluvial deposits varied in thickness between 0.3 m – 6 m. One location (BH06) indicated lacustrine / alluvial deposits continued to 16.5 m depth. Generally, these deposits were thicker in locations which drained with / from larger catchments.

Greywacke sandstone and mudstone of the Rakaia Terrane underlies the surficial soil deposits. Greywacke was encountered in all nine machine boreholes and in 24 of the 41 test pits. Greywacke varied in degree of weathering and strength across the site.

Generally, the Greywacke exhibited a shallow weathering profile, transitioning from completely weathered to moderately weathered in 0.5 m – 3 m. A thicker weathering profile was observed in BH04, which encountered ~10 m of completely weathered Greywacke. In BH08, the Greywacke alternated between highly weathered and completely weathered several times, showing a varied and non-linear weathering profile.

Table 2 provides a generalised geological profile.

Table 2: Generalised Geological Profile

Unit	Description	General Occurrence	Depth Range (m bgl)	Thickness (m)	Material Strength
Topsoil	SILT / SAND, dark brown, moist	Site-wide	0 – 0.6	0.1 – 0.6	Very Loose – Loose, Soft – Firm
Fill	SAND / SILT, with gravel, brown, well graded / LP	Developed locations, including farm tracks and adjacent to culverts	0 – 0.8	0.2 – 0.8	N/A
Loess	SAND / SILT, brownish orange to grey, poorly to uniformly grade / LP	Commonly site-wide, prevalent on western slopes	0.15 – 1.8	1.0 – 1.5	Very Loose – Dense, Stiff – Very Stiff
Loess / Colluvium	SAND / SILT, with gravel, brownish orange to grey, poorly graded / LP	Commonly site-wide on moderate slopes and near slope toes	0.2 – 4.5	1.2 – 4.3	Loose – Dense, Stiff
Lacustrine / Alluvial Deposits	SAND / SILT, with gravel and organics, grey / bluish grey to brown, poorly-well graded / LP	Generally low-lying gully areas	0.2 – 16.5	0.6 – 15.8*	Very Loose – Very Dense, Soft – Very Stiff

Unit	Description		General Occurrence	Depth Range (m bgl)	Thickness (m)	Material Strength
Greywacke	CW	SAND / SILT, with gravel, brownish orange/grey	Commonly found below surficial soils but not in all locations	0.4 – 17.2	0.4 – 10.3**	Medium Dense – Very Dense, Firm – Very Stiff
	HW	SAND / SILT or highly fractured rock	Generally found above MW rock	0.4 – 15.1	0.1 – 2.0	N/A
	MW	Weak to moderately strong, grey / brown / orange, highly fractured	Assumed to underly entire site	1.8	Unknown	N/A
	SW	Strong to very strong, light blue / grey	BH05 and BH08 only	3.1 – 8+ (BH05), 19 m+ (BH08)	Unknown	N/A

LP = Low plasticity. CW = Completely Weathered, HW = Highly Weathered, MW = Moderately Weathered.

*Observed in BH06 only. **Observed in BH04 only.

6.3 Laboratory Testing

ENGEO collected several soil samples during the 18 to 30 April 2024 test pit investigations for laboratory testing. The laboratory testing was carried out by Materials Advisory & Testing Service Ltd (MATS) and included four Particle Size Distribution (PSD) and seven Standard Compaction tests. Laboratory testing was undertaken in accordance with NZS 4407 and NZS 4402 recommendations.

Four samples were taken and used as representative samples for four common soil and rock types encountered on-site, these were loess, poorly graded sand, completely weathered Greywacke, and moderately weathered Greywacke. These samples underwent both PSD and Standard Compaction testing. Additionally, three samples comprised of mixed ratios of loess and highly / moderately weathered Greywacke underwent Standard Compaction testing. These samples were manually mixed to ratios of 75:25, 50:50, 25:75, loess to highly / moderately weathered Greywacke.

A summary of the laboratory test results is presented in Table 3.

Table 3: Summary of 2024 Laboratory Test Results

Material Type	Location ID	Sample Depth (m)	Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)	Particle Size Distribution (%)		
					Gravel	Sand	Fines
PG Sand	TP05	2.5 – 4.0	1.79	14.0	0	83	17
Loess	TP07	1.3 – 1.5	1.67	19.5	0	70	30
CW Greywacke	TP11	3.0 – 4.0	1.77	16.5	0	75	25
MW Greywacke	TP14	2.4 – 2.8	2.07	10.9	83*	11	6
75:25 Loess: MW Greywacke	TP07 & TP14	1.3 – 1.5 & 2.4 – 2.8	1.89	14.8	N/A		
50:50 Loess: MW Greywacke	TP19 & TP12	0.5 – 1.0 & 0.25 – 0.5	1.80	17.5	N/A		
25:75 Loess: HW / MW Greywacke	TP18	0.5 – 1.5 & 3.5 – 4.5	1.75	18.5	N/A		

PG = Poorly Graded, CW = Completely Weathered, MW = Moderately Weathered, HW = Highly Weathered.

*Approximately 5% of material was coarser than gravel (>60 mm).

Samples were taken for laboratory testing in the September 2025 investigation. These were taken in the areas of the proposed retention structures. A summary of samples taken is provided in Table 4. At the time of writing this report, no results have been received from the laboratory.

Table 4: Summary of 2025 Laboratory Test Requests

Test Pit No.	Depth	NZS4402 Test 2.2, 2.3 & 2.4 - LL, PL & PI	NZS4402 Test 2.8.1 - PSD	NZS4402 Test 2.8.4 - Hydrometer	NZS4402 Test 4.1.1 - Maximum dry density	ISO 17892 Part 11 - Perm (Triaxial Pressure Head	ASTM D4647-13 / BS1377 Part 5 Test 6.2	NZS4402 Test 7.1 - One-dimensional Consolidation
TP20	0.9 – 2.3		1		1	1		
TP21b	0.8 – 2.3		1					
TP37	0.4 – 2	1		1	1	1	1	1
TP38	>2 m	1		1	1		1	
TP39	2.4 – 4	1		1		1	1	1
TP39	1.8 – 2.4							1
TP40	0.7 – 3.2		1					

6.4 Instrumentation and Monitoring

Piezometers were installed in four of the nine boreholes (BH01, BH02, BH05, and BH06), and labelled PZ01, PZ02, PZ05, PZ06 correlated to the relative borehole.

6.5 Groundwater

Groundwater data was captured from the four piezometers installed in boreholes BH01, BH02, BH05, and BH06. The groundwater level in the piezometers was measured four times between 6 May and 30 May 2024. A summary of the groundwater conditions is presented in Table 5.

Table 5: Groundwater Measurements May 2024

Piezometer ID (elevation*)	Minimum Groundwater Level (bgl)	Maximum Groundwater Level (bgl)	Maximum Variation
PZ01 (67 m)	0.41 m	1.05 m	0.64 m
PZ02 (133 m)	3.64 m	3.85 m	0.21 m
PZ05 (42 m)	0.62 m	0.65 m	0.03 m
PZ06 (54 m)	1.8 m	1.9 m	0.1 m

*Elevation is relative to Wellington mean sea level.

7 Geotechnical Assessment

The following section identifies and assesses the natural geohazards present on the site. Assessing the following geotechnical factors will help determine site suitability and recommend mitigation measures.

- Establishment of appropriate geotechnical material properties
- Seismic hazard including liquefaction and lateral spreading / cyclic softening
- Slope stability (existing slopes and proposed cut / fill batters)
- Unsuitable soil embankment - structural and non-structural fill stability
- Debris flow
- Rockfall
- Consolidation settlement
- Tunnel gullies
- Collapsible soils
- Earthworks, landform modification, historical fills
- Erosion and sediment control

Each of these aspects is discussed in the following sections.

7.1 Geotechnical Material Properties

The following parameters for our analysis are based on our current understanding of the engineering geological model (Section 6) and should be re-evaluated during subsequent stages of design and / or when further geotechnical data becomes available.

ENGEO has selected the soil and rock properties for our ground model based on the observations and *in situ* strength testing completed in conjunction with our site walkover and subsurface investigations, our knowledge of similar materials from nearby sites, a review of triaxial testing of similar structural fills from the wider area, and a review of Greywacke rock strength parameters from literature and triaxial testing in similar material. The parameters used in our ground model and analysis are presented in Table 6 and Table 7.

Table 6: Assumed Engineering Properties of Slope Material (Mohr Coulomb)

Material	Unit Weight (kN/m^3)	Strength Type	Cohesion	Friction Angle (degrees)
Fill – Structural	20	Mohr-Coulomb	10*	34*
Fill – Structural (saturated)	20	Mohr-Coulomb	1	34
Loess / Colluvium	18	Mohr-Coulomb	1	32
Alluvium	18	Mohr-Coulomb	0	32
CW Greywacke	20	Mohr-Coulomb	5	35
HW Greywacke	22	Mohr-Coulomb	30	38

CW = Completely Weathered, HW = Highly Weathered.

*Based on strength parameters ENGEO obtained from triaxial testing on similar material.

The strength parameters used for structural fill in our analysis are based on previously completed triaxial testing carried out on highly to moderately weathered Wellington Greywacke. Site specific triaxial testing should be carried out to confirm these parameters.

Table 7: Assumed Engineering Properties of Slope Material (Generalised Hoek Brown)

Material	Strength Parameters			
	USC (kPa)	GSI	mi	Disturbance Factor
MW Greywacke	20,000	45	11	0

MW = Moderately Weathered.

7.2 Seismic Hazard

Potential seismic hazards resulting from nearby moderate to major earthquakes can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, and landslides.

The following sections present a discussion of seismic hazards as they apply to the site.

7.2.1 Ground Rupture

As cited in Section 4.3, the site is located approximately 400 m – 500 m from the nearest mapped active fault. At this distance, fault related ground rupture and ground lurching are considered unlikely.

However, given the tectonic setting, regional subsidence may be experienced following a large earthquake on faults in the area. In seismically active areas there is always a risk that unmapped active faults may be present, particularly when the faults may have a low recurrence interval and may be buried under younger alluvial sediments.

7.2.2 Ground Shaking and Design Ground Acceleration

Ground shaking and subsequent effects on structures, infrastructure, and engineering systems can be extensive and affect large areas. The intensity, frequency, and duration of ground shaking drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

In geotechnical assessments, amplitude, frequency, and duration of shaking are the main factors considered.

Through discussions with the client, we understand the potential site development includes Importance Level 2 (IL2) structures. Peak horizontal ground accelerations (a_{max}) for use in geotechnical assessments are provided in Table 8. A_{max} values have been taken from the recommended values provided in Table A1 - Appendix A of MBIE/NZGS Module 1 for Wellington.

