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Project Number 16998.000.002

**Cannon Point Stage 3 Geotechnical
Investigation for Resource Consent**

10 Cannon Point Road, Craigs Flat, Upper
Hutt, Wellington 5018

Submitted to:
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Executive Summary

ENGEO Limited was engaged by Cannon Point Developments Limited to complete a series of geotechnical investigations for Stage 3 of the overall subdivision development at 10 Cannon Point Road, Craigs Flat, Upper Hutt. The purpose of this report is to support Fast Track Resource Consent for the proposed subdivision. The work supplements earlier investigations and refines the natural hazard assessment relevant to earthworks, subdivision layout and future residential development using both Section 106 of the RMA and the National Policy Statement 2025.

Investigations undertaken included a targeted desktop review of published and previous project information (including geology / geomorphology, hazard datasets, LiDAR / DEM and historical imagery), an engineering geology walkover and geomorphic mapping (undertaken on 4 March 2026), and supplementary intrusive investigation comprising six test pits to approximately 2.3–2.7 m with associated Scala penetrometer and shear vane testing, supported by review of available deeper borehole / SPT information from earlier work.

Our analysis involved using relevant investigation data to develop an Engineering Geological Model (EGM) and assess the hazards most relevant to subdivision feasibility and dwelling location. This included seismic hazard (ground rupture and ground shaking), liquefaction screening, slope stability and seismic deformation assessments (including displacement estimates) for representative sections, runout length estimation, and debris flow / sediment-laden stormwater modelling (HEC-RAS 2D, 250-year ARI with climate change allowance), within a risk-based consenting context (Section 106 RMA and the National Policy Statement for Natural Hazards).

Key hazards assessed include strong seismic shaking (with low but not fully excludable surface rupture potential at site margins), slope instability and runout (including potential seismic-induced deformation at the terrace crest), debris flow / sediment-laden stormwater from upslope catchments, and localised shallow groundwater effects near the wetland/stream. Liquefaction and lateral spreading are considered of negligible likelihood given dense gravels and generally deep groundwater, and rockfall hazard is assessed as low based on the limited, generally historic evidence observed.

We consider that Stage 3 is suitable for the proposed subdivision provided suitable mitigations for the identified geohazards are incorporated into the final design. Slope instability (including terrace-crest seismic deformation and potential runout from slopes above the site) is the primary constraint and will require engineered mitigation and / or appropriate setbacks, including Special Engineered Design (SED) for affected lots. Debris flow (sediment-laden stormwater) risks are generally manageable through robust stormwater conveyance, erosion / sediment controls and ongoing maintenance, with targeted measures where modelling indicates higher exposure. With these measures in place, residual natural hazard risk is expected to be acceptable for consenting purposes.

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1 Introduction

ENGEO Ltd was engaged by Vecta Limited, on behalf of Cannon Point Developments Limited (CPDL), to undertake a geotechnical investigation and prepare associated reporting to support an application for resource consent for Stage 3 of the proposed development at 10 Cannon Point Road, Craigs Flat, Upper Hutt (herein referred to as “the site”). This work has been carried out in accordance with our signed agreement (ref: P16998.000.002_02) dated 26 February 2026.

At the time of ENGEO undertaking this investigation, Cohesive Consulting Limited (CCL) has undertaken a geotechnical site suitability report for the purpose of supporting Resource Consent application for Stage 3 (ref:100-007-07, dated 14 May 2024). Additionally, ENGEO undertook a series of investigations and produced a geotechnical report for the proposed Stage 3 of the Cannon Point Development (ref: 16998.000.001_03, dated 14 August 2020). However, CPDL wish to submit for resource consent under the Fast-Track Approval Act 2024 which has prompted the need for further work.

The purpose of this referral report is to summarise the site assessment, which has included a geomorphic walkover to identify hazards and further geotechnical investigations across the site to develop an understanding of the geotechnical and geomorphological hazards posed to the site which may affect its suitability for development.

Our scope of works is outlined in the ENGEO proposal (ref: P16998.000.002_02), and includes the following activities:

- Review of published and client provided reporting and geological information.
- Review of historical satellite imagery, LiDAR, and publicly available council hazard maps.
- Site walkover by an experienced ground engineering professional to assess geo-hazards and geomorphic risks present, on, above, and below the site.
- Undertake supplementary geotechnical investigations across Stage 3.
- Undertake preliminary slope stability analysis of critical sections to the development.
- Preliminary retaining or ground improvement options.
- Preliminary foundation recommendations.
- Earthworks recommendations, including preliminary batter angles and geotechnical parameters for retaining wall design.
- Assessment of identified geohazards against Section 106a of the Resource Management Act and National Policy Statement for Natural Hazards 2025.
- Production of this report summarising our findings and recommendations for site suitability.

2 Site Description

Stage 3 of the Cannon Point Development is proposed to encompass approximately 45,400 m² of land spread across three parcels (Lot 1000 DP 598401, Lot 3 DP 306622, and Lot 4 DP 306622), which are zoned as general rural and rural lifestyle under the Upper Hutt City Council District Plan.

The elongate northeast to southwest oriented site is raised above the Hutt Valley floor (Tōtara Park) on a series of variably elevated terraces. It is located at the base of the north-western range of Upper Hutt, which separates the Hutt Valley and the Akatarawa Valley (Figure 1). The Hutt River runs parallel to the site and is approximately 500 m south of the southernmost point of Stage 3.

The site is currently predominately grassland. It is largely flat with the undulating and steepening ground at the base of the north-western slopes. The site contains an approximately 1,500 m² wetland area in the central region.

Figure 1: General Site Location Plan



Generated using Datanest.

3 Proposed Development

The Stage 3 development master plans provided by Vecta Limited (ref: Cannon Point West Block, dated 4 May 2026; Appendix 1), shows a 67 Lot subdivision which will comprise residential dwellings, rain gardens and green spaces/communal areas, spread across the two elevated terraces. The proposed residential lots, of which there are 46, will have land areas between 229 m² and 902 m².

The proposed earthworks and civil details are outlined in the Vecta Limited drawing package (ref: 26014 Cannon Point Fast Track STAGE 3, dated 23 February 2026). A lot layout is contained within the Vecta drawings, however, should be ignored for the purpose of this resource consent. The most up-to-date layout is displayed in the architectural drawings (ref: Cannon Point West Block, dated 4 May 2026), and are yet to be included in the Vecta Civil Plans.

The current service plans indicate that the stormwater and sewer main from the northern and southern extents of the site will converge near Road 2, where gravity will drain these systems to the lower terrace. Stormwater will drain to a rain garden, where it will eventually be drained off-site, while the sewer main will continue to flow north before diverting east and connecting with the sewer mains of California Drive.

As outlined in the master plans for Stage 3 of the Cannon Point Development, some earthwork land modification is required. Across the site, planned earthworks comprise a total area of 35,400 m², with 17,070 m³ of cut, 25,830 m³ of site-won fill, and 8,760 m³ additional imported fill, totaling 34,590 m³ of fill. Due to material limitations on-site an additional 8,760 m³ of fill material will be required to be imported. A maximum cut depth and fill height of 6.0 m is proposed for the site.

Based on the available plans, it is anticipated that retaining structures between 1.0 and 6.0 m will be required at several lots. Additionally, larger retaining walls (up to 6 m) will be required where cuts into the north-western slopes are proposed. Road 01 between chainages 800 – 850, 980 – 1040, and 1150 – 1240 will likely all require engineered retaining solutions (retaining or Mass Stabilised Earth / Reinforced Soil Slope).

4 Desktop Study

As part of our desktop study, we have reviewed the following resources to help identify hazards present at the site:

4.1 Topography

The site is located across two elevated terrace that are tens of metres above the surrounding alluvial flood plain of the Hutt Valley and Hutt River. The primary terrace is located furthest upslope while a smaller lower elevation terrace positioned halfway between the existing valley floor and the dominant primary terrace. The site generally comprises undulating grassland along its northeast to southwest orientation, however, does contain some clustered areas of denser, semi-mature vegetation (6-8 m high).

To the west of the site is a series of steep ridgelines subdivided into five subcatchments, each with a southeast facing gully of variable size. The subcatchments are generally covered with mature, native vegetation.

Debris fans at the base of the primary gullies are variable in size, with some fans adjoining.

Some minor modifications have been made to the topography at the north-eastern end of the site where material is being stockpiled from existing stages of development.

An approximately 1,500 m² wetland is located in approximately the centre of the site, which appears to coincide with a gentle flattening or inversion of the terrace slope, limiting the draining capacity of the local area.

4.3 Seismicity

The GNS New Zealand Active Fault Database website indicates that the site is located within 20 km of several active faults, two of which (Wellington and Wairarapa Faults) are listed as major faults that require near-fault factor consideration (Table 3.6; NZS 1170.5:2004).

The Wellington Fault is located within 250 m of the site and is well constrained to California Drive (Tōtara Park) by Van Dissen et al., (2005) and others; however, to the northeast and southwest of the site uncertainty of the fault's location increases due to later erosional and depositional overprinting (Figure 3). The Wellington Fault, like many faults in the southern North Island is an oblique dextral reverse fault, with strike-slip motion is the primary slip component. The Wellington Fault has a recurrence interval of between 715-1575 years, with the last event estimated to be approximately 400 years ago (Van Dissen et al, 2005). Annual slip rate is estimated to be on the order of 6 mm/yr, while the horizontal net single displacement event has been estimated to be 4.22 m.

The Wairarapa Fault is located 15 km southeast of the site and is well constrained based on a series of offset geomorphic man-made features (e.g., offset streams / river and terraces), and a well-documented M_w 8.1 rupture in 1855. The recurrence interval of the Wairarapa Fault is estimated to be between 1040 and 1420 years. It is an oblique fault which displays dextral and reverse senses of motion. A net slip rate of 11.6 mm/yr has been established based on a series of high net single horizontal displacement events, the most recent of which were measured between 13 and 18 m (Rodgers and Little, 2005). Rodgers and Little (2005) additionally measured vertical displacements along the fault up to 2.7 m.

Several other faults not listed in Table 3.6 of NZS 1170.4 are within 20 km of the site, and include the Akatarawa, Moonshine, and Ohariu Fault. The Ohariu Fault is the largest of these faults, with net horizontal single displacements of 3-5 m. The last event on the fault was between 950-1000 years, with a typical recurrence interval of 800-7000 years.

Figure 3: Wellington Fault Location and Confidence of Fault Location

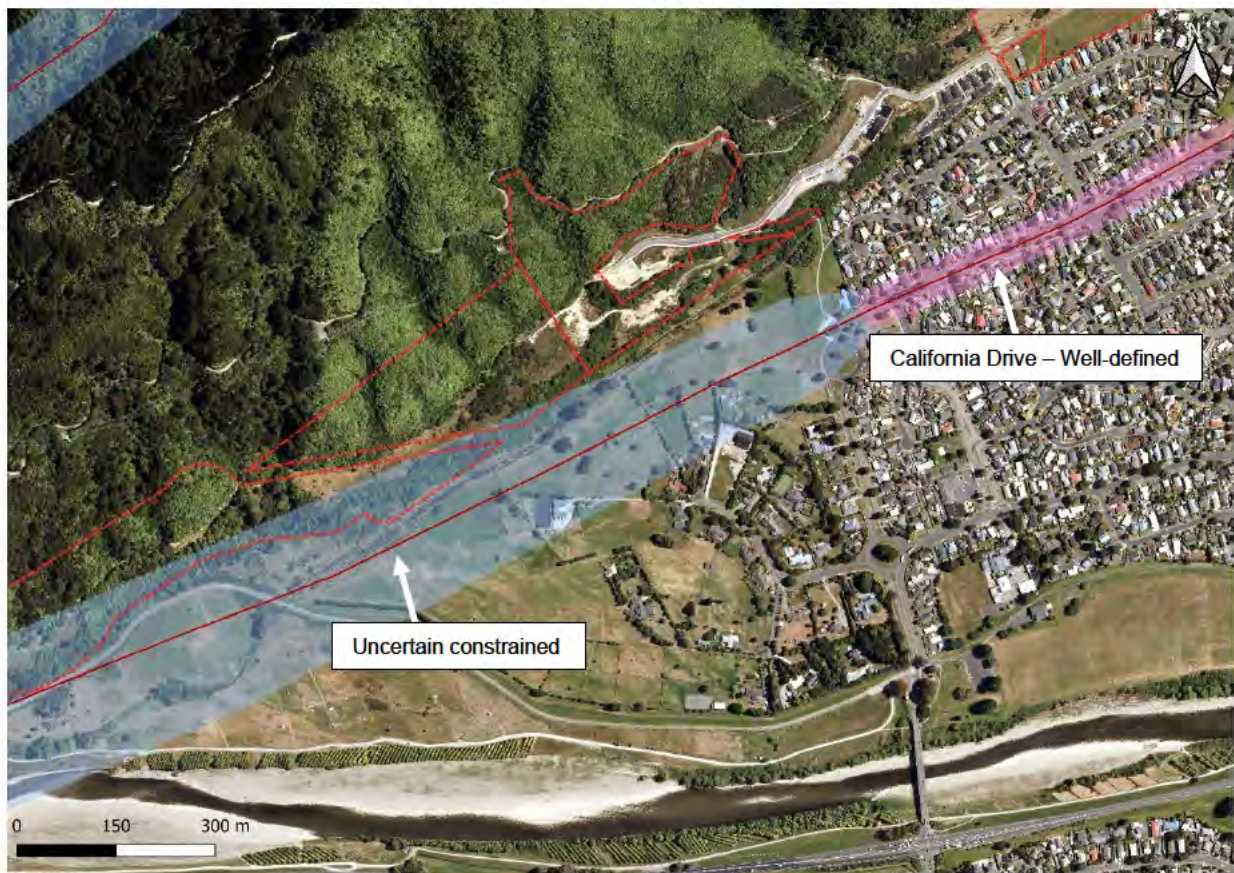


Image sourced from GNS New Zealand Active Fault Database. Reproduced using QGIS.