Table 8: Peak Horizontal Ground Acceleration

Limit State	Return Period	a_{max}	Magnitude
SLS	25 years	0.13 g	6.5
ULS	500 years	0.68 g	7.7

7.2.3 Liquefaction and Lateral Spreading

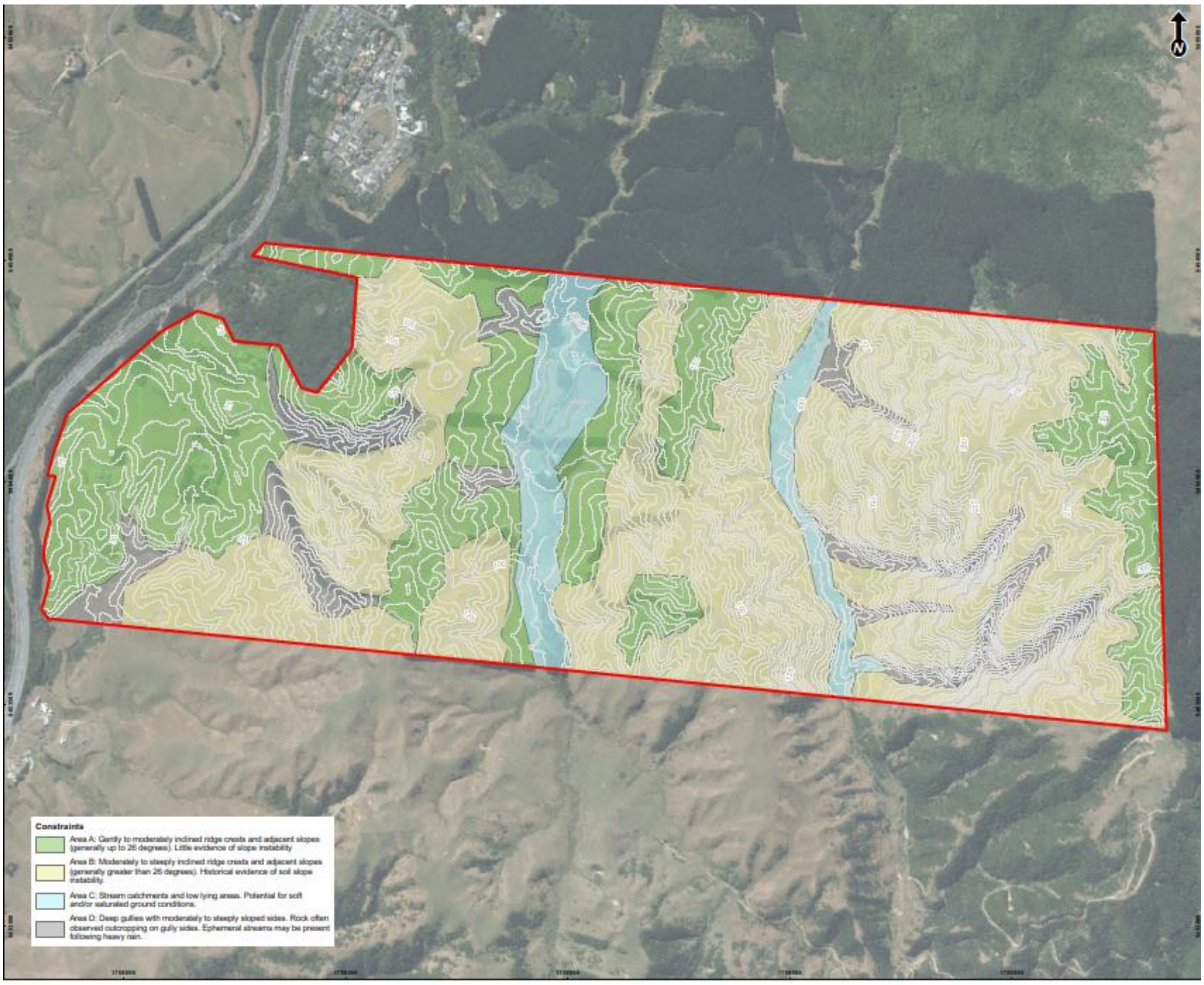
Some areas of the site contain soils comprised of loose sands and low-plasticity silts below the water table and are considered likely to be prone to liquefaction.

Our investigations indicate that the areas of the site that generally contain these potentially liquefiable soils are confined to the low-lying gullies and valleys. These areas generally show a thicker soil profile and have a higher water table. Loss of soil strength due to liquefaction should be considered for any development that may occur in these areas, this includes civil and roading, as well as the construction of earth embankments for the permanent stormwater retention wetlands.

The concept design drawings indicate that, where development is to occur in these areas, the building platforms will be formed by engineered fill placement. Loose / soft and saturated material will be removed prior to fill placement, subsoil drainage installed, and the fill is likely to be greater than 2 m thick, creating a non-liquefiable crust. As such, we do not believe the surficial soils beneath these proposed new lots will be susceptible to liquefaction.

The areas of the site that have been proposed for development for house lots generally do not sit in these potentially liquefiable areas and will be significantly altered from their current state, either through excavation or placement of engineered fill. Therefore, we do not consider liquefaction to be a significant risk to the development of the site, as is currently planned. If development were to extend into these low-lying areas in future, then detailed liquefaction and lateral spread investigation and analysis should be undertaken. The areas are shown highlighted blue on the map in Figure 3.

Figure 3: Geotechnical Constraints Map



7.3 Slope Instability

From our desktop study and site observations, we consider that slope instability is likely to be a hazard at the site due to the presence of steep slopes and the evidence of previous slope instability. Many of the slopes will be significantly altered during the earthworks phase of construction. As such, we have completed preliminary slope stability analyses for a selection of existing slopes and proposed cut and fill batters to inform our recommendations. Analysis methodology and results are discussed in the following sections.

7.3.1 Previous Slope Instability

Our slope stability assessment is based on our review of the previous geotechnical reports, observations made on-site, and review of historical imagery.

Evidence of multiple shallow landslides was observed across the site. Typically slopes steeper than 26° comprised of loess, colluvium, and residual soil / completely weathered rock are susceptible to shallow translational and rotational slides. This was particularly prevalent near gullies where the toe of slopes can become saturated following heavy rain. Some of these landslides continue to the base of the slope as saturated debris flows. It is likely that heavy rain events are the primary driver for shallow soil instabilities. The size of the landslides observed ranged from 1 – 2 m^3 to ~500 m^3 and the scarps were generally less than 3 m deep.

7.3.2 Analysis of Natural / Existing Slopes

We have undertaken an assessment of the slope stability of cross sections representative of locations where proposed new lots will be above or below existing slopes that may be susceptible to instability and cut fill batters greater than 8 m in height. Our cross sections are taken from Envelope Engineering drawings Ltd (Reference 1753-02, dated 25.09.2025, Drawings 800 – 807, Rev. P1). Sections and location plan are presented in Appendix 6.

For Sections F, G, and H, we have reduced the proposed cut batter angle in the overlying surficial soils at the top of the cut from that proposed in the Envelope Engineering Ltd drawings. The changes to the cut batter angles were made to achieve the required Factor of Safety for the slopes, and in all such cases, the alterations from design drawings have been indicated on the sections. The cut batter angles provided in the preliminary design are acceptable as average slope angles at this stage in the development, and it is usual for alterations in the cut batter angles to be made during construction based on the exposed soil / rock condition.

We have separated the analysed slopes into two groups, the first covers the western area of the site and includes Sections C, D, F, and G, the second covers the slopes in the central and eastern part of the site and includes Sections A, B, E, H, N, O, and R. Due to the topography of the site, slopes in the central and eastern section have had a topographic amplification factor applied to all seismic case stability analyses as detailed in Section 7.3.6 and in-line with the NZGS & MBIE Module 6.

We have also carried out stability analyses for the unsuitable soil embankment in the eastern section of the site as detailed in Section 7.4. The unsuitable soil embankment is an engineered buttress which is designed to contain the material retrieved from site which is not suitable to use as engineered fill.

7.3.3 Analysis Methods

As part of our assessment of the suitability of the proposed works, we have undertaken slope stability analysis using the Slide computer modelling software.

In slope stability analysis, the stability of a slope is expressed as the 'Factor of Safety' (FoS) which is the ratio of the forces resisting slope movement to the forces causing slope movement. Theoretical failure of a slope is possible when the FoS is less than 1.0, while increasing values above 1.0 indicate improving stability.

On several sections, we have modelled an additional surcharge of 10 kN/m^2. This is to model the likely load of a residential dwelling and the effect this will have on the slope below.

7.3.4 Groundwater

Groundwater was modelled as a piezometric surface under long term drained conditions. Under short term transient raised groundwater conditions, the groundwater level was raised. Additionally, the upper 1.5 m – 2 m of fill material was modelled as saturated with reduced internal cohesion to replicate surface saturation following heavy or prolonged rainfall.

7.3.5 Strength Properties

Soil properties are based on the observations and strength testing completed in conjunction with our site walkover and subsurface investigations, as well as previous experience. The parameters used in our analysis are presented in Table 9 and Table 10.

7.3.6 Design Seismic Conditions

We have used the seismic ground parameters shown in Table 9.

Table 9: Seismic Parameters

Parameter		Seismic Scenario	
		Serviceability Limit State (SLS)	Ultimate Limit State (ULS)
Design Life		50 years	50 years
Importance Level		2	2
Annual probability of exceedance for design earthquake		1/25	1/500
Peak Ground Acceleration (PGA)		0.13g	0.68g
Topographical amplification factor (a_{topo})	*Western Site	1.0	1.0
	*Central and Eastern Site	1.1	1.1
Wall displacement factor (w_d)		1.0	0.5
Design Ground Acceleration	*Western Site	0.13g	0.34g
	*Central and Eastern Site	0.14g	0.37g

*Western Site includes Sections C, D, F, and G. Central and Eastern Site includes Sections A, B, E, H, N, O, and R, and the 'Unsuitable Soil Embankment'.

A wall displacement factor has been applied to account for the soil which can tolerate movement without compromising structural integrity.

7.3.7 Analysis Results

We have assessed the overall slope stability factor of safety under static conditions, raised groundwater conditions, a 1/25-year SLS earthquake event, and a 1/500-year ULS earthquake event as shown in Table 10.

Table 10: Acceptable Factors of Safety

Case	Required FOS
Static	1.5
Seismic (1/25-year event)	1.2
Seismic (1/500-year event)	1.2*
Raised Groundwater	1.2

*Factors of safety less than 1.2 may be acceptable subject to assessment of displacement for this case.

Cut / Fill Batters

Table 11 summarises the results of the slope stability analyses for the proposed cut and fill batters with full slope stability analyses presented in Appendix 7. The red numbers indicate a FOS below the recommended factor of safety.

Table 11: Slope Stability Analysis Summary (Cut / Fill Batters)

Section	Static FOS (1.5)	SLS Event FOS (1.3)	ULS Event FOS (1.2)	Raised Groundwater FOS (1.2)
A (Fill)	1.9	1.4	0.9	1.5
B (Fill)	1.6	1.2	0.8	1.1
C (Fill)	2.2	1.5	1.0	1.8
D (Fill)	1.9	1.4	1.0	1.5
E (Fill)	1.7	1.3	0.9	1.4
F (Cut)	1.3	1.0	0.7	0.7
*F (Cut) at 1:2	1.7	1.3	0.9	1.7
G (Cut)	2.2	1.5	1.0	1.0
H (Cut)	2.6	1.7	1.1	1.3
N (Fill)	1.8	1.3	0.9	0.7
O (Fill)	1.7	1.2	0.8	0.8
R (Fill)	1.5	1.1	0.7	0.6

Section F, G, and H was remodelled with a reduced cut angle of 1V:2H.