4.4 Historical Aerial Photography Review

ENGEO has undertaken a review of Historical Aerial Photographs using Retrolens (LINZ, 2025). Aerial images are displayed in Appendix 2 from which the following observations were made:

- **1943:** Stage 3 is covered in grassland, with little vegetation across the surface of the terrace. Minor stream channels have been cut across the terrace surface from the gullies located on the western side of the site. A series of variably sized debris fan like structures appear to extend from the apex of the primary gully systems to be, however these are vegetated with grass. East of the site, the Hutt River displays a series of derelict meandering channels raised above the active channel. These may represent overflow or flood channels, however; none appear to have been eroding the base of the terrace slope. Three large what appear to be high voltage transmission piles are present on the site. There does not appear to be a wetland established during this period.
- **1962:** There appears to be some stockpiling of material at the southern end of the site above the edge of the terrace. Several farm tracks appear to have been established at the base of the slope north of the site. More vegetation has established across the centre of the site. No significant modification to the site has occurred. The smaller sub-terrace, which appears to have been developed prior to 1974, is seen to be in its original state with several minor easterly ridges and gullies surrounding. No development has commenced in the Tōtara Park area.

- **1974:** Tōtara Park to the east of the proposed development has become well established. The high voltage transmission lines have been removed from the site and replaced with larger pylons on slope above the site. The smaller sub-terrace has been earthworked, with the south-eastern extensions being reduced to a single batter.
- **1980:** Little to no notable modification was made to the site, with vegetation becoming more established.
- **2003:** The site remains largely unaltered since the previous aerials although some clearing of vegetation has occurred.

4.5 Hazard Mapping

The Greater Wellington Regional Council maps indicate that the site has a 'moderate' combined earthquake hazard rating. This rating is derived from a 'low' slope failure hazard, a 'low' liquefaction potential, and a 'low to moderate' ground shaking hazard. The Greater Wellington Region Council Online Maps display that the site is located well away from the tsunami evacuation zone.

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

4.6 New Zealand GNS Landslide Database

The New Zealand Landslide Database (Morgenstern et al., 2025) shows three recorded landslides occurred above the site that occurred in 2006 (Figure 4). The database, indicates that the previous events were rainfall induced.

Figure 4: Recorded Landslides in the New Zealand Landslide Database.



Landslide locations illustrated by orange triangles. Data sourced from GNS Landslide Database and reproduced in QGIS.

4.7 Select Land Use Register

The Greater Wellington Regional Council's Selected Land Use Register (SLUR) database indicates that no known contamination has been identified on the site, and that no hazardous activities or industries are known to have been undertaken on the site. Given the historical land use, it is possible that fertilizers and sprays may have been used leading to some soil contamination. Furthermore, offal pits may be present however, none were found in the investigations undertaken for this report.

4.8 New Zealand Geotechnical Database / Previous Investigations

The New Zealand Geotechnical Database (NZGD) shows that little investigations have been uploaded to the site. ENGEO has been provided with investigation data from the site and surrounding area for use in this report which is summarised as follows.

- RDCL undertook five borehole investigations between 28 May and 3 June 2021 using Websters Drilling & Exploration Limited within the Stage 3 area, with termination at 15 m. ENGEO has been provided with the drillers logs and SPT results for two of these investigations. Groundwater levels were recorded at greater than 15 m depth.
- CCL undertook 28 Scala penetrometer tests across the area to a maximum depth of 3.0 m.

- ENGEO undertook an investigation in 2020 (ref: 16998.000.000_03, dated 14 August 2020) which comprised 12 test-pit excavations, six of which were undertaken within the proposed Stage 3 development. Five machine boreholes were drilled within the Stage 3 area during this investigation, the logs for two of which are available. Logs are displayed in Appendix 3.

5 Site Investigation

To assess the suitability of the site for the proposed development, a site walkover was undertaken to identify potential geohazards and to understand the site's historical and active geomorphology and what constraints they might place on development of the site. This was complemented by an intrusive ground investigation to confirm subsurface conditions, validate previous geological models, and supplement the spatial dataset within the Stage 3 area.

5.1 Site Walkover and Geomorphic Observations

ENGEO visited the site on 4 March 2026 to undertake the site walkover and geomorphic mapping, the following observations were made. (site photographs are displayed in text in Figure 5):

- The main entrance to the site, located at the north, was being used as a temporary fill stockpile area, and contained some construction debris (pipes and brick), with some organic materials.
- The wetland and the area immediately surrounding the wetland are well vegetated with Kānuka.
- An historic farm track was cut into the base of the north-western slope along the full length of the site exposing colluvium faces (landslide material) and cut across the alluvial fans at the apex / termination of the major gullies on the north-western slope.
- Extending from the base of the three major catchments draining to the site were semi-circular fan shaped structures. These were steepest close to the apex of the gullies and shallowed with distance. The ground within these areas was undulating and well vegetated with Mahoe, Totara, and ferns indicating little recent activity given the relative maturity of vegetation.
- The gullies displayed varying levels of incision and debris in their channels. Incision within the channels was typically between 0.5 m and 2.0 m, with the streams meandering to some extent approximately through the centre line of the gullies. Small sub-metre thick debris flow deposits were present in the sidewalls of the stream, with at least three discrete layers identifiable. A small well-vegetated overflow channel was present to the south of gully two.
- Several isolated cobbles and boulders were noted along the upslope side of the farm track ranging in size from 50 – 200 mm (along the long axis), however, none extended any great distance away from the toe of the slope. Based on the moss and vegetated nature of some of the boulders, it would indicate that rockfall has not occurred recently
- As part of our site walkover ENGEO observed the headwaters of the catchment areas affecting the site, which were well vegetated with regenerating natural trees, some of which were estimated to be 50 to > 100 years old. No obvious source areas of failures were observed within the catchments.

- A pronounced slope occurs along the crest of the terrace to the southeast of the site. The terrace slope was well vegetated with the crest of the slope being near vertical in some locations, however it is unclear whether this is because of failure at the crest of the slope. The remainder of the slope was not visible, nor safe to access.
- At the south-western end of the site a large debris fan was present which increased in slope and elevation towards the large catchment at the southern end of the site. The drainage gully from the catchment has since incised into the debris fan and terrace below. A farm track runs the length of the site, that is cut into the toe of the northwestern slopes. The farm track crosses several streams, with culverts at the apex allowing the streams to flow out to the fan and terrace tread.
- Several power poles are present on the site along the crest of the terrace. Visually, some of the poles appeared to be overturning, and were subsequently measured with angles of 2-5° from vertical.
- The crest of the terrace in proximity to the wetland area appears to have a slight shallow / reversal of slope across the terrace, which appears to be stagnating the flow of water.
- Due to the density of vegetation cover the lower terrace was unable to be accessed.

A summary of our geomorphological observations is displayed in the geomorphic maps attached in Appendix 4.

Figure 5: Site Walkover and Geomorphic Observations



Photo 1: Untested fill stockpiling at northern end of site.



Photo 2: Catchment two feeding wetland area, central site.



Photo 3: Colluvium identified in farm track cut, west of site.



Photo 4: Small-scale debris flows in streams above site.



Photo 5: Largest rockfall evidence from slopes above site.



Photo 6: Native vegetation on slopes above the site.



Photo 7: Terrace tread debris flow indicated by red dashed line.



Photo 8: Small-scale failure at terrace crest (red dashed line).

5.2 Subsurface Investigations

ENGEO completed additional subsurface investigations to compliment and confirm the ground conditions observed during the previous investigations undertaken on the site.

Six test-pits with associated Scala penetrometers and Shear Vanes were undertaken across the site and achieved depths of 2.3 to 2.7 m. Test-pits are labelled STG3-TP01 through STG3-TP06. Material was logged in accordance with the NZGS field classification guidelines (NZGS, 2005).

The geotechnical investigation logs are displayed in Appendix 5 while the typical materials encountered are summarised in Table 1.

Table 1: Generalised Geological Profile

Material / Description	Description	Consistency / Density	Depth to Top of Layer (m)	Depth to bottom of Layer (m)
Topsoil	SILT with some sand and trace gravel	Firm	0	0.15 - 0.3
Fan Deposits ¹ / Colluvium	Fine to coarse SAND with minor silt and grave / fine to coarse GRAVEL with some cobbles & sand, and minor silt	Loose to very dense	0 – 0.15	0.5 – 2.6
Alluvium	Cobbly fine to carse GRAVEL with some sand and minor silt	Tightly packed	0.3 – 1.2	14.15 – N/A
Slightly Weathered Greywacke Sandstone	Greywacke Sandstone	Very dense	14.15	N/A

Notes

1. STG3-TP01 undertaken within largest debris fan on-site, located at the southern end of the site.
2. Greywacke Sandstone only encountered in BH09.

6 Engineering Geological Model

An Engineering Geological Model (EGM) has been developed to provide a conceptual framework for understanding the site's geological and geomorphic conditions and the processes influencing them. The EGM integrates geomorphic mapping, engineering geological observations, and subsurface investigation data to support assessment of natural hazards relevant to the proposed development.

6.1 Model Development

The EGM has been developed in general accordance with Level 2 of the IAEG Commission 25. The site is considered as a medium sized engineering development with several geohazards, and some potential constraints (Baynes & Perry, 2022).

The EGM is a culmination of information obtained from our desktop review using aerial images from 1943 - 2003, an understanding of the evolution of the Hutt Valley as a fault angle basin (Begg & Mazengarb, 1996), subsurface investigations (during and prior to ENGEO's investigations), and ENGEO's site walkover and geomorphological investigation.

Our mapping and investigations are generally in line with the regional geological setting. However, recent geomorphological processes have overprinted the basic terrace morphology in a number of locations as described in the following section.

6.2 Geomorphology

Alluvium

The site today resides on an alluvial terrace which is elevated above the surrounding Hutt Valley, some of which is likely attributed to uplift across the Wellington Fault, which is located southeast of the site. A lower sub-terrace is present at the north-eastern end of the primary terrace, which is approximately half the height of the primary terrace.

Several terrace systems are present throughout the Hutt Valley region and are well constrained to the Late Pleistocene / Early Holocene, a transition which occurred approximately 11,700 years ago.

Based on evidence of similarly aged terraces found to the north, along with visual observations, these alluvial gravels typically overly Greywacke Sandstone. Since the formation of the terrace which the site resides on, the Hutt River has since eroded back into and cut the lower (younger) Holocene terrace. Remnants of the lower terrace are evident in the slight shallowing of contours along the south-eastern terrace riser.

A hydrological change, likely during the Holocene, diverted the Hutt River further east, away from the slope, such that the river now does not erode the toes of the primary terraces. There is evidence in the historical aerial images to suggest that there have been periods when the Hutt River has directed its course somewhat back to the base of the terrace during flood events, however, it does not appear to have resulted in any significant erosion.

Colluvium / Landside Debris

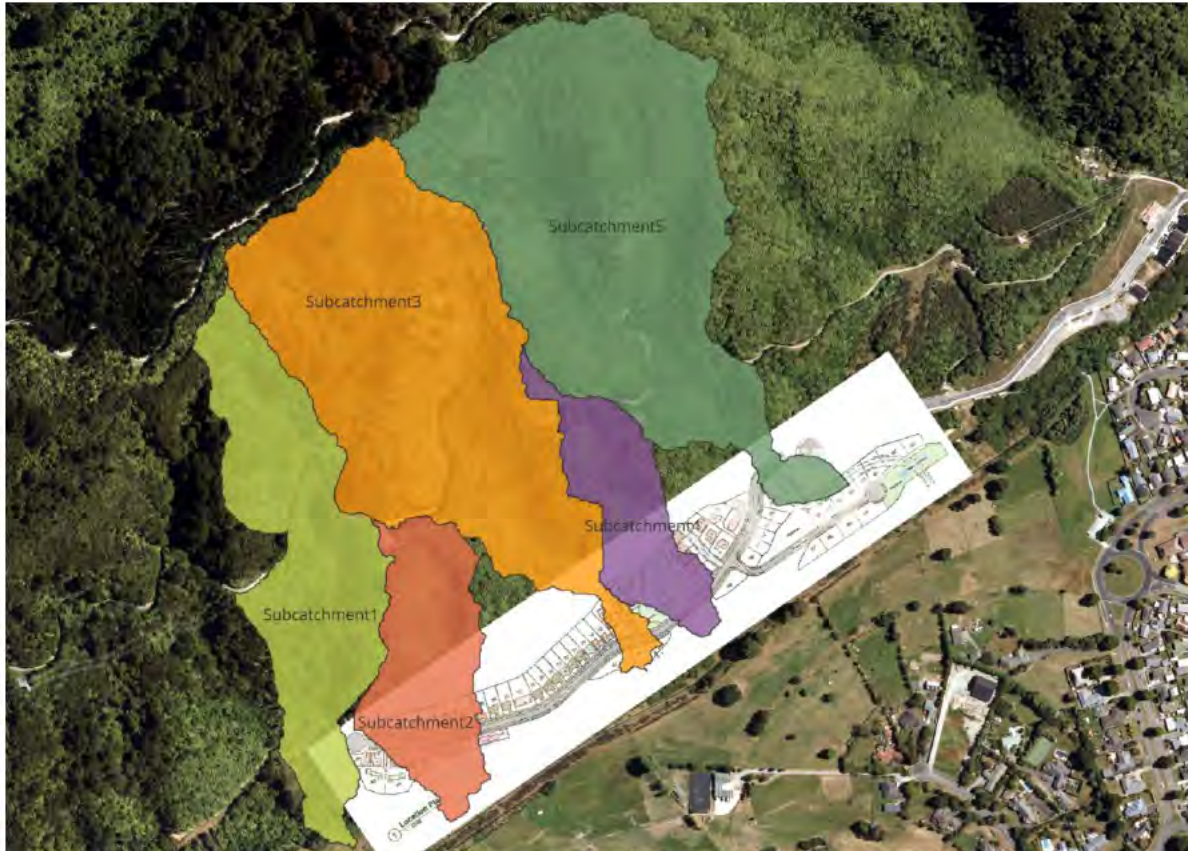
Following, and likely during the aggradation of the primary terrace, the slopes above the site would have been actively distributing material in the form of shallow landslides and erosion onto the terrace. Colluvial / Landslide material is evident in small slumps and fanlike structures at the base of the slopes, however, these areas are now well vegetated. There is little evidence to suggest that there has been recent landslide activity on Stage 3. The only recent documentation is observed in the GNS Landslide Database described in Section 4.6.