Table 11 shows the calculated Factors of Safety (FOS) for each section under four scenarios: Static (target FOS 1.5), Seismic SLS Event (target FOS 1.2), Seismic ULS Event (target FOS 1.2), and Raised Groundwater (target FOS 1.2).

Fill Sections (A, B, C, D, E, N, O, R):

Most fill sections achieve acceptable FOS under static and SLS seismic conditions, but all fall below the target FOS under the ULS event. Sections B, N, O, and R fall below the target factor of safety for raised groundwater scenarios.

Cut Sections (F, G, H):

The analysis indicated that the cut batter represented by Section F did not return acceptable factors of safety for any design case. This was due to weak underlying materials to greater than 10 m. Following discussions with PPG, we reanalysed the cut with a reduced slope angle of 1V:2H (26°). The stability analysis completed with the reduced slope angle returned acceptable factors of safety for the Static, Raised Groundwater, and Seismic SLS cases.

The modified Sections G (Cut) and H (Cut) generally meet or exceed the required FOS in most scenarios, however they did not meet the required FOS under ULS conditions.

Natural Slopes

Table 12 summarises the slope stability analyses for representative natural slopes located below proposed lots.

Table 12: Slope Stability Analysis Summary (Natural Slopes)

Section	Static FOS (1.5)	SLS Event FOS (1.2)	ULS Event FOS (1.2)	Raised Groundwater FOS (1.2)
J	1.0	0.8	0.5	0.7
K	1.9	1.3	0.9	1.0
L-1	1.0	0.8	0.6	0.6
L-2	1.0	0.8	0.6	0.7
L-3	0.8	0.6	0.4	0.5

The analysis suggests that colluvium, completely weathered rock, and potentially the upper portion of highly weathered rock on slopes steeper than approximately 25° may be susceptible to failure under increased groundwater conditions, as well as SLS and ULS earthquake scenarios.

Based on these outcomes, it is advised that Specific Engineer Design (SED) Zones be applied to all proposed lots and infrastructure located above natural slopes exceeding a 25° angle. The locations and zone areas will be confirmed during detailed design. SED zones shall be included in development drawings and confirmed during construction once the interface with moderately weathered rock is identified and final contour data is available. Full delineation of all SED zones will be necessary for inclusion in the final Geotechnical Completion Report (GCR) documentation.

7.3.8 Seismic Displacement Analysis

The analysis suggests that batters under ULS conditions, may be prone to failure. We have undertaken a preliminary displacement analysis to determine the magnitude of displacement expected under ULS conditions. The magnitude of displacement may indicate the requirement for mitigation measures such as reducing slope angles or geogrid reinforcement.

During a ULS earthquake, the yield acceleration (the seismic acceleration at which the sliding forces equal the resisting forces) may be exceeded only briefly and hence the slope may only displace a small amount, assuming the soil on the slope does not significantly lose strength during these brief displacements. Calculated yield accelerations for the sections that may be prone to failure under ULS earthquake conditions are presented in Table 13.

As the soils at the site are not expected to lose strength during shaking, we have compared the yield acceleration with the amplified peak ground accelerations of 0.68 g (western site) and 0.748 g (central and eastern site) and then undertaken a seismic displacement analysis using the methods of Jibson, Bray & Travasarou, and Bray & Macedo based on the calculated acceleration.

We have analysed several slope types including:

- Fill (A, B, C, D)
- Cut (F, F at 1:2, G)

We have used the following input parameters for both Bray & Travasarou, and Bray & Macedo displacement analysis methods:

- V_s30 of fill material – 400 m/s.
- V_s30 of site – 400 m/s to 600 m/s.
- Moment Magnitude (M_W) of 7.5 obtained from National Seismic Hazard Model (GNS Science) website, for 1/500-year earthquake event for Wellington with V_s30 of 400 m/s.
- Displacement estimates were obtained using moment magnitudes for both shallow crustal earthquakes (M_W 7.5) and subduction zone interface earthquakes (M_W 8.2) following the methods of Bray & Macedo (2019). The estimates presented below are from whichever earthquake type (shallow crustal or subduction zone interface) gave the largest displacement.
- Initial Fundamental Period calculated using results from our slope stability analysis and following methods of Bray (2007).
- Spectral acceleration obtained from National Seismic Hazard Model (GNS Science) website, using input parameters obtained from our slope stability analysis results.

Table 13 presents the estimated downslope displacements that may occur under a ULS earthquake.

Table 13: Yield Acceleration and Displacement Estimates for ULS Seismic Event

Section	Yield Acceleration (k_y)	Estimated Displacement (mm)		
		Jibson	Bray & Travasarou (2007)	Bray & Macedo (2019
A (Fill)	0.34 g	160	180	160
B (Fill)	0.24 g	440	400	340
D (Fill)	0.34 g	120	230	200
E (Fill)	0.28 g	290	230	260
F (Cut)	0.13 g	1400	630	430
F (Cut) with a cut at 1:2	0.25 g	300	530	220
G (Cut)	0.35 g	130	150	80

It should be noted that the methods for calculating displacement are highly theoretical. We acknowledge that in an ULS earthquake event there is likely to be significant variability in the ground motion across the site, that the quality and type of fill material is likely to vary across the site depending on the source, and that changes in these parameters would likely significantly alter our displacement analyses. There is also inherent variability within and between the seismic displacement analysis methods themselves.

The values obtained from our seismic displacement analyses should be treated as an indication of performance rather than true estimates of expected displacement. Our analysis generally indicated that the seismic displacement risk for cut, fill, and natural batters is likely to vary based on the slope angle and height and displaced material strength.

Our analysis indicates that the anticipated seismic displacements for the proposed fill batters are likely to be moderate to large.

Cut batters in stronger, well-cemented rock and batters with joint sets with favourable orientations, such as those dipping into the slope, exhibit lower susceptibility to seismic displacement. Conversely, batters located on weaker, highly fractured, or weathered rock, or those where the rock bedding or jointing is unfavourably aligned (for example, dipping out of the slope), are associated with a significantly higher risk of displacement during seismic events. The variability seen in Table 13, such as the difference between Section F and G, highlights the importance of site-specific geological assessments: some locations may model moderate displacements, while others—particularly where unfavourable discontinuities are present—may show substantially larger movements. Therefore, the displacement risk is not uniform across the site, and careful evaluation of rock mass properties at each cut batter location is essential to inform appropriate mitigation strategies.

Due to the expected seismic displacements, mitigation solutions will be required for all large fill batters to reduce the displacement risk during ULS earthquake events. We have provided recommendations for each slope type in Sections 8.2 and 8.3.

7.4 Unsuitable Soil Embankment

Based on the plans provided to ENGEO and our discussion with Envelope and PPG, we understand that a permanent embankment comprised of site-won unsuitable material is proposed to be constructed within one of the large gullies on the eastern side of the site. The proposed design currently shows a structural bund at the base of the 'unsuitable' embankment, and a structural fill batter at the head of the gully.

We have undertaken an assessment of the slope stability of the proposed 'unsuitable' embankment and of the structural fill batter at the head of the gully. Our cross sections are taken from the Envelope Engineering Ltd drawings (Reference 1753-03, dated 11.06.2024, Drawings 228-229). The conceptual design plan for the 'unsuitable embankment' and cross section are provided in Appendix 8.

The current design plans show the 'unsuitable' embankment will extend part way up the structural batter, essentially buttressing the toe of the batter. However, as the final extent of the 'unsuitable' embankment is not currently known, we recommend designing the structural fill batter to not rely on buttressing from the 'unsuitable' embankment.

7.4.1 Analysis Methods

We have followed the same analysis methods as outlined in Section 7.3.3. Several other key assumptions were made in our analysis and are outlined below:

- Subsoil drainage will be installed beneath the unsuitable material prior to placement on the existing ground.
- Saturation of the top 2 m of the unsuitable and structural fill material during high rainfall / raised groundwater modelling scenario.
- We have assessed the stability of the 'unsuitable' embankment and the structural fill batter at the head of the gully under two scenarios:
  - Scenario 1 – Stability of the 'unsuitable' embankment and structural fill batter as designed (with buttressing of batter toe).
  - Scenario 2 – Stability of structural fill batter with no buttressing of toe from 'unsuitable' embankment (i.e. following catastrophic collapse of embankment or embankment with reduced volume).

7.4.2 Strength Properties

The soil parameters used in our analysis are the same as those presented in Table 6 and Table 7 with the addition of several additional soil types as defined in Table 14.

Table 14: Assumed Engineering Properties of Unsuitable Embankment Material

Material	Unit Weight (kN/m^3)	Strength Type	Cohesion	Friction Angle (degrees)
Fill – Unsuitable	18	Mohr-Coulomb	1	20
Fill – Unsuitable (Saturated)	18	Mohr-Coulomb	0	20

In our analysis, we have modelled the lower structural embankment (supporting the ‘unsuitable’ embankment) with geogrid reinforcement. The geogrid material used in our analysis was Cirtex SGU300 with an allowable tensile strength of 136 kPa. Other geogrid material parameters are presented in the relevant slope stability analyses in Appendix 9.

7.4.3 Seismic Design Conditions

We have used the same design seismic ground accelerations in our calculations as those defined in Table 8 for Eastern Site sections.

7.4.4 Analysis Results

We do not believe that the acceptable Factors of Safety as defined in Table 9 are applicable when considering the stability of ‘unsuitable’ embankment as a catastrophic failure of the embankment is unlikely to be a hazard to the development as there will be no lots beneath the ‘unsuitable’ embankment. Catastrophic failure of the ‘unsuitable’ embankment will result in inundation downslope, either along the embankment itself or into the proposed wetland area. This area could be remediated in the event of a catastrophic failure of the ‘unsuitable’ embankment and is unlikely to result in any life safety risk to nearby lots.

The acceptable Factors of Safety as defined in Table 10 should be applied when considering the stability of the structural fill batter at the head of the gully as lots will occupy the top of the batter.

We have assessed the overall slope stability factor of safety under static conditions, raised groundwater / high rainfall conditions, a 1/25-year earthquake event, and a 1/500-year ULS earthquake event. Table 15 summarises the slope stability analyses for the proposed ‘unsuitable’ embankment and structural fill batter with full slope stability analyses presented in Appendix 9.

Table 15: Slope Stability Analysis Summary (Unsuitable Embankment and Structural Batter)

Failure Scenario	Static FOS	Raised Groundwater FOS	SLS Event FOS	ULS Event FOS
Scenario 1 (as designed)	1.9	1.5*	1.1	0.6
Scenario 2 (structural fill batter without toe buttressing)	1.7	1.4	1.3	0.9

*Critical failure plan is in structural batter.

Our analysis indicated that the global stability of the proposed unsuitable ‘embankment’ returned acceptable factors of safety for the Static and Raised Groundwater cases in the currently proposed configuration (Scenario 1).

Our analysis indicated that the global stability of the structural fill batter at the head of the gully without buttressing from the ‘unsuitable’ embankment (Scenario 2) returned acceptable factors of safety for the Static, Raised Groundwater, and Seismic 1/25-year cases.

The analyses indicated that in the currently proposed configuration (Scenario 1), the ‘unsuitable’ embankment, lower structural embankment, and perhaps the upper structural batter, could be prone to failure under ULS earthquake conditions. The analysis also indicated that without the buttressing effect of the ‘unsuitable’ embankment (Scenario 2) the structural batter at the head of the gully may be prone to failure under ULS earthquake conditions. As such, we have completed a seismic displacement analysis for these sections.

7.4.5 Seismic Displacement of Structural Fill

We have undertaken seismic displacement analyses for the ‘unsuitable’ embankment and structural fill for the two scenarios outlined above using the methods of Jibson, Bray & Travasarou, and Bray & Macedo based on the calculated acceleration.

We have used the same input parameters for both Bray & Travasarou, and Bray & Macedo displacement analysis methods as outlined in Section 7.3.8 with the following addition:

- V_s30 of unsuitable material – 200 m/s.

Table 16 presents the estimated downslope displacements that may occur under a ULS earthquake.

Table 16: Yield Acceleration and Displacement Estimates for ‘Unsuitable’ Embankment Area

Section	Yield Acceleration (k_y)	Estimated Displacement (mm)		
		Jibson	Bray & Travasarou (2007)	Bray & Macedo (2019
Scenario 1 (as designed)	0.17 g	960	290	350
Scenario 2 (structural fill batter without toe buttressing)	0.28 g	300	N/A	200

The values obtained from our seismic displacement analyses should be treated as an index of performance rather than true estimates of expected displacement. The results from our analysis give a categorical indication that the expected seismic displacements for the ‘unsuitable’ embankment will be moderate to large.

The analysis suggests that failures originating within the ‘unsuitable’ embankment may extend into the structural batter above and therefore displacement may occur within the structural fill. Due to limitations in assessing the stability of ‘unsuitable’ embankment and fill batter using a 2D model and pseudo-static seismic load, we have not completed a seismic displacement analysis for failure of the structural fill material in Scenario 1.

Results from our stability analysis for Scenario 2 indicate that displacement within the structural fill batter at the head of the gully is likely to be moderate.

Due to the shallow slope of the ‘unsuitable’ embankment, we expect the mechanism of failure of the material, should it occur, to be through slumping / displacement rather than as a debris flow down the face. As there are no proposed lots beneath the ‘unsuitable’ embankment, we do not consider the displacement of this material under ULS earthquake conditions, as indicated in our analysis, to pose a significant life safety risk.

Mitigation solutions will need to be considered for the structural fill batter at the top of the gully due to the risk of moderate displacement of material under ULS earthquake loading, as indicated in our analysis. Potential mitigation options for this batter are detailed in Section 8.2.

As the structural bund at the base of the unsuitable embankment has been modelled with geogrid, it will need to undergo detailed design before engineering approval is given. The geogrid use in all attached slope stability models are concept design only required to satisfy resource consent.

7.4.6 Previous Geotechnical Experience and Observations

ENGEO has previously undertaken slope stability assessments for similarly sized unsuitable material stockpiles along the Transmission Gully motorway. ENGEO has carried out detailed ground investigations and slope stability analyses on these embankments which have been constructed with broadly similar materials to what will likely make up the ‘unsuitable’ embankment at this site. We have observed that the Transmission Gully material stockpiles have generally stood up well over the last four to five years, with only minor surficial failures. These stockpiles were generally left at angles steeper than those proposed for the unsuitable embankment at the site.

Based on this evidence, as well as our slope stability analysis for the unsuitable embankment presented above, we believe that slope instability will not be a significant hazard for the proposed unsuitable embankment, provided that the lower embankment is subject to detailed design, the surface is well drained and that the embankment is vegetated.

7.5 Debris Flow

Topographic and aerial photography reviews suggest that there may be potential source areas for debris flows in the gully slopes near the eastern boundary of the property, due to the steep topography. Historical aerial image review and observations made during ENGEO’s 2022 and 2025 site investigation found evidence of small debris flows occurring at the base of shallow soil landslides.

We consider that debris flow events could be initiated under prolonged high intensity rainfall, likely as a result of evacuative shallow soil failures on steep slopes. Once initiated, debris flows would be expected to follow the existing gully features throughout the site.

At this stage the conceptual earthworks plans do not show development in the low-lying gully areas of the site, and therefore we do not consider debris flows to pose a significant hazard to the development of the site.

7.6 Rockfall

Several of the proposed cut slopes across the site are greater than 6 m in height. There is a potential risk of rockfall beneath any large cut slopes that are made in Greywacke sandstone. Cuts made in completely to highly weathered Greywacke sandstone that is highly fractured are more likely to exhibit localised frittering rather than large rock release which is required for significant rockfall.

All large cuts (>6 m in height) made on-site should be assessed by a suitably qualified Ground Engineering professional on a case-by-case basis and where applicable, rockfall mitigation solutions may be required. Mitigation options may include benching and planting of cuts faces, or construction of debris barriers at the base of cut slopes to protect dwellings from impact during a rockfall event.

7.7 Consolidation Settlement

We consider that some of the flat areas of the site with softer / less dense soils may be subject to static settlements under loads associated with further fill placement or the proposed residential developments. We anticipate that this may occur due to consolidation of soft soils, particularly within gully features across the site. The observed soft material within these gully features varies and in some cases is very loose / soft to stiff / medium dense. This material will need to be excavated prior to fill placement.

7.8 Tunnel Gullies

Tunnel gullies were identified in one location on the southeastern portion of the site. This is on steep terrain which is proposed to be cut. Tunnel gullies are not static features—they evolve through a sequence of subsurface and surface processes that are highly sensitive to environmental conditions, land use, and soil properties.

The area identified as containing tunnel gullies will be excavated through bulk earthworks and as such are not considered to pose a significant hazard to the development of the site.

Any cut batters that have loess soils present will require a site-specific erosion design to prevent tunnel gullies occurring. If other tunnel gullies are identified which were not identified during our site walkover, then they should either be removed through the bulk earthworks or resolved through specific design.

7.9 Collapsible Soils

Collapsible soils are unsaturated materials that can experience sudden settlement when wet or loaded. Although they seem stable when dry, increased moisture may cause significant volumetric strain and threaten structural integrity. While site is not currently mapped as containing collapsible soils, the presence of aeolian (loess) and alluvial deposits, low-plasticity silts, and loose sands suggest that collapsible soils may be present that were not identified during our investigation. Historical geotechnical investigations in similar terrain have identified collapse-prone layers in loess and silty sands, particularly where weak cementation and high void ratios are present.

At present, the risk from collapsible soils is assessed as very low, based on our site investigations, laboratory testing and geotechnical characterisation. However, this risk should be reassessed if future investigations identify moisture-sensitive or low-density soils within the development footprint.

7.10 Earthworks, Landform Modification, Historical Fills

Earthworks and landform modification are integral to the development of the site and must be carefully assessed to ensure geotechnical stability, compliance with NZS 4431:2022, and long-term performance. Historical fills, undocumented ground disturbance, and cut / fill transitions present potential risks that require mitigation through design and construction controls. Development of shear keys, removal of soft subgrade or uncontrolled fill materials, and implementation of suitable erosion and sediment controls as well as drainage controls will be important for creating stable building footprints.

8 Recommendations

8.1 Earthworks

Detailed earthworks recommendations will be outlined in the project Earthworks Specification. In this section are key considerations regarding the proposed earthworks.

8.1.1 Vegetation

Within the earthwork areas, any permanent unsealed slopes shall be re-vegetated to reduce the likelihood of future erosion. All new unsealed slopes shall be protected from overland water flow by the addition of bunds or other suitable measures.

Beyond the earthwork areas, de-vegetation shall be minimised to reduce the likelihood of future slope instability or erosion.

8.1.2 Placement of Structures

All structures should be located a safe distance back from the crest of steeply sloping areas (steeper than 25 degrees) or have specific engineer designed foundations. A safe distance is likely to be in the order of 5 m. This distance will be assessed during detailed design and confirmed during the construction stage.

Structures at the base of slopes may require catch fences or appropriate set back distances to mitigate against slope instability or rockfall as discussed in Section 8.3.

8.1.3 Groundwater

Although unlikely, if groundwater is encountered during excavation, specific measures to control groundwater (such as a sump and pump, plus bridging fill) may be required. The presence of water may impact fill compaction quality, as such groundwater controls should be implemented. This will be detailed in the earthworks management plan.

8.2 Permanent Fill Batters

The modelled fill batters as shown on the development plans do not meet the ULS seismic FOS. Specific design of the fill batters will be required during detailed design stage. Mitigation options for fill batters include geogrid reinforcement, reducing the batter slope angle, or designing foundations and services specifically for structures within the fill material at the top of batters if appropriate. In addition, site-specific triaxial testing should be conducted to confirm the soil parameters of the fill material. If the soil parameters differ significantly from those used in the analyses in this report, the slope stability modelling should be updated.

8.2.1 Filling

All filling works should be completed in accordance with NZS 4431:2022 and the site-specific earthworks specification.

Free flowing water must be kept away from all fill slopes.

It is recommended that hydro-seeding take place shortly after completion of the fill slopes.

8.2.2 Use of Existing Soils as Engineered Fill

Rock, clean sand, and sand / silt / gravel mixtures were encountered across the site. Four Particle Size Distribution (PSD) and seven Standard Compaction tests were undertaken on a selection of these soils, as well as three hand-mixed samples with different variations of rock and silt / sand. Based on the results and ENGEO's experience working with similar material on nearby sites, we consider that the majority of the soils and rock likely to be encountered on the site will be re-usable as fill, provided they are processed to remove any organic material, debris, and over-sized particles.

Some surficial soils encountered on the site, including topsoil and the alluvial / lacustrine deposits, contain silts with varying amounts of organic material, and generally a high moisture content. These materials are unlikely to be suitable for re-use as fill.

It is likely that additional compaction testing will be required during the construction phase to obtain accurate maximum dry density estimates of site-won material as this material is expected to vary across the site, spatially and well as with depth.

8.2.3 Fill Batter Gradients

Permanent fill batter angles will be confirmed in the Project Earthworks Specification after all required laboratory testing has been completed, and batter specific slope stability analyses have been carried out. All fill batters greater than 8 m in vertical height will require site specific design to mitigate seismic displacements as highlighted in Section 4.

For unreinforced fill batters lower than 8 m in height, a safe long term batter angle should be established through slope modelling using the specific parameters obtained through the laboratory testing and used in the final designs.