Debris Flows / Streams

Large fanlike structures are distributed from the terminations of five main catchments located to the north-western above the site (in text Figure 6). These catchments are of variable size and slope angle, with variable amounts of debris and incision within their primary channels. We infer that the structures at the base of the catchments represent debris fans, most likely derived from repeated, small scale (0.4 – 0.7 m thick) debris flows which have originated within the upper parts of the catchments. Evidence of previous debris flows were observed within the incised channels 10 – 15 m inside the apex of the gullies, furthermore, a test-pit undertaken within one of the debris fans (STG3-TP04), revealed angular to sub-rounded gravely fine to coarse sand.

Small streams are evident within the gullies, however, are absent from the terrace bench. A wetland in the centre of the site represents the accumulation area of the two central streams. A distinct channel is present, which drains the wetland to the southeast, however the baseline level of the stream was low, and the ground immediately surrounding was saturated. Minor farm modifications along the crest of the slope appear to be limiting the flow of water off the site.

Figure 6: Catchment Areas Located Above Stage 3



Rock Fall

Some evidence of rockfall is present at the base of the slopes, generally northwest of the farm track, with dimensions between 50 – 200 mm. However, these are limited to isolated areas and not found along the length of the site. Most of the visible rockfall was lichen covered or surrounded by vegetation.

6.3 Groundwater

During ENGEO's intrusive investigations groundwater was only observed within STG3-TP04, which was located at the apex of the largest catchment, sub-catchment 3. Groundwater here was observed seeping through the side of the test-pit excavation at a depth of 1.4 m bgl.

However, given the free-draining granular nature of material which the terrace has been established with, groundwater generally appears to be deeper than what was measured in our investigation. RDCL's investigation extended to depths of 15 m bgl and the water table was not encountered.

6.4 Identified Geohazards

The EGM identifies debris flows, slope instability of the crest of the terrace riser and the slope above the site, and rockfall to be the primary geohazards posed to the site.

These hazards are discussed in greater detail in the following sections, and levels of risk in accordance with the National Policy Statement for Natural Hazards are outlined in Section 11.

6.5 Assumptions and Uncertainties

Given the lateral and vertical elevation of the site (primary and sub-terrace), we have developed our EGM based on the assumptions that all terraces were likely formed under similar geomorphological processes. We have assumed that the terraces which the site is proposed to be established on were formed during a similar period as those formed throughout the Upper Hutt Valley. Based on observations around the site, and documentation of terraces of similar age, we have assumed that the terrace was established directly overlying Greywacke sandstone.

Due to the densely vegetated nature of the slopes immediately above the site, it was not possible to access, or view, the slopes above the site to determine whether active landslides, or rockfall sources were present above the site.

6.6 Engineering Geological Units

Based on our site investigations the two primary Engineering Geological Units which are described with their relevant *in situ* test data in Table 2.

Table 2: Engineering Geological Units

EGM Unit	Test ID	Description	In situ Testing SPT N, DCP	
			M bgl	In situ results
Fill	Previous ENGEO TP04	SILT with some sand and minor gravel	0 – 2.0	DCP (2 – 15+ blows/100 mm)
Holocene Debris Flow Material	STG-TP01 (ENGEO)	Gravelly fine to coarse SAND with minor silt.	0.15 - 2.5	DCP (4 – 15+ blows/100 mm)
	STG3-TP04 (ENGEO)	Fine to coarse GRAVEL with some cobbles, some sand and minor silt.	0.25 – 2.6	DCP (2 – 15+ blows/100 mm)
Pleistocene / Holocene Alluvium	STG-TP05 (ENGEO)	Cobbly fine to coarse GRAVEL with some sand and minor silt	0.5 – 2.5	DCP (3 – 15+ blows/100 mm)
	BH07 (RDCL)	N/A (drillers logs) ¹	1.1 – 15.0	SPT undertaken at 1.5 m intervals starting at 1.5 m and finishing at 15 m. All SPT results uncorrected obtained 50+
	BH08 (RDCL)		1.5 – 15.0	SPT undertaken at 1.5 m intervals starting at 1.5 m and finishing at 15 m. All SPT results uncorrected obtained 50+
Greywacke Sandstone ³	BH06 (RDCL)	Highly weathered Greywacke sandstone ²	5.6 – 9.0	SPT 6.0 m: 25/25 for 125 mm (N=50+)

¹ Drillers logs have not been included as no geotechnical professional logs have been provided.

² As stated by Cohesive Consulting Limited, no ENGEO investigations encountered Greywacke Sandstone.

³ RDCL's BH06 was located north of the proposed development but importantly shows the alluvial gravels overlying.

7 Analysis

We consider the following geotechnical aspects are required to be assessed to evaluate the safe and economic development of the site:

- Seismic Hazard.
- Liquefaction and lateral spreading.
- Undocumented fill and historical landform modification.
- Slope stability including rockfall.

- Earthworks requirements including preliminary foundation recommendations as well as erosion and sediment control

7.1 Seismic Hazards

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 termed surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, and rockfall.

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

7.1.1 Ground Rupture

As cited in Section 4.3, the site is located approximately 250 m from the Wellington Fault. At this distance, the likelihood of co-seismic ground rupture and ground lurching are low; however, the southernmost section site lies within the uncertain constrained mapped extent, and therefore we cannot exclude ground rupture as a possibility.

Given the tectonic setting, regional subsidence or uplift may be experienced following a large earthquake on any of the faults in the area. In seismically active areas there is always a likelihood 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.

7.1.2 Ground Shaking

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 drive the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation. Given the proximity of the site to the Wellington Fault which has produced large single event displacements, it is likely that the site will experience a high degree of ground shaking.

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

Through discussions with CPDL, we understand the potential site development will be limited to Importance Level 2 (IL2) structures. Peak horizontal ground accelerations (a_{max}) for use in geotechnical assessments are provided in Table 3. A_{max} values have been taken from the recommended values provided in Table A1 - Appendix A of MBIE/NZGS Module 1 for Upper Hutt.

Table 3: 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 Liquefaction and Lateral Spread

For soil to be considered liquefiable, it is required to be uniformly graded (typically fine grained), have a low consistency / density when measured *in situ*, be located below the water table, and be sufficiently close to the surface that a non-liquefiable crust cannot be penetrated.

The site is largely dominated by gravels and other well graded materials, which are typically moderately dense to very dense from approximately 0.5 m depth. Importantly, the water table is considerably below the site level (> 15.0 m bgl). Previous investigations, in particular the RDCL boreholes have shown that sandy silt is present at depth; however, SPT values indicate that this material is dense to very dense and above the water table.

There is an area adjacent to the wetland where groundwater was measured to be 1.4 m bgl; however, this is likely due to proximity to the stream, and wetland area, and does not reflect the general site groundwater conditions. We therefore consider the likelihood of liquefaction (and by extension, lateral spreading) to negligible.

7.3 Terrace Slope Stability Assessment

Based on our desktop study, review of historical aerial imagery, site observations and investigations, and review of the proposed development plans, we consider that slope instability is likely to be a hazard at the site for future development, which will straddle the crest of the terrace riser. As it is currently proposed, lots and roads are proposed to be positioned along the crest, and in doing so, filling will be undertaken to thicknesses between 3 and 5 m. Furthermore, above the site is a series of large slopes and ridgelines.

We evaluated the potential stability of the terrace crest after the subdivision development by undertaking a series of non-circular Limit Equilibrium analyses, using Slide2 modelling software from Rocscience. In Limit Equilibrium analysis, the stability of a slope is expressed as a 'Factor of Safety' (FoS) which is defined as the ratio of the forces resisting slope movement to the forces causing slope movement. Theoretically increased likelihood of failure of a slope occurs with decreasing FoS values below 1.0, while increasing values above 1.0 indicate improving stability.

Depending on the level of certainty around the various input parameters in the ground model, the following minimum FoS are typically adopted:

- FoS > 1.5 for slopes under normal conditions.
- FoS > 1.2 for elevated groundwater and SLS seismic conditions.
- FoS > 1.0 for Ultimate Limit State (ULS) seismic conditions.

7.3.1 Sections Analysed

Four sections were analysed across the proposed Stage 3 development with locations as shown in Appendix 6. Sections were analysed as they are proposed to be developed in accordance with 26014 Cannon Point Fast Track STAGE 3 drawings, dated 13 February 2026.

Section A: Located at the south-western most point of the site, Section A intersections Lots 37, 39, and 40, and is oriented perpendicular to the terrace riser below the development, and the large slope above.

Section B: Located at approximately chainage 1220 of the main road coinciding with Lots 27 and 28 is a critical section of roading infrastructure where a gully is proposed to be filled. A yet to be determined retaining solution will be required for the proposed 5 m of fill that is to be placed.

Section C: Located immediately northwest of the wetland area, and intersect Lots 7 to 12, which are proposed to be cut into the established debris fan.

Section D: Located at the north-eastern most end of the development on the lower terrace, has a slope / terrace riser above and below the proposed Lots 60 and 64.

7.3.2 Upslope Instability

Across Stage 3 there is evidence of multiple shallow landslides along the base of the north-western slopes, and along the crest of the main terrace riser. These are typically occurring in steeper sections of the hill slope. As there is no evidence of deep-seated evaculative failures on the slopes above the site, it is likely that the rock underlying the colluvial face limits the extent of the failure. However, there have been previously documented failures in the slope above, outlined in Section 4.6.

The slopes and visible shallow landslides are well vegetated, indicating that failure occurred some time ago. Review of historical aerial imagery from as early as 1943 (Section 4.4) does not show any evidence of slope instability.

7.3.3 Assumptions and Limitations

We assume that all proposed dwellings are Importance Level 2 (IL2) structures with a design life of 50 years.

We have assumed a relatively uniform thickness of colluvial cover on the face of the slopes above the site, which are underlain by a moderately to highly weathered Greywacke sandstone.

When assessing slope instability risk to structures that are located at the top of slopes in our analysis, we have discounted failures that occur on the slope surface only and do not extend onto the flat area at the top of the slope.

7.3.4 Groundwater

Groundwater has been modelled as a piezometric surface under long-term drained conditions. Under short-term conditions the upper 1 m was modelled as saturated wet front.

7.3.5 Geotechnical Material Properties

The following geotechnical parameters have been adopted in our analyses. These parameters for design 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.

The material parameters used for the alluvial gravels, particularly the friction angle and cohesion, are derived from testing undertaken within the last 12-months from a similarly aged terrace within the Hut Valley. The values presented below still represent a conservative estimate when compared to the true values obtained from our previous works.

Table 4: Geotechnical Parameters

EGM Unit	Unit Weight (kN/m ³)	Strength Type	Effective Stress Parameters		USC (kPa)	GSI	mi
			Ø' (°)	c (kPa)			
Fill Material	18	Mohr-Coulomb	32	2	N/A		
Colluvium / Debris Flow	17	Mohr-Coulomb	30	5			
Saturated Colluvium	18	Mohr-Coulomb	30	1			
Alluvial Gravels	21	Mohr-Coulomb	35	40			
Saturated Alluvium	23	Mohr-Coulomb	35	10			
Completely Weathered Greywacke Sandstone	20	Mohr-Coulomb	32	5			
Highly Weathered Greywacke Sandstone	21	Hoek-Brown	N/A		15,000	30	7
Moderately Weathered Greywacke Sandstone	24	Hoek-Brown			75,000	45	7

7.3.6 Seismic Design Conditions

We have used the following information provided in Table 5 for our calculation of the design seismic ground acceleration. Earthquake magnitudes and ground accelerations can be obtained from Appendix A of Module 1: Earthquake Geotechnical Engineering Practice 2021.

Table 5: Design Seismic Ground Acceleration

Parameters	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)
Design Life	50 years	
Importance Level	2	
Annual probability of exceedance for design earthquake	1/25	1/500
Peak Ground Acceleration (PGA)	0.13 g	0.68 g
Topographic amplification factor	1.0	
Wall Displacement Factor (Wd)	N/A	
Design ground Acceleration (PGA)	0.13 g	0.68 g

7.3.7 Analysis Results

We have assessed slope stability under several cases as presented in Appendix 6. Results are summarised within Table 6 and Table 7 below.

Discussion of the results is provided in the following.

Table 6: Slope Stability Results for Slope Above the Site

Example Heading	Scenario	Required FoS	Calculated FoS	Acceptable?
Section A-A'	Static	1.5	1.0	No
	Raised Ground Water + Wet Front	1.2	0.6	No
	SLS	1.2	0.8	No
	ULS	1.0	0.3	No
Section B-B'	Static	1.5	0.8	No
	Raised Ground Water + Wet Front	1.2	0.6	No
	SLS	1.2	0.7	No
	ULS	1.0	0.3	No
Section C-C'	Static	1.5	1.0	No
	Raised Ground Water + Wet Front	1.2	0.9	No
	SLS	1.2	0.8	No
	ULS	1.0	0.4	No
Section D-D'	Static	1.5	1.4	No
	Raised Ground Water + Wet Front	1.2	1.0	No
	SLS	1.2	1.1	No
	ULS	1.0	0.5	No

Table 7: Slope Stability Results for Slope Below the Site

Example Heading	Example Heading	Example Heading	Calculated FoS	Acceptable?
Section A-A'	Static	1.5	1.7	Yes
	Raised Ground Water + Wet Front	1.2	1.5	Yes
	SLS	1.2	1.4	Yes
	ULS	1.0	0.7	No
Section B-B'	Static	1.5	2.1	Yes
	Raised Ground Water + Wet Front	1.2	1.9	Yes
	SLS	1.2	1.7	Yes
	ULS	1.0	0.8	No
Section C-C'	Static	1.5	1.9	Yes
	Raised Ground Water + Wet Front	1.2	1.3	Yes
	SLS	1.2	1.5	Yes
	ULS	1.0	0.8	No
Section D-D'	Static	1.5	2.0	Yes
	Raised Ground Water + Wet Front	1.2	1.4	Yes
	SLS	1.2	1.5	Yes
	ULS	1.0	0.7	No

Static Condition

In Sections A, B, and C, above the site, a Factor of Safety (FoS) around and below 1.0 was achieved under static conditions, indicating that the slope is marginally stable. However, the minimum FoS for long-term stability (1.5) has not been achieved. Our site walkover did not identify any areas of creeping soil or recent instability that would indicate that the slope is able to maintain its existing slope angle under normal conditions. Meanwhile, Section D achieved a minimum FoS value of 1.41, which is below the typically required minimum, but is likely to have an acceptable level of stability under static conditions.

In Sections A and C, below the site, the minimum FoS of 1.5 has been achieved or exceeded, which indicates that the slopes are likely to have an acceptable level of stability under long-term static conditions.

Raised Groundwater Conditions

Our analysis of the slopes above the site under “raised groundwater” conditions, show that failure planes are localised within the surficial 0.5 – 1.0 m of saturated material (the wetted front), where a FoS values less than the minimum requirement of 1.2 is achieved.

In cases where saturation induced failures occur, we consider that the most likely scenario that failures are to occur during or following prolonged heavy rainfall. By way of example a 100-year Annual Recurrence Interval (ARI) over 72-hours will cause a higher degree of saturation than a 100-year ARI over five hours. Furthermore, it is typical for saturation induced failures to occur when a relatively wet period has preceded a large rainfall event by days or weeks.

In the slopes below the site, the “raised groundwater” condition failure plans similarly display surficial failures, typically 0.8 m in depth; however, achieve and exceed the minimum FoS requirement of 1.2. These higher FoS values suggest that while surficial failures may occur under extreme erosional circumstances, the underlying stability of the terrace is not significantly compromised.

Seismic Conditions

Our analysis indicates that the minimum recommended FoS value under Ultimate Limit State (ULS) was not achieved in any slopes above or below the proposed development.

Above the site, FoS values achieved were below 0.5, and therefore do not meet the minimum requirement. This indicates that the stability within the colluvial slope, and possibly the more weathered Greywacke sandstone, is compromised when subjected to the design seismic loading conditions.

Below the site global failures are indicated to extend back from the crest of the terrace to varying extents. Accordingly, we have undertaken an assessment of the likely co-seismic displacement, with the intent of establishing a specific engineering design (SED) zone above the crest of the terrace riser. This analysis is described in the following section.

7.3.8 Seismic Displacement Analysis

During a ULS earthquake, the yield acceleration (the seismic acceleration at which the sliding forces equal the resisting forces), may briefly be exceeded, however, depending on the material type their strength, deformation may occur.

To assess the likely co-seismic displacement, we have made several assumptions, as detailed in Section 7.3.3, based on the information available to us. 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 soil and rock material is likely to vary across the site, and that changes in these parameters would likely alter our displacement estimates.

We have completed seismic displacement analysis using the methods of Bray and Macedo (2019), Rathje and Antonakos (2011), Anderson et al. (2008), and Macedo et al. (2023).

We have used the following input parameters where applicable for the various displacement analysis methods:

- Assumed V_{s30} of site – 400 m/s. V_{s30} is a parameter that represents the average shear-wave velocity in the top 30 m of the Earth's surface.
- Moment Magnitude (M_w) of 7.8 obtained from National Seismic Hazard Model (GNS Science) website, for 1/500-year earthquake event for Wellington with a V_{s30} as outlined above.
- Displacement estimates were obtained using moment magnitudes for shallow crustal earthquakes and subduction zone interface and intraslab earthquakes.
- Spectral acceleration obtained from National Seismic Hazard Model (GNS Science) website, using input parameters obtained from our slope stability analysis results.

Based on the average of the different displacement methods (outlined above), the following displacements are estimated for the slopes above the site (Table 8), and the slopes below the site (Table 9).

Table 8: Displacement ULS Estimates for Slopes Above the Site

Section ULS	K_y	Indicative Displacement Estimate	Expected Failure Mass Material Type ¹	Categorical Slope Performance ²
Section A	0.00	1000+ mm	Ductile	Severe Slope Movement
Section B	0.00	1000+ mm	Ductile	Severe Slope Movement
Section C	0.01	1000+ mm	Ductile	Severe Slope Movement
Section D	0.21	600 mm	Ductile	Major Slope Movement

¹ Ductile material can deform significantly under stress before fracturing whereas brittle material tends to fracture with little or no plastic movement.

² Slope performance category is based on the displacement estimate and material type of failure mass as per draft Unit 3 of the Slope Stability Guidance.

Table 9: Displacement ULS Estimates for Slopes Below the Site

Section ULS	K _y	Indicative Displacement Estimate	Expected Failure Mass Material Type ¹	Categorical Slope Performance ²
Section A	0.36	200 – 250 mm	Ductile	Large Slope Movement
Section B	0.48	150 – 200 mm	Ductile	Large Slope Movement
Section C	0.42	200 – 250 mm	Ductile	Large Slope Movement
Section D	0.39	250 – 300 mm	Ductile	Large Slope Movement

¹ Ductile material can deform significantly under stress before fracturing whereas brittle material tends to fracture with little or no plastic movement.

² Slope performance category is based on the displacement estimate and material type of failure mass as per draft Unit 3 of the Slope Stability Guidance.

Our analysis indicates that failures above the site are likely to result in large displacement events at peak ground accelerations lower than the 1/500-year seismic event. Based on Table 18 of Unit 3 'Slope Stability Guidance', Severe Slope Movement is possible. Severe Slope Movement is defined as 'damaging landslide movement where significant slope destabilisation is likely'. It is possible that during high yielding seismic events (ULS) complete evacuative failure may occur. Similarly, it is possible during high rainfall events partial or complete failure of areas above the site may occur. The slope stability analysis therefore reinforces the observations recorded in the New Zealand Landslide Database (Section 4.6), and observations of shallow failures around some of the slopes. Given this, we have assessed appropriate runout distances from the toe of the slopes above the site as outlined in Section 7.3.10.

In the slope below the site, estimated displacements are considerably lower. Our analysis suggest ULS displacements in Sections A to D to be in the range between 150 to 300 mm, which is classified as a 'Large Slope Movement' in Slope Stability Guidance Unit 3. The 150 – 300 mm displacement category, particularly for ductile material (soil) has received considerable attention in the literature. There is a commonly reported threshold at about 300 mm of displacement where movement in a ductile soil can either be accommodated by slip, or complete / evacuative failure (Anderson et al., 2008 & Blake et al., 2002).

We therefore consider a mitigation, such as a Special Engineered Design zone for the proposed lots along the crest of the terrace riser to be necessary, as discussed in the following section.

7.3.9 Crest Special Engineered Design Areas

Due to the risk posed by seismic displacement in the slopes below the site to the proposed residential houses, we consider it necessary that Special Engineered Design (SED) be included for lots located near the crest of the terrace riser.

Using the failure planes calculated in the computational slope stability analysis (Appendix 6), and morphology changes in the terrace risers, three SED zones have been developed, and are displayed in Appendix 7, and are outlined Table 10.

The terrace riser geometry and failure plane encroaching distances can be separated into three distinct areas along its length:

- In the southern extent, the terrace riser is steeper, and failure planes are modeled to extend between 20 to 22 m into the terrace surface from the crest and affect Lots 23 to 29, 38, 40, 41, and 44 to 46.
- The north-eastern half of the riser is slightly lower with a minor concave break in slope halfway up the riser, which approximates the height of the sub-terrace containing Lots 47 to 55. The modeled failure planes under ULS extend 10 m into the terrace surface from the crest.
- The lower sub-terrace, which appeared to be earthworked in the 1970's has a steep terrace riser geometry, which is proposed to have additional surcharge in the form of fill added to the tread. The result is a failure plane which extends 32 m from the crest of the slope, which will require Lots 56 to 67 to be within an SED zone.

Table 10: Specific Engineered Design Zones

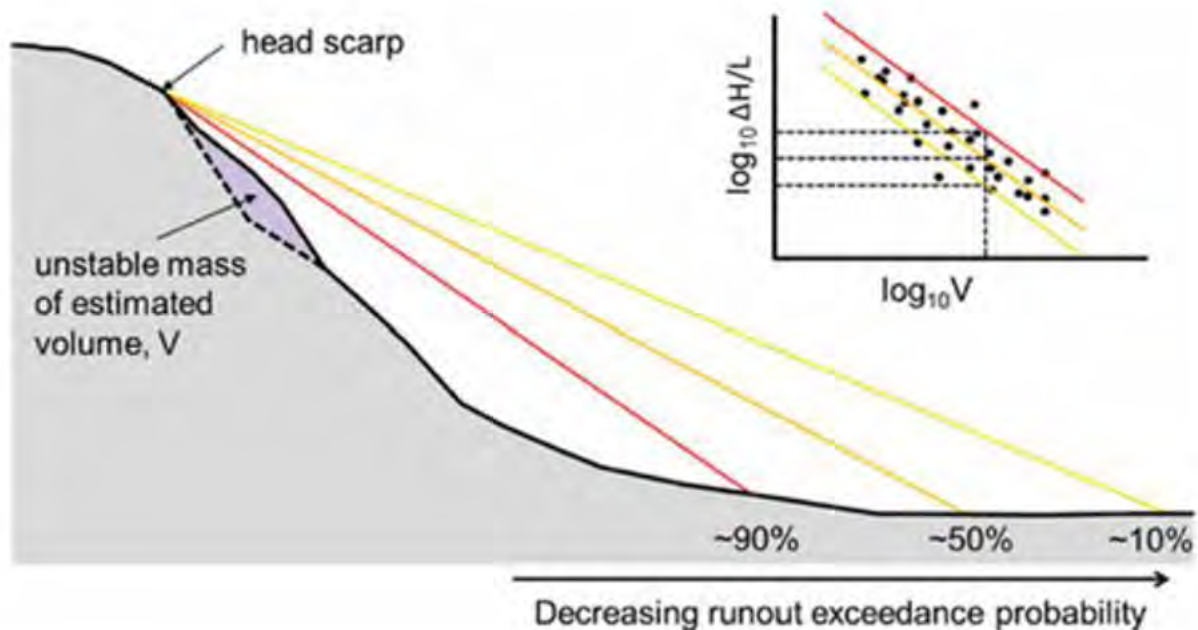
Zone	Distance	Affected Lots	Design Seismic-Induced Displacement
Zone A	10 m	47 to 55	300 mm
Zone B	21 m	23 to 29, 38, 40, 41, and 44	
Zone C	32 m	56 to 67	

7.3.10 Debris Flow Failure Assessment – Runout Distance

Possible runout distances from shallow failures upslope of the proposed subdivision have been assessed using shadow angle analysis. This is a simple methodology that uses a series of empirical relationships to determine landslide run out in relation to landslide volume, which has been displayed to be a function of fall height.

The method outlined in Brideau et al (2020) uses estimated volumes and a ratio of slope height over length to determine an angle for run out. This angle is then measured in the downslope direction from the estimated head scarp area, which is typically the highest point of a given slope, to an intersection point at the toe of the slope (as displayed in Figure 7). An angle appropriate to the assumed volume and state of saturation is used to determine the angle of intersection.

Figure 7: Schematic illustration of the Shadow Angle estimations for determining debris runout in (Brideau et al., 2020).



In our analysis we searched for slopes along northwest of the site which were capable of generating three possible volumes of landslides, 200 m³, 500 m³, and 1000 m³ (Note 1), and given the uncertainty in upslope soil profile we have assumed that at these slopes were underlain by thickness of 1.0 m of colluvium before intercepting rock; however, it is likely some amount of rock underlying the colluvium will be residual soil or completely weathered rock. Based on the assumptions above two angles have been projected in the identified slopes:

1. A 30° angle in a 25% probability exceedance scenario which has been calculated by averaging the runout distance for a soil which is partially saturated.
2. The second is a 25° angle with a 25% probability of exceedance, assuming that the material is completely saturated.

The angles selected represent the 500 m³ volume, due to the minimal angle change between the 500 m³ and the 1000 m³ volume this is considered acceptable.

The results of the shadow angle analysis are displayed in Appendix 8. This has been developed based on a 500 m³ volume, due to the minimal angle change between the 500 m³ and the 1000 m³ volumes.

¹ These volumes were adopted based on typical slope failures that occur in the greater Wellington hillslopes during and following historical rainfall events.