8.3 Permanent Cut Batters

Our slope stability and displacement analysis indicate that the proposed cut slopes do not meet the required FOS under ULS conditions. In the case of Section F, as unmodified, the factor of safety is below the required amount in all scenarios modelled. Potential mitigation options for cut batters that may be prone to instability include reduction of batter slope angles, setback zones at the toe and crest of batters, benching, or construction of a catch fence / barrier at the base of cut batters. We recommend that a suitably qualified Ground Engineering professional assess the stability of all cut batters during construction.

8.3.1 General

Cut batters should be formed during the summer months where possible and vegetated promptly after construction. Stormwater from the ground above the proposed cuts should be picked up and discharged around the face in a controlled manner to minimise erosion of the cut faces. All permanent cut batters should be protected by planting and / or hydroseeding. Cut batters (generally those greater than 8 m depth) that have loess soils present will require a site-specific erosion design to mitigate against tunnel gullies occurring.

Cut batters above 1.5 m depth and / or cut batters near existing / proposed structures should be inspected by a geotechnical professional during construction to confirm (or modify) safe batter angles.

All excavated soil should be removed from site, reused immediately or placed in a stockpile such that it does not cause slope instability.

In the unlikely event that seeps are encountered in exposed cut faces, this may lead to slumping and possibly small muddy flows. Such seeps may need treating by surface and / or subsurface drains, or other site-specific measures. If encountered, advice from a geotechnical professional should be obtained.

Catch fences / barriers may be required at the bottom of cuts to protect structures or roads from rock fall loosened by fretting or erosion.

8.3.2 Cut Batter Angles

Preliminary cut batter angles are presented in Table 17. These have been developed from site observations and experience with other similar materials. This table provides preliminary estimates of suitable slope angles for cuts up to 8 m in height. The conceptual plans provided to ENGEO indicate cuts significantly higher than this, and as such, specific slope stability analysis has been carried out for several representative large cuts as detailed in Section 7.3.

If the recommended cut batter angles are not achievable, the slope should be retained by an engineer designed structure and / or detailed slope stability analysis should be undertaken. Further investigations would be needed for the design of any retaining structures.

Table 17: Proposed Cut Batter Angles

Unit	Permanent Cut Batter Angle (degrees above horizontal)	Temporary Cut Batter Angle (degrees above horizontal)	Comments
Topsoil or old uncontrolled fill	1V:3H (18°)	1V:2H (26°)	
Natural silts and clays	1V:3H (18°)	1V:2H (26°)	
Natural sands and gravels	1V:2H (26°)	1V:1.5H (34°)	
Greywacke	1V:1H (45°)	1V:0.35H (70°)	Appropriate cut batter angles in Greywacke rock are likely to be governed by the rock structure and defect orientation. Permanent slopes at 1V:0.8H (55°) may be acceptable for cuts under 1.5 m high which are more than 1.5 m from structures.

8.3.3 Cut / Fill Transitions

Buildings whose subgrade contains both cut and filled ground could experience differential soil movements. Structures proposed on such sites should be designed to accommodate differential settlements.

Alternatively, such building sites can be reconstructed to create uniform subgrade conditions. This can be accomplished by excavating the soil across the entire building pad to a minimum depth of 0.6 m below founding level and replacing the excavated material with uniformly compacted fill.

8.4 Stormwater Retention Wetlands

Based on the proposed stormwater attenuation plans (Envelope Engineering ref: 1753-02 Rev P1, Drawn September 2025), we understand that five stormwater retention wetlands will be constructed for the development. These conceptual design plans show that the wetlands will be formed by constructing earthen embankments to retain stormwater discharges from the development, with embankment slopes varying between 1V:2H to 1V:3.5H. The wetlands will all maintain a permanent water level.

Each of the stormwater retention wetlands appear to collect runoff from different catchments and are not hydraulically linked. A cascading dam failure has therefore not been considered plausible for this development.

Soils won from the development are anticipated to be used to form the embankments and will comprise sandy silt / silty sand or excavated Greywacke sandstone. Based on our experience of similar soils, we believe that the site won materials can be reused to form stable earthen embankments with performance that meets the requirements of the New Zealand Dam Safety Guidelines (NZSOLD, 2024). This assumption will be validated during detailed design of the stormwater retention wetland embankments.

A summary of the proposed stormwater retention wetland characteristics is provided in Table 18. These characteristics are based on the provided conceptual design plans (ref: 1753-02 Rev P1, Drawn September 2025) and will be refined during detailed design of the development. Based on discussions with Envelope, we understand that the reduced levels are based on the NZVD2016 for vertical datum.

Table 18: Proposed Stormwater Pond Characteristics

Characteristic	Wetland A	Wetland B	Wetland C	Wetland D	Wetland E
Type of Embankment	Earth Fill	Earth Fill	Earth Fill	Earth Fill	Earth Fill
Upstream Batter Slope	1V:2.5H	1V:2H	1V:2.5H	1V:3H	1V:3.5H
Downstream Batter Slope	1V:2.5H	1V:2H	1V:2.5H	1V:2H	1V:3H
Crest Width	3.6 m*	39.2 m*	8.0 m*	3.0 m*	8.2 m*
Crest Level	44.00*	58.17*	68.00*	110.00*	93.80*
Downstream Toe Level	36.75	50.45	60.40	100.40	89.12
Wetland Invert Level	38.00*	52.60*	61.90*	108.50*	89.10*
Permanent Water Level	41.00	54.30	64.50	108.80	91.40
Height of Embankment	7.30 m*	7.80 m*	7.60 m*	9.40 m*	4.30 m*
Auxiliary Spillway Level	42.60	57.00	67.00	109.52	93.00
Auxiliary Spillway Scruffy Dome Diameter	1300 mm*	1300 mm*	1200 mm*	1200 mm*	2700 mm*
Emergency Spillway Weir Level	43.50	58.17	67.50	109.70	93.70
Permanent Water storage volume	770 m^3	480 m^3	720 m^3	50 m^3	1,090 m^3
10-year storage volume	2,480 m^3	3,020 m^3	2,350 m^3	130 m^3	5,820 m^3
100-year storage volume	3,680 m^3	4,980 m^3	4,050 m^3	180 m^3	8,150 m^3
Maximum Possible stored volume	5,700 m^3	10,700 m^3	6,950 m^3	261 m^3	11,650 m^3

* = Measured from the plans at 1:250.

Under the Building (Dam Safety) Regulations (MBIE, 2022), each of the proposed stormwater retention wetlands are not considered to be ‘classifiable’ as their maximum possible stored volume is less than the regulations specify (storage of <20,000 m^3). Therefore, a Potential Impact Classification assessment is not required for the wetlands.

A comprehensive earthworks specification including an inspection and testing plan will be provided to the earthworks contractor prior to starting excavations, along with a robust erosion and sediment control plan.

8.5 Surface Water Management

During construction, appropriate measures shall be undertaken to control and treat stormwater runoff, with silt and erosion controls complying with local body guidelines for erosion and sediment control.

Overland flows should be directed away from existing slopes to reduce the risk of ponding and erosion leading to slope instability concerns.

8.6 Erosion and Sediment Control

Higher fill and cut slopes could be vulnerable to erosion if concentrated flows develop. Mitigation solutions should be implemented for all large fills (greater than 3 m in height) to control runoff from the fill batters and scouring of the surficial fill. Mitigation options may include benching of the face of the fill, addition of geogrid within the fill (and wrapping of the face), a site-specific hydroseeding plan (to reduce the likelihood of erosion), and bunds at the top of batters during construction. It should be noted that the addition of geogrid to fill batters will require specific detailed design.

9 Safety in Design

ENGEO considers the items in Table 19 to present a significant Health & Safety risk to persons working on the project, these will need to be considered by the project team during the design phase, to eliminate or mitigate the risk they present.

Table 19: Safety in Design

Risk	Risk Rating	Potential Control Measures
Large Cuts	Significant	Select appropriate temporary cut angles to reduce risk of slope failure. Implement H&S plans that reduce the amount of time people spend around large cuts. Block off cut areas when work is not actively occurring in the area. Engineered temporary stability measures if other solutions are not viable.
Steep Slopes	Moderate	Contractor shall not carry out works on, immediately above, or immediately below slopes during or immediately after heavy rainfall. Complete planting work within embankment / batter faces during construction, rather than at the end. A geotechnical professional should assess the stability of unretained slopes on site during construction.
Water collecting in base of cuts, reducing slope stability and creating a dangerous work environment	Moderate	Include water management in the construction management plan prior to undertaking earthworks, including contingency planning for containing unexpected water sources (e.g. springs).

10 Assessment Against Section 106 RMA

The proposed development is situated on and near land that is subject to natural hazards. To grant subdivision consent, the consenting authority will need to consider the risk from these natural hazards. Section 106 of the Resource Management Act requires a combined assessment of the likelihood of the hazards, the material damage (consequence) of the hazards, and whether the proposed use of the land would accelerate or worsen the hazard.

We have assessed the natural hazards in Section 7 of this report and have provided a summary in Table 20.

Table 20: Natural Hazards Risk

Hazard	Event	Proposed mitigation measure(s)	Risk once mitigation has been implemented	Are the works (including mitigation) likely to accelerate or worsen the hazard?
Fault rupture through the site during earthquake	Building collapse or major damage; major damage to other infrastructure	X	Low	No
Slope Instability	Landslide / displacement causing damage to dwellings or infrastructure	Follow permanent cut / fill batter angle recommendations. hydroseeding of batter and slope faces. SED zones or set back zone established for lots above natural slopes > 25°. For fill batters with seismic displacement risk - reinforcement with geogrid or SED zones within fill material at top of batters.	Low	No
	Debris flow causing damage to dwellings or infrastructure	Do not construct dwellings / structures in likely debris flow paths (stream axis, gully / low-lying areas). If necessary, construct debris flow barriers or channels.	Low	No
	Rockfall causing damage to dwellings or infrastructure	Reduce angle of permanent cuts. Bench and hydroseed cut faces. Construction of catch fence at base of cuts.	Low	No

Liquefaction	Ground settlement causing damage to dwellings or other infrastructure	Follow earthworks guidance. If construction of dwellings is to occur in potentially liquefiable soils, detailed liquefaction analysis will be required.	Low	No
	Lateral Spreading causing damage to dwellings or other infrastructure	X	Low	No
Settlement	Settlement of buildings under normal conditions	Follow Earthworks Specification and fill and lot testing requirements.	Low	No
Erosion	Erosion of soil on slopes	Follow recommended cut and fill batter angles, control stormwater and overland flow paths, hydroseed faces of cut / fill batters and natural slopes, carry out bulk earthworks in summer.	Low	No
Collapsible Soils	Settlement under high groundwater or rainfall events	Compact material that is identified as sensitive to their optimum moisture contents.	Low	No
Tunnel Gullies	Erosion or subsidence below dwellings, Difficulty in earthworks.	Excavate out areas affected by tunnel gullies. Pile foundations through material susceptible to tunnel gully formation	Low	No

We have not assessed the risk of inundation from flooding or tsunami as these assessments lie outside our area of expertise. Several flooding zones are located within the site as indicated by Porirua City Council's mapping.