Without any mitigation measures, lots along the toe of the northwest slopes (Lots 3, 6, 7, 14, 15, 16, 18 to 37, 42, and 43) there is a 25% probability of debris runout extending beyond the areas displayed in Appendix 8. In the 25% exceedance area for completely saturated debris, Lots 1, 2, 4, 5, 8, 13, 17, 41, 44, and 45 become exposed to potential debris runout. Given the current uncertainty in the soil profile of the slopes above the site, the lots located within the 25% probability of exceedance for a mixture of saturated and dry soils would not be considered suitable for development without the implementation of mitigations.

7.3.11 Slope Stability Recommendations

Given the extent of area affected by both landslide runout from the slope above, and the SED requirements for the crest of the terrace riser along the length of the development, suitable engineered mitigations are required to reduce the hazard level / extent of coverage of both upslope and downslope hazards.

The SED zones are required to mitigate against significant damage within the duration of the building's design life. The SED buffer zones, which vary in distance from the crest between 10 to 32 m, must be designed to withstand seismic-induced ground displacement up to 300 mm without catastrophic failure. Outside of the SED zones, house structures and foundations can be built as per NZS 3604:2011.

For lots located downslope of the slopes above the site, geotechnical mitigations will be required to reduce the estimated slope runout. Prior to this, further investigation into the soil / rock profile will be required, in order to accurately assess the volume of material able to runout, after which time a suitable design option, of which bunds, catch fences (either isolated from retaining structures (i.e., constructed above and separately) or have the retaining structures design to withstand loading from both failure behind the wall and the slopes above), or soil reinforcements are options.

During ULS seismic events, or events of similar size, failures of a greater area to those of low frequency rainfall events (e.g., 1-100 to 1-500 year events) can occur, and in some cases result in total collapse of the slope face. This phenomenon was observed around Wellington and Wairarapa following the 1855 Wairarapa Earthquake (Hancox, 2005). The slopes above Stage 3, despite being steep, appear to be less likely candidates for earthquake induced catastrophic failure for a number of reasons.

- The hill slopes are separated into a series of small faces by a series of catchments that have incised the slopes above the site. The result is that each individual slope has a lower surface area, which ranges from 500–1500 m², with the exception of one slope which is approximately 3,000 m² (but is only proposed to be partially developed below); thereby is likely to have less material available for failure.
- The height of the slope is lower than that of those that typically failed during the 1855 Wairarapa Earthquake, and lower compared to the slopes surrounding the site where terraces are not present.
- There is no evidence across the terrace tread (which is believed to be approximately 11,000 years old) of previous large scale failures, even when historical aerial imagery is evaluated.
- The slopes above the site are well vegetated with some mature native vegetation (Pungas), which we understand take up to a century to grow to maturity.

The engineered solutions that will be designed to mitigate debris runout from rainfall induced landslides will likely be adequate to deal with possible seismically-induced landslides given the size of the slopes located above the site.

7.4 Rockfall

During the site walkover and geomorphic mapping ENGEO observed evidence of rockfall at the base of multiple catchments along the north-western boundary. Rockfall debris was observed in catchment 2, 3, 4, and 5 (geomorphological map; Appendix 4).

The observed debris typically consisted of Greywacke clasts up to approximately 200 mm, however, generally measured around 50 – 100 mm. With increasing distance from the toe of the slopes, the average clast size and number of clasts decreased. Where debris was observed, it was typically covered in moss, indicating that the last rockfall was some time ago. No large rocks were observed past the vegetation line, which we understand to be the approximate location of the proposed dwellings. Due to the dense vegetation cover above the site, it was difficult to determine the exact source location, however, the same vegetation is likely to reduce the rockfall hazard in the future.

We consider that the likelihood of rockfall debris impacting the proposed dwellings to be 'low'. Based on the observation outlined above. We consider the consequence is likely to be 'low' to 'medium', as any clast / boulder sizes are likely to be smaller, and therefore less damaging to the proposed dwellings.

7.5 Debris Flow Analysis

As discussed in Section 6.2, structures were identified onsite that represent debris fans, most likely derived from repeated, small scale (0.4 – 0.7 m thick) debris flows which have originated within the upper catchments. It is therefore considered that the predominant risk to the proposed subdivision development from debris flow is associated with sediment laden stormwater, rather than large scale soil mobilisation.

We used the computer program Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 6.6 published by the United States Army Corps of Engineers (USACE) to assess sediment laden flood / debris flow runout lengths, heights, and velocities. HEC-RAS is a computational fluid dynamics simulation software that is used to model Newtonian and non-Newtonian flow regimes.

7.5.1 Model Build

The baseline terrain used in the 2D hydraulic model was developed using the New Zealand LiDAR 1 m Digital Elevation Model (DEM) (Toitū Te Whenua – Land Information New Zealand [LINZ], 2025). A 10-m-by-10-m grid was overlain onto the terrain. A Manning's roughness 'n' of 0.07 was used as a default to represent the medium to dense brush in the hillside and site (Chow, 1959).

7.5.2 Rainfall

The 250-year annual recurrence interval (ARI) storm event was selected for modelling as it is broadly equivalent to a reasonable worst case release scenario described in Unit 6 of the NZGS Slope Stability Guidance.

A 2D rain-on-grid model was constructed for the site, which allows for evaluation of the surface flood response and mapping of areas that are temporarily inundated during the 250-year annual recurrence interval (ARI) 12-hour-duration rainfall event. Design rainfall for the 250-year ARI event was obtained from NIWA's [High Intensity Rainfall Design System V4](#) (2026). A climate change factor based on scenario RCP 8.5 for the period 2081 – 2100) was included.

Curve numbers (CN) and initial abstraction values were estimated in general accordance with Wellington Water's Reference Guide for Design Storm Hydrology (Cardno, 2019). Wellington Water has developed a [digital curve number map](#) for the Wellington region based on the underlying soil drainage characteristics. An initial abstraction coefficient of 0.1 was used because the catchment is undeveloped / vegetated.

7.5.3 Simulation Inputs

Within HEC-RAS, we used two non-Newtonian equations to simulate debris flow events. The Bingham equation was used to simulate a debris flow event with 30% volumetric solids concentration. The Bingham equation is suited to model transitional flows between floodwater and debris flows, with solid concentrations between 20-40%. The Bingham model is a function of the volumetric solids concentration (Cv) and yield stress, represented by the Mohr-Coulomb angle of internal friction.

The O'Brien equation was used to simulate a debris flow event with a 60% volumetric solids concentration. The O'Brien equation works well for ~60% Cv flows because, at these very high concentrations, debris flows are dominated by strong particle interactions that cause shear stress to increase non-linearly with shear rate. The O'Brien formulation explicitly includes this quadratic (shear-squared) behavior, making it more physically representative than linear models for dense, dilatant debris flows in HEC-RAS. The O'Brien model is a function of the volumetric solids concentration (Cv), yield stress, and representative grain size. A grain size of 15 mm was selected to represent the representative diameter of silt particles.

"Bulking" was applied to the debris flow events, which increased the volume of the runoff proportionally with the Cv specified. Bulking only controls flow volume, not the non-Newtonian physics.

The scenarios executed are shown in Table 11.

Table 11: Debris Flow Scenarios

Non-Newtonian Equations	Volumetric Solids Concentration (Cv)	Yield stress	Representative Grain Size	Bulked
Bingham	30%	35	NA	Yes
O'Brien	60%		15 mm	

7.5.4 Results

Maximum flood / debris flow depth and velocity values for each lot have been provided in Appendix 9; maps showing inundation depth and velocities are included in Appendix 10.

30% VOLUMETRIC SOLIDS CONCENTRATION SCENARIO

In general, the inundation depths and velocities from the 30% volumetric solids concentration scenario were limited to 0.35 m and 0.6 m/s, respectively. Exceptions are included in Table 12.

Table 12: Debris Flow Scenarios - 30% Volumetric Solids Concentration Scenario

Lot	Approximate Inundation Percentage	Maximum Inundation Depth (m)	Maximum Debris Flow Velocity (m/s)
9	<10%	0.1	0.8
13	<10%	0.9	0.7
15	<10%	0.35	1.3
17	<10%	0.05	0.9
18	<10%	0.25	0.7
19	10%	0.35	0.7
29	<10%	0.1	0.9
30	<10%	0.5	0.8
32	25%	0.4	0.4
36	10%	0.25	0.95
37	25%	0.7	1.5
57	10%	0.3	0.95
58	10%	0.6	0.9
59	20%	0.6	0.75

60% VOLUMETRIC SOLIDS CONCENTRATION SCENARIO

In general, the inundation depths and velocities reported for the 60% volumetric solids concentration scenario were limited to 0.55 m and 0.5 m/s, respectively. Exceptions are included in Table 13.

Table 13: Debris Flow Results - 60% Volumetric Solids Concentration Scenario

Lot	Approximate Inundation Percentage	Maximum Inundation Depth (m)	Maximum Debris Flow Velocity (m/s)
12	100%	0.65	0.3
13	25%	1.3	0.25
15	50%	0.65	0.60
27	50%	0.65	0.6
30	10%	0.75	0.25
37	15%	0.60	0.05
48	75%	0.60	0.35
54	25%	0.3	0.65
59	40%	0.7	0.55

7.5.5 Vulnerability Assessment

Unit 6 of the NZGS Slope Stability Geotechnical Guidance provides a framework for assessing vulnerability of property and infrastructure to damage by debris flow. Developed as a method of determining building damage from debris flows, Jakob et al (2012) define Intensity Index (IDF) for building damage. The IDF surrogates impact force and thus correlates with building damage. The flow depth and velocity are correlated to determine an intensity as shown in the equation below.

$$I_{DF} = dv^2$$

Where:

d = the maximum expected flow depth (m)

v = the maximum flow velocity (m/s).

The IDF value correlates to a Damage Class and Damage Class Probability, as shown in the excerpts from Unit 6 below.

Figure 8: Table 14 and 15 from Slope Stability Geotechnical Guidance (NZGS, 2026)

Table 14: Damage Class and definitions for impacts to residential buildings (modified from Jakob et al, 2012)

Damage Class	Damage Description	Typical I_{DF} (note 1)
Class I – Some Sedimentation	Sediment-laden water ingresses buildings main floor or basement, requiring renovation. Up to 25% insured loss	Less than 1
Class II – Some structural damage	Some supporting elements damaged and could be repaired with major effort; 25 – 75% insured loss	1 – 10
Class III – Major structural damage	Damage to foundation piles, pillars and will likely to require complete building reconstruction >75% insured loss	10 – 100
Class IV – Complete destruction	Structure is completely destroyed and/or physically transported from original location 100% insured loss	>100

Table 15. Damage Class Probabilities in research of Jakob et al (2012)

Damage class probabilities given I_{DF} (% ALL DATA)					
Class IV – Complete destruction	0	6	25	67	100
Class III – Major structural damage	0	22	38	28	0
Class II – Some structural damage	30	50	37	5	0
Class I – Some Sedimentation	70	22	0	0	0
I_{DF}	0-1	1-10 ¹	10 ¹ -10 ²	10 ² -10 ³	>10 ³

The IDF values were calculated for each lot (Appendix 9). The highest IDF values are associated with the lots included in Table 12 and Table 13 and are shown in Table 14.

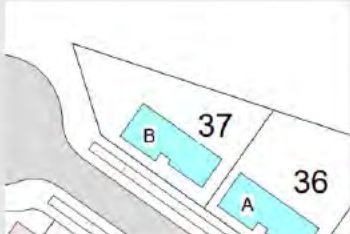
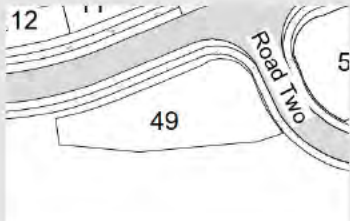
Table 14: Debris Flow Scenarios

Lot	IDF	Damage Class	Damage Class Probability
9	0.06	Class I – Some Sedimentation	70% - Class I – Some Sedimentation 30% - Class II – Some Structural Damage
12	0.06		
13	0.44		
15	0.59		
17	0.10		
18	0.12		
19	0.17		
27	0.23		
29	0.08	Class I - Some Sedimentation	70% - Class I – Some Sedimentation 30% - Class II – Some Structural Damage
30	0.32		
32	0.06		
36	0.23		
37	1.58	Class II – Some Structural Damage	22% - Class I – Some Sedimentation 50% - Class II – Some Structural Damage 22% Class 3 – Major Structural Damage
48	0.07	Class I - Some Sedimentation	70% - Class I – Some Sedimentation 30% - Class II – Some Structural Damage
54	0.13		
57	0.27		
58	0.49		
59	0.34		

7.5.6 Risk Assessment

The Damage Class and Damage Class Probability were used as a starting point for risk assessment for debris flow. It was concluded that lots with an IDF below 0.40 are firmly within the Class I range; additional assessment was not considered necessary. Lots with IDF values above 0.30 are included in Table 15. The architectural plans (dated 4 May 2026) were used to provide additional context for risk assessment.

Table 15: Risk Assessment Discussion

Lot	IDF	Location	Discussion
13	0.44		The architectural plans show a stormwater pipe running parallel to the southwestern lot boundary. A portion of the modelled inundation will be conveyed through the stormwater pipe, reducing impact to the lot. The model results show that significant inundation at Lot 13 is limited to the area within the no build zone. This reduces risk of property damage.
15	0.59		The architectural plans show a stormwater pipe running parallel to the northeastern lot boundary. A portion of the modelled inundation will be conveyed through the stormwater pipe, reducing impact to the lot.
37	1.58		The architectural plans do not show stormwater infrastructure near the lot; however, it is possible this will be included in the civil design. Lot 37 backs up to the toe of the mountain slope. The terrain indicates that a natural channel may be present within the site running parallel to the northern boundary. Inundation at the building platform is negligible. Debris flow mitigation may be required at the northern lot boundary.
49	0.49		No building development is proposed for Lot 49.

8 Recommendations

8.1 Preliminary Foundation Recommendations

Most of the lots proposed to be established along the crest of the terrace riser will require Specific Engineered Design (SED) such that foundations are able to withstand up to 300 mm of seismic-induced displacement. Options may include deep piled foundations or a hybrid system of a geogrid reinforced gravel raft with a slab foundation. The final choice of foundation type should be informed by lot-specific geotechnical assessment.

Our investigation identified that the sites soil profile is varied, with some location displayed dense to very dense gravels, while in other areas, loose to medium dense sands are present across the site. We recommend that each lot have a suitably qualified geotechnical assessment of the ground conditions undertaken prior to the detailed design of any foundations.

8.2 Bearing Capacity

Shallow foundations may be able to bear on the native sand, silt, and gravel material underlying the topsoil at variable depths between 0.5 and 0.9 m; additionally, founding within the proposed fill areas will be suitable given the earthworks methodology complies with NZS 4431. Lot specific testing will be required to determine the Ultimate Rupture Bearing Capacity, for individual lots.

Our previous investigation identified areas on the primary terrace tread and the sub-terrace tread where uncontrolled fill was present, these are largely within the proposed 32 m SED zone. In areas where fill is present the client may wish to have this removed and backfilled with suitable material, otherwise piling through the fill may present a suitable option for founding (however will also need to comply with the SED requirement).

8.3 Earthworks

The planned earthworks comprise a total area of 35,400 m², with 17,070 m³ of cut, 25,830 m³ of site-won fill placement, and 8,760 m³ of additional imported fill, totaling 34,590 m³ of fill. A maximum cut and fill height of 6.0 m is proposed in the master plans.

ENGEO has reviewed the proposed earthwork plans and understands that there are several geotechnical design which will require detailed design in order to make some lots viable, particularly along the north-western boundary of the development.

8.3.1 Topsoil & Vegetation

All topsoil should be stripped and stockpiled away from the earthworked areas.

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

8.3.2 Groundwater

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 covered in the earthworks management plan.

Free flowing water must be kept away from fill areas.

ENGEO should be given the opportunity to review any changes in the final earthworks plans prior to works commencing.

8.3.3 Filling

All earthworks' activities must be carried out in general accordance with NZS 4431 under the guidance of a suitably qualified and experienced Chartered Professional Geotechnical Engineer who is familiar with bulk earthworks.

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

Silt, sand, angular and rounded gravels were encountered across much of the site, with isolated areas of organic rich material and uncontrolled fill. ENGEO has not undertaken, and at the time of this report is not aware of any fill suitability testing (Particle Size Distributions or Compaction Curves).

Based on ENGEO's experience with similar material, and observations that were made during our investigations, it is likely that large portions of the material encountered on the site (given the appropriate processing) will be suitable for re-use. Compaction testing and Particle Size Distribution testing will be required before final comments can be made.

We recommend that temporary fill batters are formed no steeper than 1V:2H (26° above horizontal). Permanent fill batter angles will be confirmed in the Project Earthworks Specification after all required laboratory testing has been completed, and specific slope stability analyses have been carried out. Fill batters greater than 8 m in vertical height will require site specific design to mitigate seismic displacements.

8.3.4 Cuts

All cut batters should be protected from excessive rain during construction to minimise erosions and destabilisation. All permanent cut batters should be protected by planting and / or hydroseeding.

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

Catch fences / barriers may be required at the bottom of cuts greater than 4 m into the toe slopes to protect structures, roads, or workers from rock fall or possible landslide debris.

Cut batter angles for permanent and temporary slopes are to be determined in the next stage of consenting.

8.4 Retaining Walls & Soil Slope Reinforcing

Several retaining walls and reinforced soil structures are proposed in the current earthwork' plans, and should consider the effects of any slopes above, or below the wall. The plans indicate retaining structures between 2.5 and 6.5 m to create both building platforms and increase the developable space behind dwellings through excavation and retaining of the toe of the slopes above the site. All retaining wall will require site specific investigations and detailed design to account for the geohazards summarised in Section 9.

Timber post or cantilever walls will likely be acceptable for the terracing and elevating of building platforms across the subdivision. Where substantial cuts are proposed, such as the proposed 6 m retained heights behind Lots 35 to 37, steel post or anchor shotcrete systems may be required. Furthermore, these areas may require isolated engineered catchfence structures above to limited debris to the proposed lots below large cuts along the north-western extent of the site.

In areas where roading is proposed to span existing valley / gully systems, a retained heights up to approximately 5 m are proposed. In these areas Mass Stabilised Earth (MSE) walls may be suitable. These could comprise a near vertical face, with compacted fill and geogrid placed behind. However, a tied back steel post wall may also be suitable.

Where the retaining structure is greater than 3 m, we recommend that the seismic design of retaining walls is carried out in accordance with MBIE & NZGS's Module 6: Earthquake resistant retaining wall design. In particular, the module allows for a reduction in design ground acceleration due to a wall's acceptable deformation (see Figure 4.1 and Table 5.2 of the Module, noting that there is no reduction for stiff walls). The module also requires an allowance for topographical amplification.

8.5 Stormwater

The civil plans provided by Vecta Limited depict a series of culverts which can allow water captured in the catchments above the site to pass under the proposed development and be discharged below the site. Where possible stormwater should be discharged at the base of the terrace riser as to not erode the slope face.

We understand that access to the site will be provided by an retaining wall structure with a 10 m wide culvert at its base to allow for continued stream flow.

Stormwater collected from within the site catchment and the proposed dwellings will be conveyed via pipes beneath Road 1 to the sub-terrace. There, it will be discharged into a rain garden, where a network of collector pipes directs the stormwater to the base of the terrace riser. An overland flow path is depicted as an emergency spill way. This should be lined with a suitably sized rip-rap, as to not erode the terrace riser and undermine the rain garden.

8.6 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.

This is particularly relevant for the site due to the nature of the catchments above the site and the presence of the wetland on the site, which will require appropriate erosion and sediment control measures to ensure little sediment contamination occurs.

All service trenches should be capped with low permeability materials, so that excavations do not become points of entry for surface run-off. We recommend maintenance of the stormwater network be performed annually and after major storm events to keep the channels and culverts free of sediment and debris.

8.7 Debris Flow

The results of debris flow modelling suggest that some form of debris flow mitigation may be required along the northern boundary of Lot 37. Stormwater and / or sediment-laden water should be captured at the toe of the slope and directed toward a culvert.

Based on the model results, we recommend that the stormwater system design should be designed to allow for debris flow inundation; however no additional engineered mitigations are considered necessary.

9 Safety in Design

ENGEO considers the items outlined in Table 16 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 16: Safety in Design

Risk	Risk Rating	Potential Control Measures
Large Cuts > 6.0 m	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. Implement safe construction methodology to lower risk to workers. Do not work above or below large cuts immediately following heavy rainfall (should consider geotechnical assessment following heavy rainfall).
Steep Slopes (above and below the site)	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.
Streams and standing water	Moderate	Silt fences constructed around stream channels and wetland areas to reduce thoroughfare. Stay away from channels during elevated rainfall. Complete a comprehensive water management plan prior to undertaking earthworks.

10 Sustainability

Geotech

We encourage you to consider sustainability when assessing the options available for your project. Where suitable for the project, we recommend prioritising the use of sustainable building materials (such as timber in favour of concrete or steel), locally sourced (materials readily available to Contractors as opposed to materials requiring import), and installed in an environmentally friendly way (e.g., reduced carbon emissions and minimal contamination).

If you would like to discuss these options further, ENGEO staff are available to offer suggestions.

11 Section 106 Assessment

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 under both, Section 106 of the Resource Management Act considering the National Policy Statement for Natural Hazards 2025 (approved under Section 52 (1) of the Resource Management Act 1991). This requires a combined hazard assessment approach to be considered, where the likelihood of the hazard occurring, the material damage (consequence) of the hazard, and whether the proposed modification of the land will accelerate or worsen the hazards.

The National Policy Statement' Appendix 1 risk matrix allows the assessment of natural hazards (risk matrix replicated in Figure 9 below).

Figure 9: National Policy Statement for Natural Hazard Risk Matrix

Consequence Level	Likelihood Level						
	Almost Certain	Very Likely	Likely	Possible	Unlikely	Rare	Very Rare
	ARI (years) up to 10	10-20	20-50	50-100	100-500	500-5000	>5000
AEP	10% or more	10% to 5%	5% to 2%	2% to 1%	1% to 0.2%	0.2% to 0.02%	<0.02%
Catastrophic	Very High	Very High	Very High	High	Medium	Medium	Medium
Major	Very High	Very High	High	High	Medium	Medium	Medium
Moderate	High	High	High	Medium	Medium	Low	Low
Minor	Medium	Medium	Medium	Medium	Low	Low	Low
Negligible	Low	Low	Low	Low	Low	Low	Low

Replicated from National Policy Statement for Natural Hazards 2025.

We have assessed the natural hazards in Section 7 of this report and have put together a summary, in Table 17, to determine based on the above-mentioned methodology, whether the development is likely to increase the likelihood of the hazards occurring.

Table 17: Natural Hazards Risks

Hazard	Consequence / Assessed Risk	Proposed mitigation measure(s)	Risk following mitigation	Are the works likely to accelerate or worsen the hazard?	Additional Comments
Liquefaction	Ground settlement causing damage to dwellings and infrastructure Low	None	Low	No	During our investigation no potentially, liquefiable material was identified below the groundwater table (see discussion in Section 7.2)
	Lateral Spreading causing damage to dwellings and infrastructure. Low				
Slope instability above and below the site	<u>Lots Above Slope:</u> Undermining foundations / collapse of building from slope below. Seismic-induced differential settlement across foundations. Medium (given reoccurrence interval of Ultimate Limit State seismic event)	Proposed Lots along crest of the slope have foundations capable of withstanding seismic-induced displacement of up to 300 mm (Specific Engineer Design).	Low	Our analysis has been undertaken factoring in the addition of fill and a load approximating a lightweight structure, and suitable stability has been achieved to ULS.	See discussion in Section 7.3 on slope stability.
	<u>Lots Below Slopes:</u> Inundation / destruction of property from evacuated material from the slope above High	Increase stability of the crest of the slope. Catch fences below the upslope faces, or reinforcement of slopes.	Low	Possibly given the crest of the slope if proposed to have additional surcharge from fill and houses placed above the crest.	Stability of the slopes above and below the site are discussed in Section 7.3. The works required to reduce the risk following mitigation will be extensive.

Rockfall	Damage to property Low	Not Applicable / Rockfall catch fence.	Low	No	The average size of rockfall is small, and isolated to a couple of areas (see Section 7.4)
Debris Flow	Sediment inundation of buildings and stormwater system. Structural damage is unlikely. Low	The culvert and stormwater system proposed to be established as part of the development will manage and divert water away from lots close to the apex of primary gullies.	Low	No	The apex of the gullies is well vegetated and display little recent evidence of debris flows (further discussion in Section 7.5)

12 Conclusions

We conclude that, although there are considerable risks posed to the site now, the risk to the proposed buildings from natural hazards will be able to be reduced to levels less than 'significant' (i.e. Low) once the proposed works and suitable proposed mitigation measures, as described in this report have been designed and constructed.

12.1 Future Work

There are two areas we believe undertaking further work improve our understanding and therefore reduce the level of conservatism that has been required to be adopted in preparing this report.

1. Earthquake induced slope instability along terrace edge. We propose to undertake four boreholes to rock, and within such boreholes undertake two triaxial tests to more accurately assess the material parameters suitable for use in our analysis.
2. Slope failure above the subdivision. We have estimated a thickness of soil which may be present above the rock surface. The thickness heavily influences the possible volumes of material which runout analysis shows extending across significant parts of the subdivision. The hand excavations will also provide valuable information in the detailed design of landslide mitigations for lots proposed to be developed along the north-western boundary of the site.

General items:

- ENGEO or a suitably qualified Geotechnical Engineer with earthworks experience to review the final earthworks drawings prior to commencement.
- A suitably qualified ground-engineering professional shall be engaged to undertake construction monitoring during the earthworks.

- Lot specific investigations should be undertaken for foundation recommendations within SED zones.
- A suitably qualified geotechnical engineer should be consulted for the design of landslide runoff mitigations along the northwest of the site.