We have not considered the risk of building damage due to ground shaking during earthquakes. Although this risk has a geotechnical element, we consider that this risk has been addressed in NZS1170.5.

11 Conclusions

We conclude that, although there are geohazards present at the site, the risk to proposed buildings from natural hazards will be acceptably low once the proposed works and the proposed mitigation measures and recommendations described in this report have been completed. We also consider that the proposed works will not accelerate, worsen, or result in material damage to the land so long as the recommendations in this report are followed.

Site-specific soil strength testing (laboratory triaxial testing) is awaited to determine if the soil parameters for the fill material are in line with those used in the slope stability analyses presented within this report. Should these results differ from the soil parameters outlined in this report, then the slope stability analyses carried out in Sections 7.3 and 0 should be reassessed. It is possible that this may alter the geotechnical recommendations made in this report.

Most of the surficial soils are suitable for reuse as engineered fill. Rock material will be suitable for reuse as fill, however, consideration will need to be made in regard to the variability of rock weathering and strength characteristics. We recommend additional lab compaction testing is carried out during the construction phase as materials change throughout the project lifecycle.

Slope stability assessments of the proposed cut and fill batters indicate that batters may be prone to failure during ULS earthquake events, and that seismic displacement of material may be moderate (< 100 mm) to large (< 2 m). As such, mitigation solutions will need to be considered for these batters.

The proposed cut batters showed unsuitable factors of safety under seismic conditions. The modelled failures are expected to occur in the softer surficial materials. When modelled with a reduced slope angle in the surficial soil, the factor of safety increases. The final cut batter angles will be confirmed during construction when the ground conditions are confirmed at each cut batter location.

The site has five proposed stormwater retention wetlands. The embankments forming these retention wetlands and the slopes surrounding the retention wetlands will require a detailed investigation, analysis, and design.

The proposed large fill slopes do not meet the required FOS for ULS conditions and several do not meet the required FOS under raised groundwater conditions. The identified fill embankments will be specifically designed and will likely require geogrid reinforcement.

Any cut batters that have loess soils present will require a site-specific erosion design to mitigate against tunnel gullies occurring. If other tunnel gullies are identified which were not identified during our site walkover, then they should either be removed through the bulk earthworks or resolved through specific design

12 Limitations

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

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

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

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

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (04) 472 0820 if you require any further information.

Report prepared by	Report reviewed by
Callum Whitten	**Adam Smith, CMEngNZ (PEngGeol)**
Engineering Geologist	Associate Engineering Geologist
Thomas Vollebregt	**Karen Jones CEnvP**
Engineering Geologist	Principle Engineering Geologist

13 References

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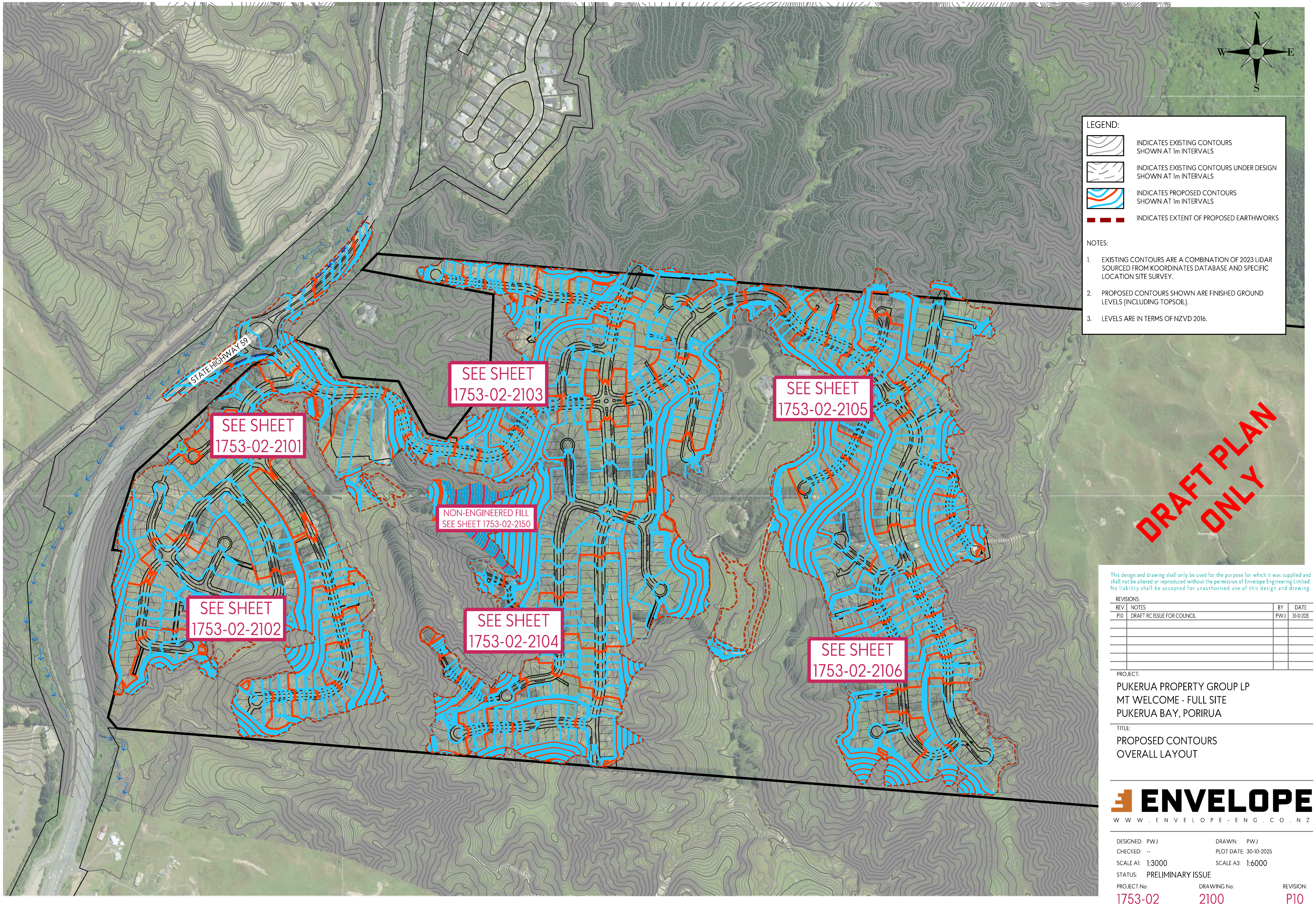
We also acknowledge the New Zealand GeoNet project and its sponsors EQC, GNS Science and LINZ, for providing data used in this report.

APPENDIX 1:

Proposed Site Contour Plan



LEGEND:

INDICATES EXISTING CONTOURS SHOWN AT 1m INTERVALS

INDICATES EXISTING CONTOURS UNDER DESIGN SHOWN AT 1m INTERVALS

INDICATES PROPOSED CONTOURS SHOWN AT 1m INTERVALS

INDICATES EXTENT OF PROPOSED EARTHWORKS

NOTES:

1. EXISTING CONTOURS ARE A COMBINATION OF 2023 LiDAR SOURCED FROM KOORDINATES DATABASE AND SPECIFIC LOCATION SITE SURVEY.
2. PROPOSED CONTOURS SHOWN ARE FINISHED GROUND LEVELS (INCLUDING TOPSOIL).
3. LEVELS ARE IN TERMS OF NZVD 2016.

REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:

PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

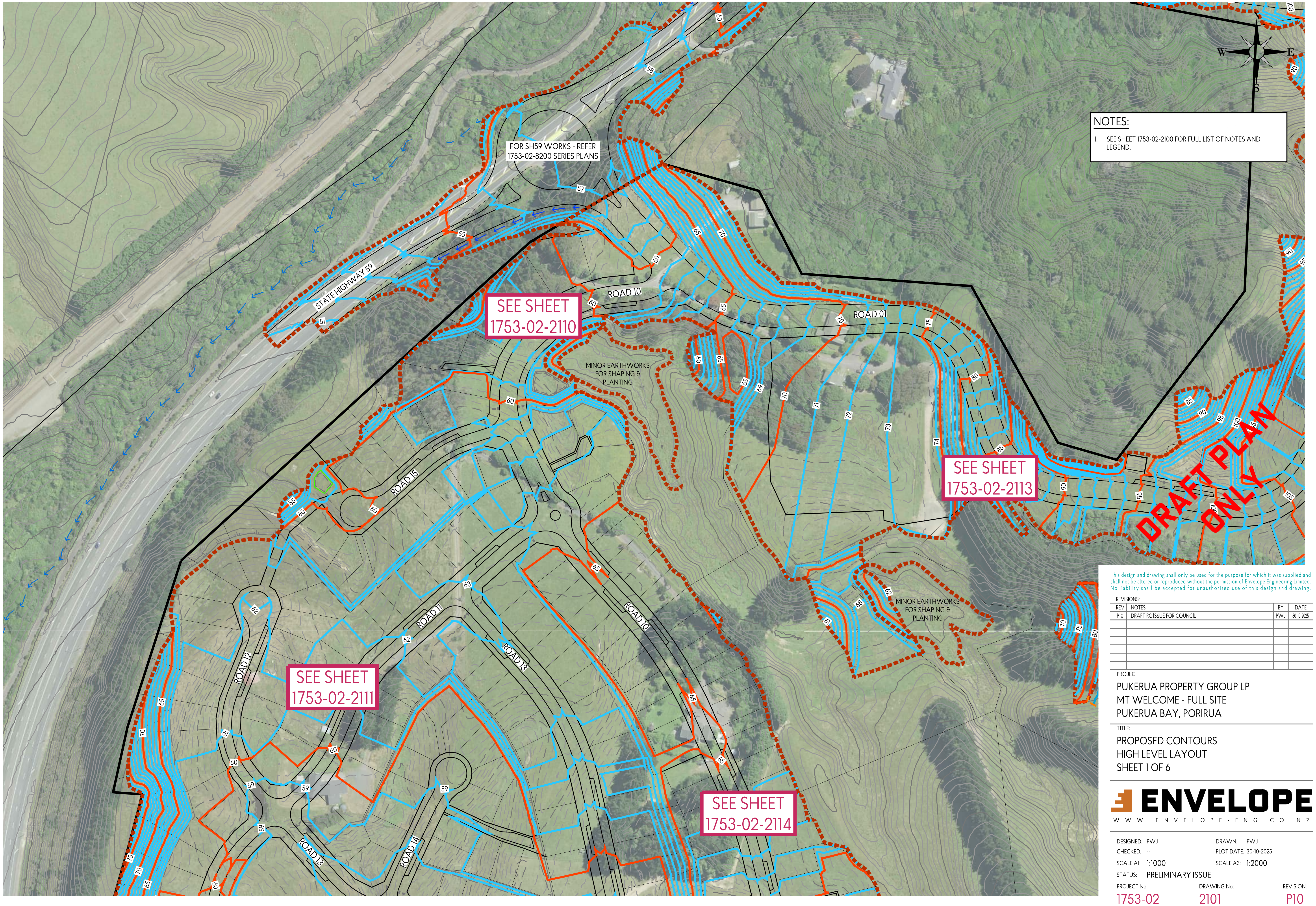
TITLE:

PROPOSED CONTOURS OVERALL LAYOUT

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
SCALE A1: 1:3000
SCALE A3: 1:6000
STATUS: PRELIMINARY ISSUE

PROJECT No: 1753-02
DRAWING No: 2100
REVISION: P10



REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:
PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

TITLE:
PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 1 OF 6

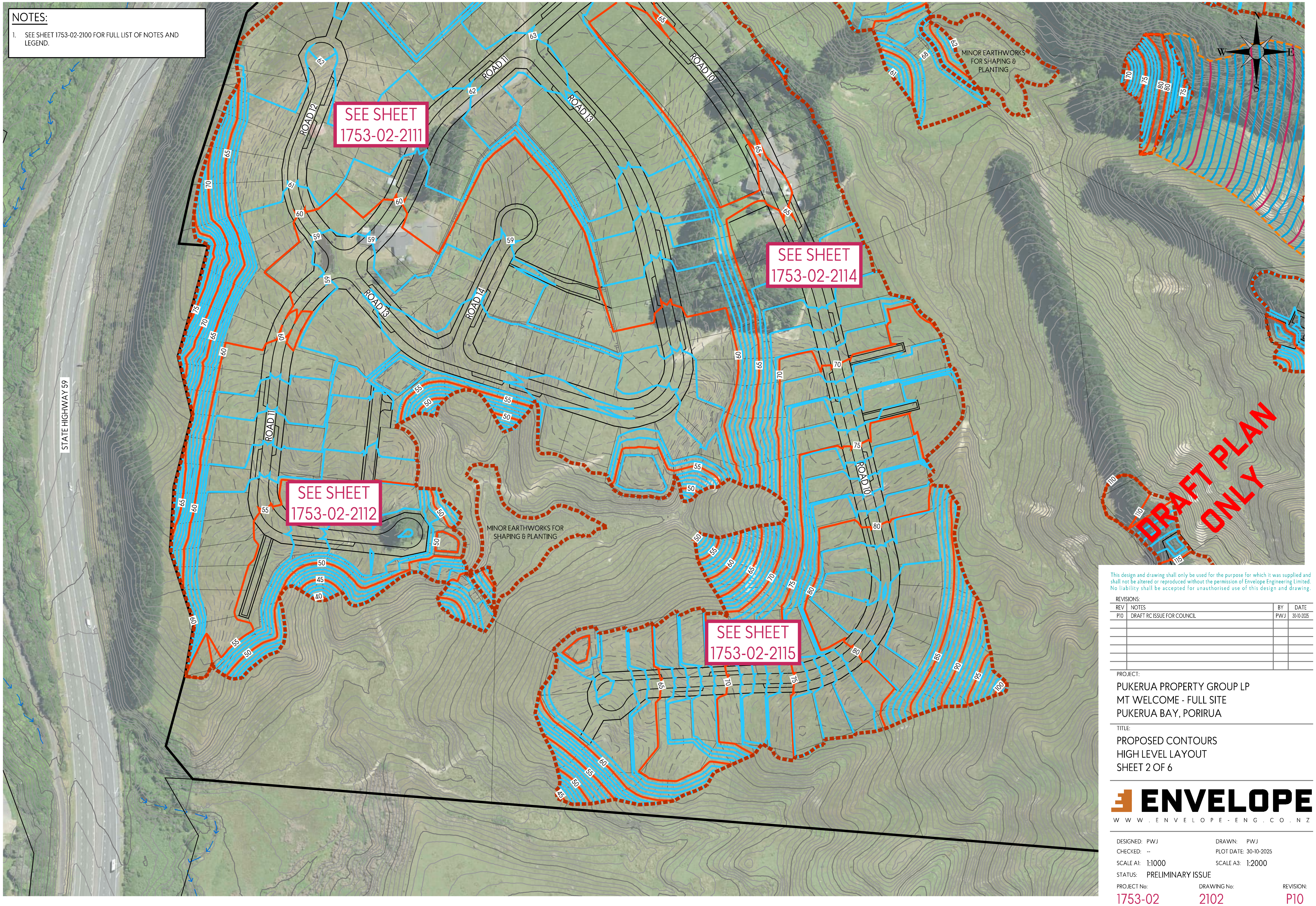
ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
SCALE A1: 1:1000
SCALE A3: 1:2000
STATUS: PRELIMINARY ISSUE

PROJECT No: 1753-02
DRAWING No: 2101
REVISION: P10

NOTES:

1. SEE SHEET 1753-02-2100 FOR FULL LIST OF NOTES AND LEGEND.



REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:

PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

TITLE:

PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 2 OF 6

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
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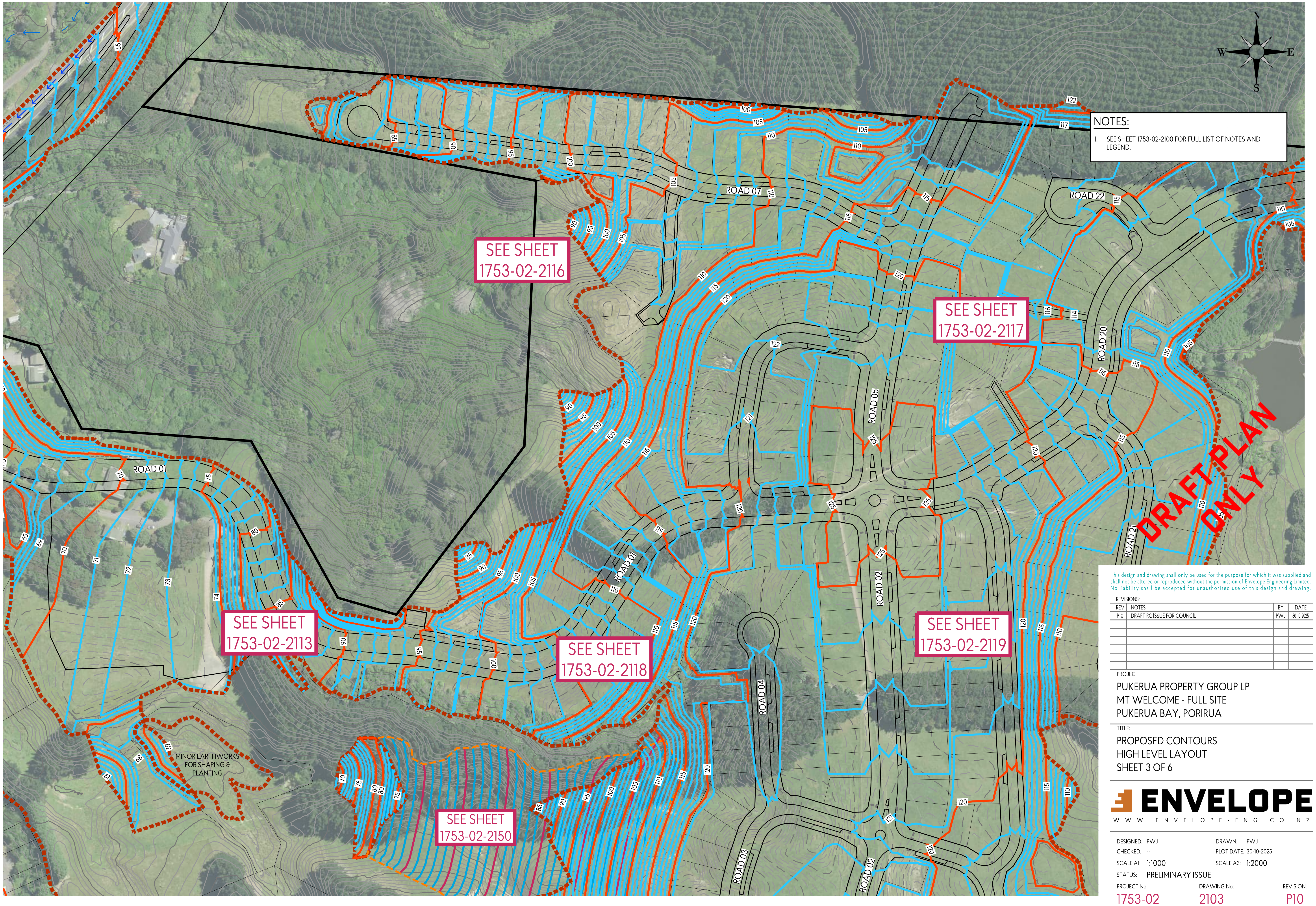
SCALE A3: 1:2000
STATUS: PRELIMINARY ISSUE

PROJECT No: 1753-02
DRAWING No: 2102
REVISION: P10



NOTES:

1. SEE SHEET 1753-02-2100 FOR FULL LIST OF NOTES AND LEGEND.

REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:

PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

TITLE:

PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 3 OF 6

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
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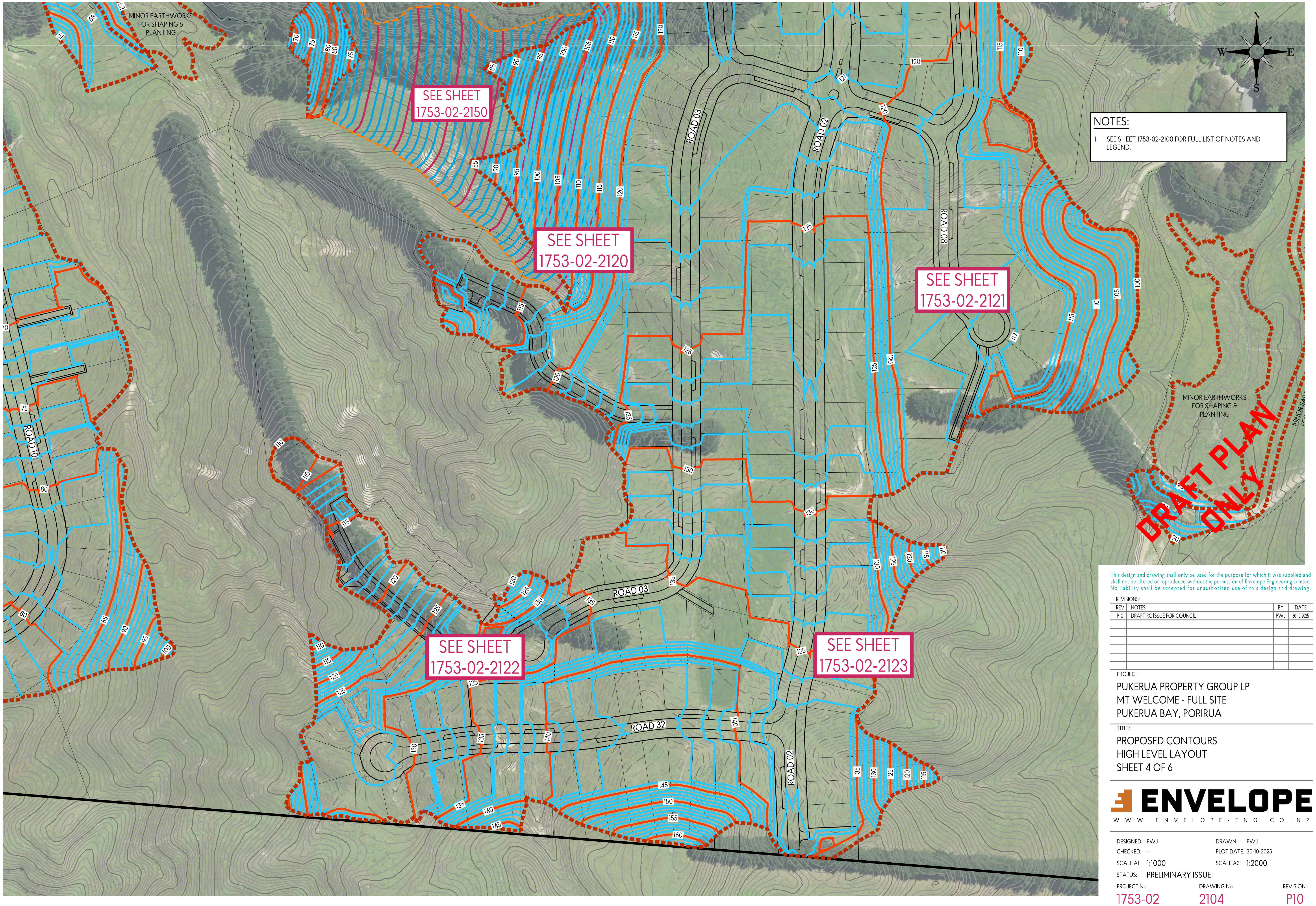
PROJECT No: 1753-02
DRAWING No: 2103
REVISION: P10