13 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, Cannon Point Developments Limited, their professional advisers, and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.
- ii. The recommendations in this report are based on the ground conditions indicated from published sources, 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 financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground 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



Kees Jansen

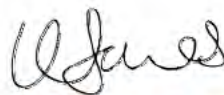
Engineering Geologist

Report reviewed by



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Principal Engineering Geologist



Karen Jones, CEnvP, CMEngNZ (PEngGeol)

Principal Engineering Geologist

14 References

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Appendix 1:

Cannon Point Stage 3 Master Plans

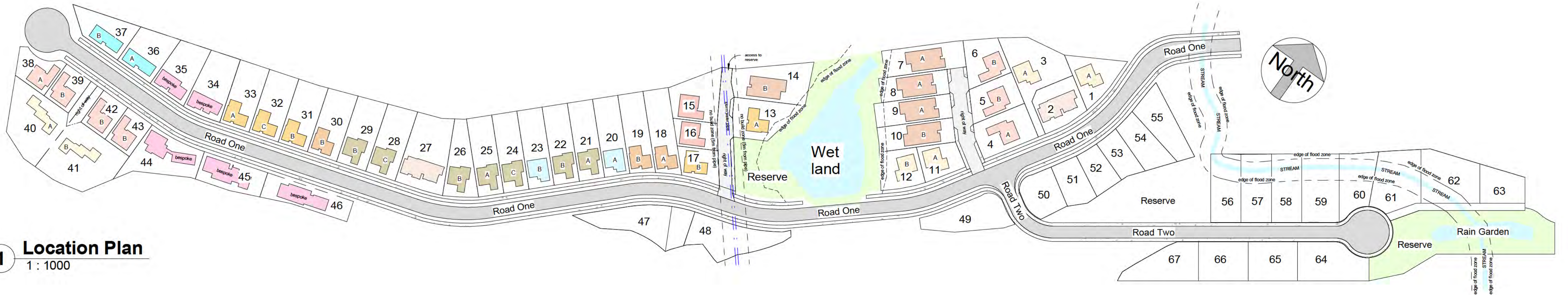
Drawing List		
Sheet Number	Sheet Name	Sheet Issue Date
000	Cover Page	04/05/2026
A101	Location Plan	04/05/2026
A102	Street Elevations	04/05/2026
A103	Street Elevations	04/05/2026
A104	Street Elevations	04/05/2026
A105	Lot 1 - Lower	04/05/2026
A106	Lot 1 - Upper	04/05/2026
A107	Lot 2 - Plan	04/05/2026
A108	Lot 3 - Lower	04/05/2026
A109	Lot 3 - Upper	04/05/2026
A110	Lot 4 - Lower	04/05/2026
A111	Lot 4 - Upper	04/05/2026
A112	Lot 5 Lower	04/05/2026
A113	Lot 6 Plan	04/05/2026
A114	Lot 7 & 8 Plan	04/05/2026
A115	Lot 9 & 10 Plan	04/05/2026
A116	Lot 11 & 12 Lower	04/05/2026
A117	Lot 13 & 14 Lower	04/05/2026
A118	Lot 13 - Upper Floor	04/05/2026
A119	Lot 15 - 16 Plan	04/05/2026
A120	Lot 17 - 19 Plans	04/05/2026
A121	Lot 20 - 22 Lower	04/05/2026
A122	Lot 20 - 22 Upper	04/05/2026
A123	Lot 23 - 26 Lower	04/05/2026
A124	Lot 23 - 26 Upper	04/05/2026
A125	Lot 27 - Plan	04/05/2026
A126	Lot 28 - 30 Lower	04/05/2026
A127	Lot 28 - 30 Upper	04/05/2026
A128	Lot 31 - 33 Lower	04/05/2026
A129	Lot 31 - 33 Upper	04/05/2026
A130	Lot 34 - 36 Lower	04/05/2026
A131	Lot 34 - 36 Upper	04/05/2026
A132	Lot 37 - Plans	04/05/2026
A133	Lot 38 - 42 Lower	04/05/2026
A134	Lot 38 - 42 Upper	04/05/2026
A135	Lot 41 - 43 Lower	04/05/2026
A136	Lot 41 - 43 Upper	04/05/2026
A137	Lot 44 & 45 Plan	04/05/2026
A138	Lot 46 - Plan	04/05/2026

Cannon Point West Block

Totara Park
Upper Hutt



SITE INFORMATION
District Council: Upper Hutt City Council
Zone: Rural & Plan Change 50 (Residential)
Wind Zone: High
Climate Zone: Zone 4
Earthquake: Zone 3
Exposure: Zone C



1 Location Plan
1 : 1000

Schedule of Site Areas & Site Cover				
Proposed Site Cover				
LOT	Area	Buildings	Decks	Site Cover
1	377 m2	85m2	0m2	85m2 or 22.6%
2	419 m2	145m2	0m2	145m2 or 34.6%
3	640 m2	85m2	0m2	85m2 or 13.3%
4	454m2	94m2	0m2	94m2 or 21%
5	363m2	94m2	0m2	94m2 or 26%
6	366m2	94m2	0m2	94m2 or 26%
7	467m2	150m2	0m2	94m2 or 32%
8	358m2	150m2	0m2	94m2 or 26.3%
9	395m2	150m2	0m2	94m2 or 38%
10	399m2	150m2	0m2	94m2 or 37.6%
11	279 m2	85m2	0m2	85m2 or 30.5%
12	336 m2	85m2	0m2	85m2 or 25.3%
13	450 m2	83m2	0m2	83m2 or 18.5%
14	588 m2	150m2	0m2	150m2 or 25.5%
15	450 m2	101m2	0m2	101m2 or 22.5%
16	246 m2	101m2	0m2	101m2 or 41%
17	259 m2	83m2	0m2	83m2 or 32%
18	463 m2	81m2	0m2	81m2 or 17.5%
19	436 m2	81m2	0m2	81m2 or 18.6%
20	430 m2	85m2	0m2	85m2 or 20%
21	430 m2	88m2	0m2	88m2 or 20.5%
22	430 m2	88m2	0m2	88m2 or 20.5%
23	430 m2	85m2	0m2	85m2 or 20%
24	430 m2	88m2	0m2	88m2 or 20.5%
25	430 m2	88m2	0m2	88m2 or 20.5%
26	397 m2	88m2	0m2	88m2 or 21.4%
27	693 m2	145m2	0m2	145m2 or 21%
28	471 m2	87m2	0m2	87m2 or 18.5%
29	458 m2	87m2	0m2	87m2 or 19%
30	446 m2	81m2	0m2	81m2 or 18.2%
31	436 m2	83m2	0m2	83m2 or 19%

Schedule of Site Areas & Site Cover				
Proposed Site Cover				
LOT	Area	Buildings	Decks	Site Cover
32	438 m2	83m2	0m2	83m2 or 19%
33	437 m2	83m2	0m2	83m2 or 19.1%
34	488 m2	95m2	0m2	95m2 or 19.5%
35	441m2	98m2	0m2	98 m2 or 22.3%
36	403m2	86m2	0m2	86m2 or 21.4%
37	367m2	86m2	0m2	86m2 or 22.3%
38	284m2	94m2	0m2	94m2 or 33%
39	248m2	94m2	0m2	94m2 or 38%
40	763m2	85m2	0m2	85m2 or 11%
41	902m2	85m2	0m2	85m2 or 9.4%
42	247m2	94m2	0m2	94m2 or 38%
43	249m2	94m2	0m2	94m2 or 38%
44	557m2	143m2	0m2	143m2 or 25.7%
45	401m2	160m2	0m2	160m2 or 40%
46	429m2	149m2	0m2	149m2 or 34.8%
47	m2	m2	0m2	m2 or %
48	m2	m2	0m2	m2 or %
49	m2	m2	0m2	m2 or %
50	m2	m2	0m2	m2 or %
51	m2	m2	0m2	m2 or %
52	m2	m2	0m2	m2 or %
53	m2	m2	0m2	m2 or %
54	m2	m2	0m2	m2 or %
55	m2	m2	0m2	m2 or %
56	m2	m2	0m2	m2 or %
57	m2	m2	0m2	m2 or %
58	m2	m2	0m2	m2 or %
59	m2	m2	0m2	m2 or %
60	m2	m2	0m2	m2 or %
61	m2	m2	0m2	m2 or %
62	m2	m2	0m2	m2 or %

Dwelling Typologies

	Type 1A	Lot 1 Lot 3 Lot 11	155m2 155m2 155m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 1B (mirror of Type 1A)	Lot 12	155m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 2	Lot 2 Lot 27	145m2 145m2	Single Story 3 beds, garage, 2.5 bathrooms
	Type 2A	Lot 4 Lot 38	162m2 162m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 2B (mirror of Type 2A)	Lot 5 Lot 6 Lot 39 Lot 42 Lot 43	162m2 162m2 162m2 162m2 162m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 3A	Lot 7 Lot 8 Lot 9	149m2 149m2 149m2	Single Story 3 beds, garage, 2.5 bathrooms
	Type 3B (mirror of Type 3A)	Lot 10 Lot 14	149m2 149m2	Single Story 3 beds, garage, 2.5 bathrooms
	Type 4A	Lot 13 Lot 33	159m2 159m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 4B	Lot 17 Lot 31	159m2 159m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 4C	Lot 32	159m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 5	Lot 15 Lot 16	101m2 101m2	Single Story 2 beds, garage, 2 bathrooms
	Type 6A	Lot 18	154m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 6B	Lot 19 Lot 30	154m2 154m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 6C	Lot 32	154m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 7A	Lot 20	154m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 7B	Lot 23	154m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 8A	Lot 21 Lot 25	158m2 158m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 8B	Lot 22 Lot 26 Lot 29	158m2 158m2 158m2	2 Story 3 beds, garage, 2.5 bathrooms, study
	Type 8C	Lot 24 Lot 28	158m2 158m2	2 Story 3 beds, garage, 2.5 bathrooms, study

	Type 9	Lot 34	158m2	2 Story 4 beds, garage, 2.5 bathrooms, study
	Type 10	Lot 35	169m2	2 Story 4 beds, garage, 2.5 bathrooms, study
	Type 11A	Lot 36	172m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 11B	Lot 37	172m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 12A	Lot 40	148m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 12B (mirror of Type 12A)	Lot 41	148m2	2 Story 3 beds, garage, 2.5 bathrooms
	Type 13	Lot 44		
	Type 14	Lot 45		
	Type 15	Lot 46		

2 Site Cover
1 : 100

3 Typologies
1 : 1000

Sheet
Location Plan

Project
Cannon Point West Block
Totara Park
Upper Hutt

aymac
architecture
limited
5/88 Oriental Parade
Oriental Bay
Wellington
Phone 09(2)(a)
Email 09(2)(a)

Date
Designed
Drawn
Checked
Scale
Revision
04/05/2026
Aymac
Aymac
Aymac
As indicated
Project Number
Sheet Number
2409
A101



1 Street View H1 - 11
1 : 150



2 Street View H12 - 26
1 : 150



3 Street View H21 - 26
1 : 100



4 Street View H27 - 30
1 : 100

Sheet	
Street Elevations	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
Date	04/05/2026
Designed	Aymac
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Revision	
Project Number	2409
Sheet Number	A102



1 Street View H31 - 35
1 : 100



2 Street View H36 - 37
1 : 100



3 Street View H38 - 43
1 : 100



4 Street View H44 - 45
1 : 100

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Street Elevations	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/68 Oriental Parade Oriental Bay Wellington Phone s 9(2)(a) Email	
Date	04/05/2026
Designed	Aymac
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Project Number	2409
Sheet Number	A103



1 Street View H46
1 : 100

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Street Elevations	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
s 9(2)(a)	
Date	04/05/2026
Designed	Aymac
Drawn	Aymac
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Scale	1 : 100
Revision	
Project Number	Sheet Number
2409	A104



1 Lot 1 - Lower
1 : 100

Sheet
Lot 1 - Lower
Project
Cannon Point West Block
Totara Park
Upper Hutt

aymac
architecture
limited

5/88 Oriental Parade
Oriental Bay
Wellington
Phone 04 929(2)(a)
Email

Date 04/05/2026
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Revision

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2409 A105



Sheet	aymac architecture limited 5/88 Oriental Parade Oriental Bay Wellington Phone Email s 9(2)(a)	Date	04/05/2026
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Totara Park		Scale	1 : 100
Upper Hutt		Revision	
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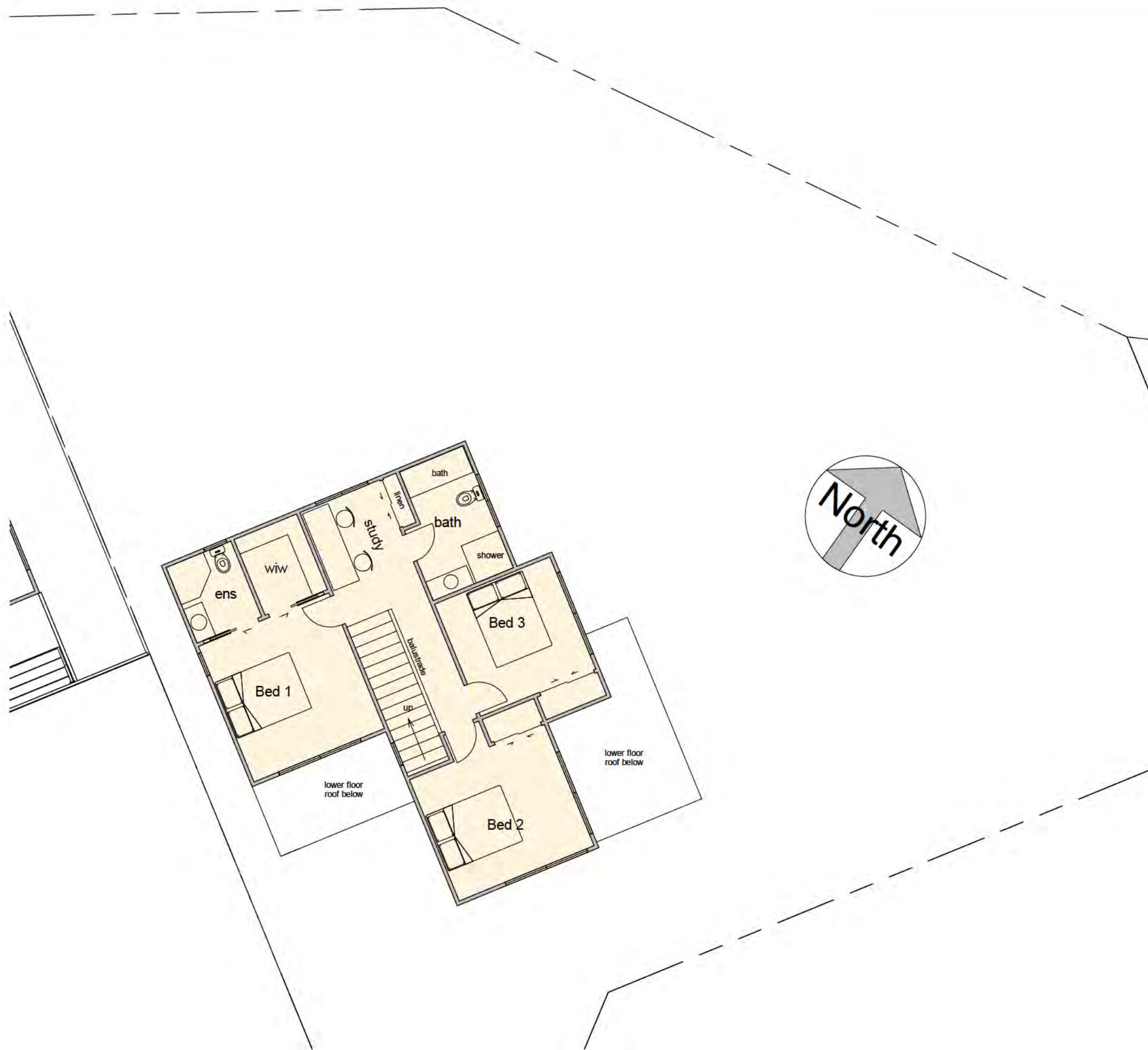


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Lot 2 - Plan	
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Totara Park	
Upper Hutt	
aymac	
architecture limited	
5/88 Oriental Parade	
oriental Bay	
Wellington	
Phone	s 9(2)(a)
Email	
Date	04/05/2026
Designed	Designer
Drawn	Author
Checked	Checker
Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A107

1 Lot 2 - plan
1 : 100



Sheet	
Lot 3 - Lower	
Project	
Cannon Point West Block	
Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington	
Phone	s 9(2)(a)
Email	
Date	04/05/2026
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Drawn	Aymac
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Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A108



Sheet	
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Cannon Point West Block	
Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington Phone Email	
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Date	04/05/2026
Designed	Aymac
Drawn	Aymac
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Scale	1 : 100
Revision	
Project Number	Sheet Number
2409	A109



1

Lot 4 - Upper

1 : 100

Sheet	
Lot 4 - Upper	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington Phone Email s 9(2)(a)	
Date	04/05/2026
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Drawn	Aymac
Checked	Aymac
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Revision	
Project Number	Sheet Number
2409	A111



1 Lot 5 - Lower
1 : 100

Note:
Upper floor the same
as Lot 4

Sheet	
Lot 5 Lower	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington Phone Email	
s 9(2)(a)	
Date	04/05/2026
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Revision	
Project Number	Sheet Number
2409	A112



Note:
Upper floor the same
as Lot 4

Sheet	
Lot 6 Plan	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington Phone Email	s 9(2)(a)
Date	04/05/2026
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Drawn	Aymac
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Revision	
Project Number	Sheet Number
2409	A113



1 Lot 7 & 8 Plan
1 : 100



Sheet	
Lot 7 & 8 Plan	
Project	
Cannon Point West	
T. Block	
Upper Hutt	
aymac	
architecture	
limited	
5/88 Oriental Parade	
Oriental Bay	
Wellington	
Phone	
Email	
s 9(2)(a)	
Date	04/05/2026
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Revision	
Project Number	Sheet Number
2409	A114



Lot 8

edge of flood zone

raised flat terrace

Lot 9
Total Floor
Area 149m²

Outdoor area
90m²

Outdoor area
100m²

Lot 10
Total Floor
Area 149m²

raised flat terrace

edge of flood zone

reserve

Wet
land

washing line

Lot 12

boundary

1000

boundary

1000

1000

water tank

water tank

raided foot path

boundary

boundary

raided foot path

Lot 11

Sheet

Lot 9 & 10 Plan

Project

Cannon Point West Block
Totara Park
Upper Hutt

aymac
architecture
limited

5/88 Oriental Parade
Oriental Bay
Wellington
Phone
Email

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Date 04/05/2026
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Scale 1 : 100
Revision

Project Number Sheet Number

2409 A115

1

Lot 9 - 10 Plan

1 : 100



Sheet
Lot 13 & 14 Lower

Project
Cannon Point West
Block
Totara Park
Upper Hutt

aymac
architecture
limited

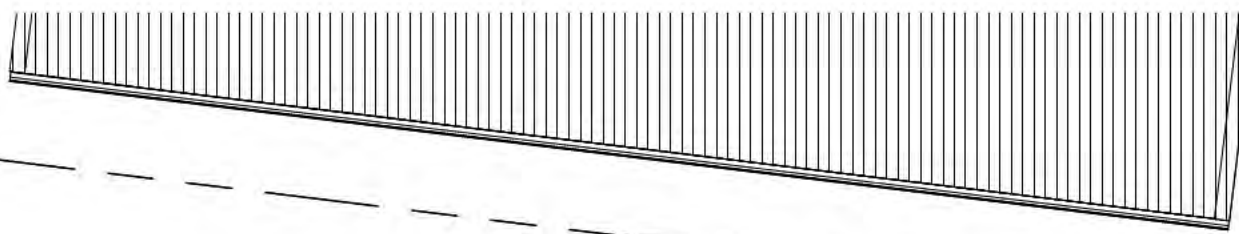
5/88 Oriental Parade
Oriental Bay
Wellington
Phone
Email

Date : 04/05/2026
Designed : Aymac
Drawn : Aymac
Checked : Aymac
Scale : As indicated
Revision

Project Number : 2409
Sheet Number : A117

1 Lot 13 & 14 Lower Plan

1 : 100



Lot 13

Sheet	
Lot 13 - Upper	
Project	
Cannon Point West Block	
Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade oriental Bay Wellington Phone Email	
s 9(2)(a)	
Date	04/05/2026
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Revision	
Project Number	Sheet Number
2409	A118



Sheet	
Lot 15 - 16 Plan	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
s 9(2)(a)	
Date	04/05/2026
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Checked	Aymac
Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A119



1 Lot 17 - Lower

1 : 100



2 Lot 17 - Upper

1 : 100

Sheet	
Lot 17 - 19 Plans	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone 04 488 9999 Email info@aymac.co.nz	
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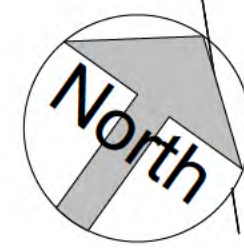
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Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Date	04/05/2026
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Revision	
Project Number	Sheet Number
2409	A121



Sheet	
Lot 20 - 22 Upper	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email s 9(2)(a)	
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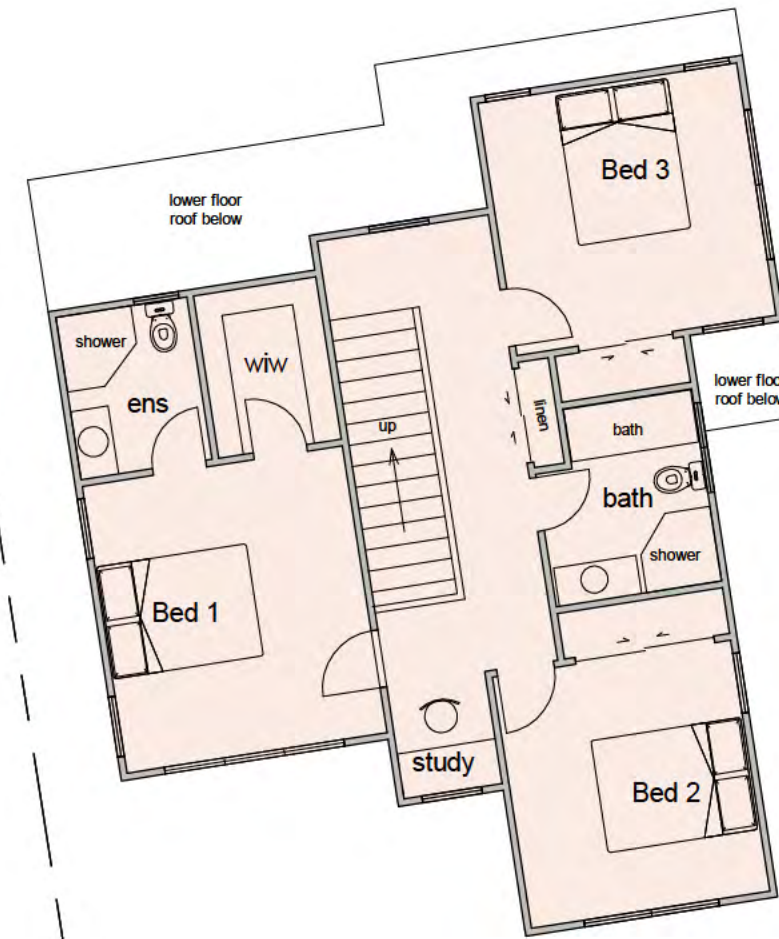
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Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Drawn	Aymac
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Revision	
Project Number	Sheet Number
2409	A123



Lot 26



Lot 25



Lot 24



Lot 23



1

Lot 23 - 26 Upper

1 : 100



Sheet	
Lot 23 - 26 Upper	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A124



Sheet	Lot 27 - Plan
Project	Cannon Point West Block Totara Park Upper Hutt

aymac
architecture
limited

5/88 Oriental Parade
Oriental Bay
Wellington
Phone
Email

Date	04/05/2026
Designed	Aymac
Drawn	Aymac
Checked	Aymac
Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A125



1 Lot 28 - 30 Lower
1 : 100



Sheet	
Lot 28 - 30 Lower	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Date	04/05/2026
Designed	Aymac
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Scale	As indicated
Revision	
Project Number	Sheet Number
2409	A126



Lot 30



Lot 29



Lot 28

1 Lot 28 - 30 Upper
1 : 100

Sheet	
Lot 28 - 30 Upper	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Revision	
Project Number	Sheet Number
2409	A127



Sheet	
Lot 31 - 33 Lower	
Project	
Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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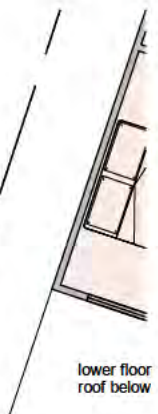
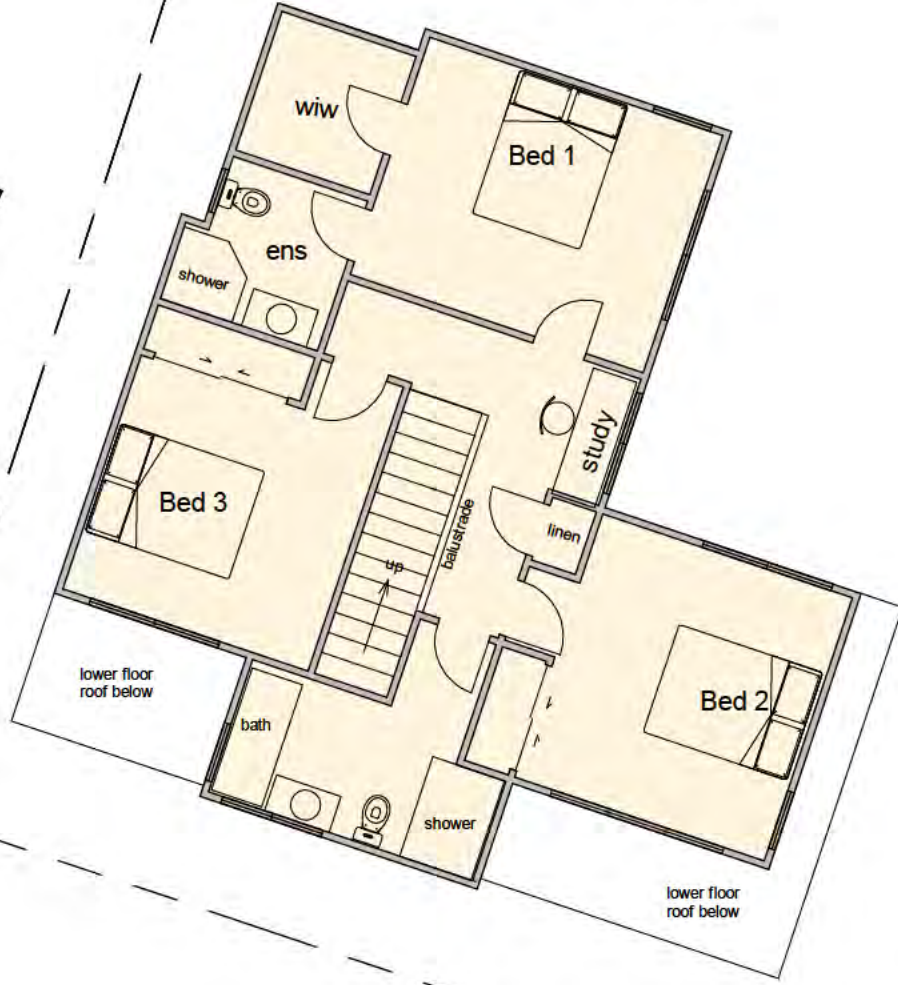


Lot 33

Lot 32

Lot 31

1 **Lot 31 - 33 Upper**
1 : 100



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Cannon Point West Block Totara Park Upper Hutt	
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
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Date	04/05/2026
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Drawn	Aymac
Checked	Aymac
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Revision	
Project Number	Sheet Number
2409	A129



Sheet	Lot 34 - 36 Lower
Project	Cannon Point West Block Totara Park Upper Hutt
aymac architecture limited	
5/88 Oriental Parade Oriental Bay Wellington Phone Email	
Date	04/05/2026
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