NOTES:

1. SEE SHEET 1753-02-2100 FOR FULL LIST OF NOTES AND LEGEND.

REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:

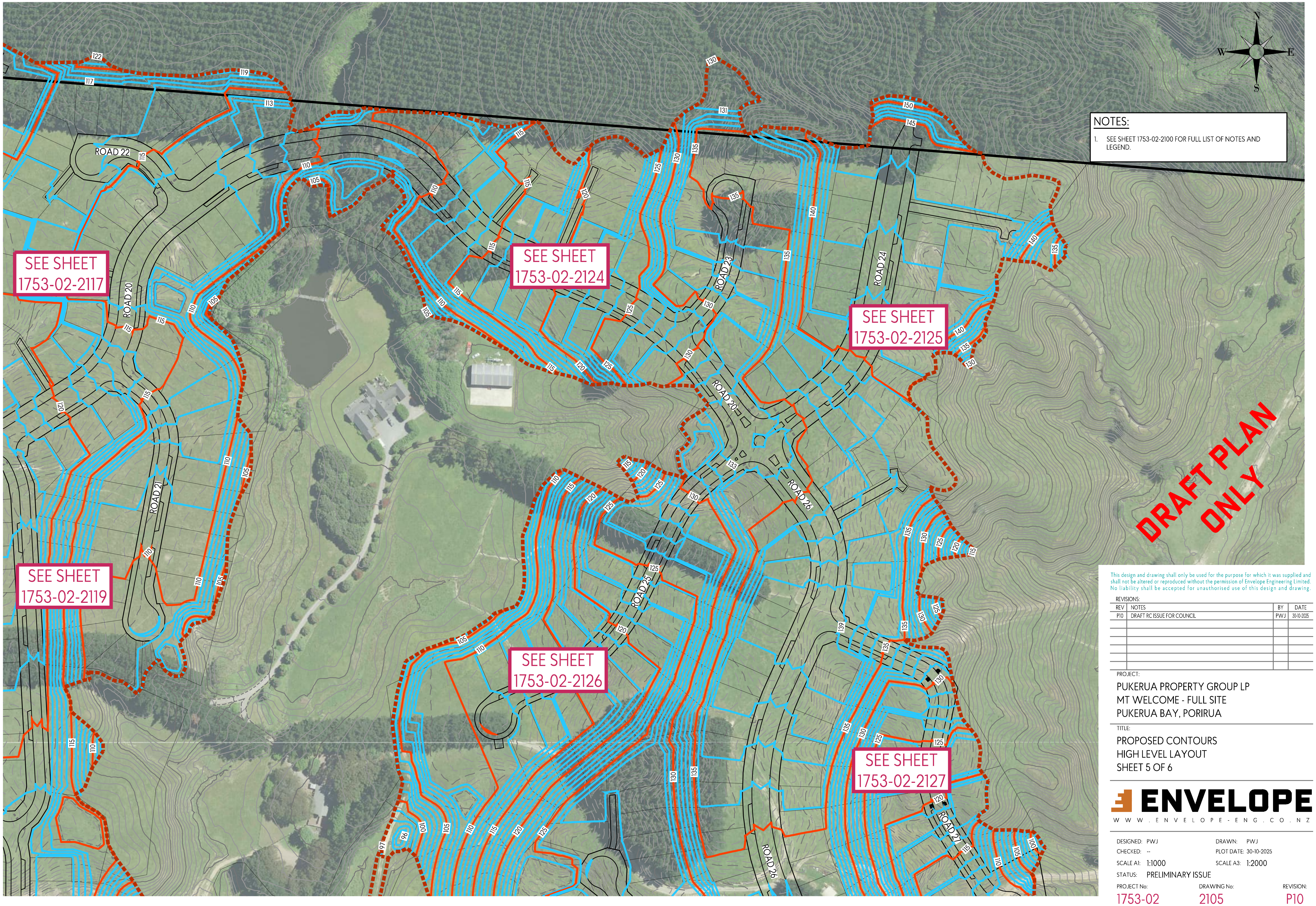
PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

TITLE:

PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 4 OF 6

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
SCALE A1: 1:1000
SCALE A3: 1:2000
STATUS: PRELIMINARY ISSUE
PROJECT No: 1753-02
DRAWING No: 2104
REVISION: P10



NOTES:

1. SEE SHEET 1753-02-2100 FOR FULL LIST OF NOTES AND LEGEND.

REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:

PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

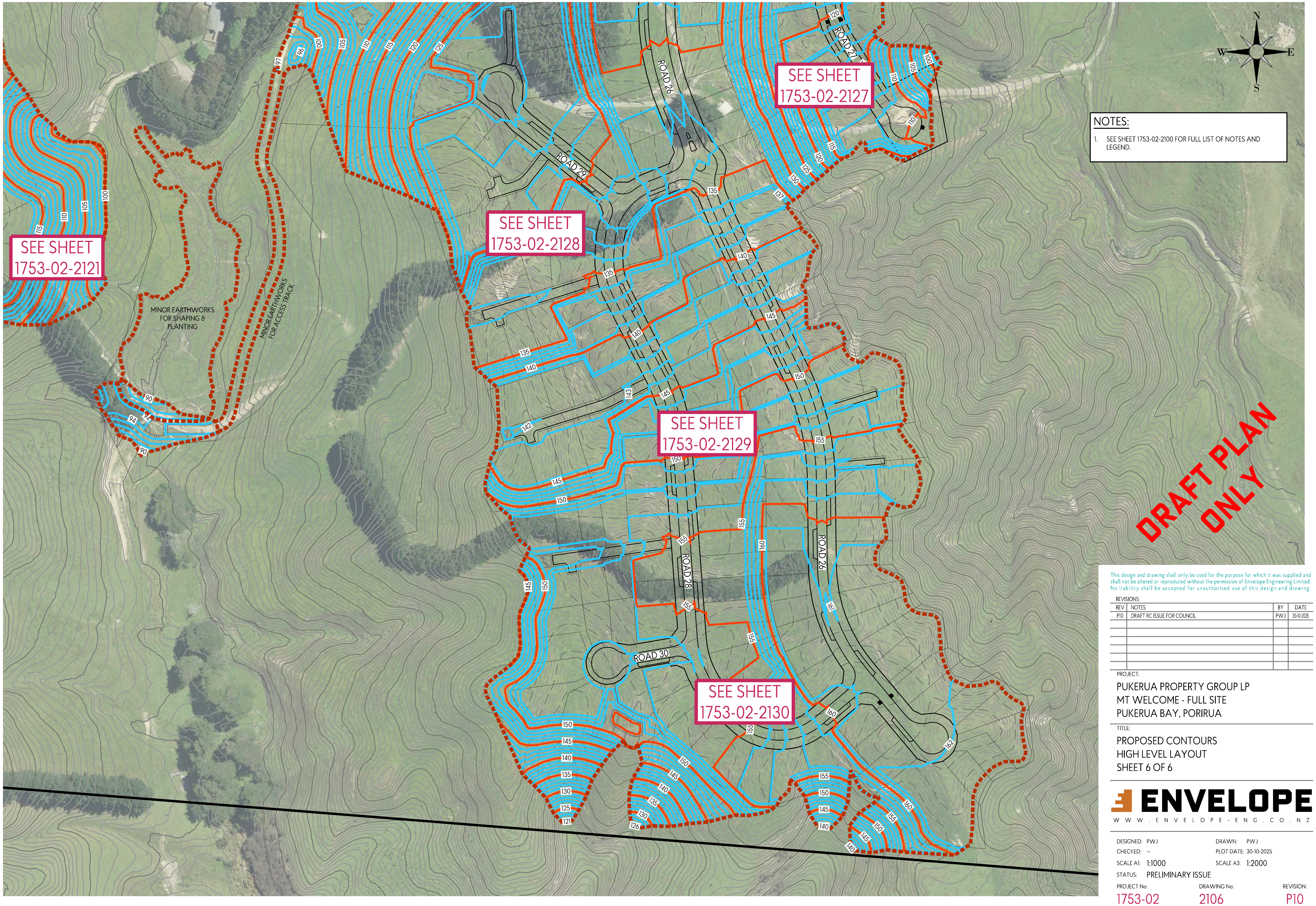
TITLE:

PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 5 OF 6

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
SCALE A1: 1:1000
SCALE A3: 1:2000
STATUS: PRELIMINARY ISSUE

PROJECT No: 1753-02
DRAWING No: 2105
REVISION: P10



NOTES:

1. SEE SHEET 1753-02-2100 FOR FULL LIST OF NOTES AND LEGEND.

REVISIONS:

REV	NOTES	BY	DATE
P10	DRAFT RC ISSUE FOR COUNCIL	PWJ	30-10-2025

PROJECT:
PUKERUA PROPERTY GROUP LP
MT WELCOME - FULL SITE
PUKERUA BAY, PORIRUA

TITLE:
PROPOSED CONTOURS
HIGH LEVEL LAYOUT
SHEET 6 OF 6

ENVELOPE
WWW.ENVELOPE-ENG.CO.NZ

DESIGNED: PWJ
DRAWN: PWJ
CHECKED: --
PLOT DATE: 30-10-2025
SCALE A1: 1:1000
SCALE A3: 1:2000
STATUS: PRELIMINARY ISSUE
PROJECT No: 1753-02
DRAWING No: 2106
REVISION: P10