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**Project Number 16998.000.002**

**Cannon Point Stage 4 - Geotechnical  
Investigation for Resource Consent**

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

Submitted to:  
Cannon Point Developments Limited  
101 Endeavour Drive  
Whitby



## Executive Summary

ENGEO Limited was engaged by Cannon Point Developments Limited to undertake a geotechnical investigation and prepare a resource consent report for Stage 4 of the proposed Cannon Point Development at 10 Cannon Point Road, Craigs Flat, Upper Hutt. The proposed Stage 4 development comprises a 99-lot subdivision including residential dwellings, green space, wetlands, and a commercial precinct. Our assessment included a desktop review of available information, a geomorphic walkover, supplementary intrusive investigations, development of an engineering geological model, and preliminary geotechnical and natural hazard assessment.

The site extends across alluvial floodplain and elevated terrace land at the base of steep slopes on the north-western margin of the Hutt Valley. Investigations indicate the site is generally underlain by alluvial gravel, local fan, or colluvial deposits, and Greywacke sandstone at depth, with a historic landslide feature identified within the central part of the site. The principal geohazards relevant to the proposed development are seismic shaking, slope instability from the slopes above the site, local instability associated with proposed cut and fill batters, debris flow effects from upslope catchments, and isolated rockfall along the north-western margin. Liquefaction and lateral spreading are assessed as low risk based on the generally dense granular soils and limited groundwater encountered during investigation.

Our assessment indicates that, while much of the site can be developed subject to appropriate geotechnical design and construction controls, there are significant constraints associated with the slopes above the site and with some portions of the current lot layout located within potential landslide runout and debris flow influence areas. Preliminary slope stability analyses indicate that the upslope terrain does not achieve commonly adopted factors of safety under static, wet, or seismic conditions, and that a large seismic event could trigger substantial slope displacement. Debris flow modelling indicates generally low to moderate consequences across much of the site, although a limited number of lots may be exposed to higher levels of inundation or potential structural damage unless final earthworks and stormwater design suitably manage runoff pathways. Rockfall hazard to the proposed building platforms is considered low where green space buffers are maintained.

In summary, the site is considered developable. Engineered mitigation will be required in a number of areas, and specifically below the western hillslopes, where development is proposed. Further detailed design work should include targeted upslope investigation, review of final civil and earthworks plans, assessment of permanent batters, fill suitability testing, and lot-specific foundation recommendations.

The proposed subdivision presents geotechnical and natural hazard considerations typical of Wellington hill developments. While significant mitigation and refined earthworks design will be required, we consider that the site is capable of supporting development subject to detailed geotechnical design and appropriate hazard mitigation measures.

**ENGEO Document Control:**

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## 1 Introduction

ENGEO Limited 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 the resource consent for Stage 4 of the proposed development at 10 Cannon Point Road, Craigs Flat, Upper Hutt (referred to herein as 'the site'). This work has been carried out in accordance with our signed agreement (ref: P16998.000.002\_02, dated 26 February 2026).

Prior to ENGEO's current engagement for the site, Cohesive Consulting Limited (CCL) has undertaken a series of reports to assess the geotechnical site suitability for the purpose of supporting CPDL in the resource consent process. The previous works are outlined in the following:

- Cannon Point Development: East Block Geotechnical Site Suitability Report V2.0 (ref: 1000-007-067, dated 8 September 2025).
- Cannon Point Development: East Block Geotechnical Addendum – Slope Stability Assessment V2.0 (ref: 1000-007-067, dated 8 September 2025).
- Cannon Point East Block – Preliminary Rockfall Assessment (ref: 1000-007-067, dated 12 November 2025).

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 understand 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:16998.000.0002\_02), and includes the following activities:

- Review of the 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 potential geomorphic risks present on and above the site.
- Supplementary subsurface investigations across Stage 4.
- Preliminary slope stability analysis of critical sections to the development.
- Provision of preliminary retaining or ground improvement options.
- Provision of preliminary foundation options for earthworks, batter angles, and geotechnical parameters for retaining wall design.
- Production of this report summarising our findings and recommendations for site suitability.

## 2 Site Description

Stage 4 of the Cannon Point Development is proposed to encompass approximately 87,030 m<sup>2</sup> of land which is spread across two parcels, which have the legal description Lot 301 DP 588162 and Lot 2001 DP 588162, and are zoned as general rural under the Upper Hutt City Council District Plan.

The elongated northeast to southwest oriented site is located level with, and partially elevated above, the Hutt Valley flood plain, near Tōtara Park. The site 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 perpendicular to the northern end of the site (~50 m from the northernmost boundary), before turning and running parallel with the site.

The site is currently predominantly grassland. The south-eastern half of the site is largely flat, with some minor abandoned channels at the southern extent. The site then steps up to the northwest, to form a terrace that is elevated tens of metres above the surrounding flood plain.

Figure 1: General Site Location Plan



Produced using Datanest.

## 3 Proposed Development

The Stage 4 development plans have been provided by Vecta Limited (ref: 26014 – Cannon Point Fast Track-EB, dated 6 June 2026), shows a 99 Lot subdivision which is comprised of residential dwellings, wetlands, green spaces, and a commercial precinct, which is to be established on a partially elevated flood plain. The proposed lots comprise land areas ranging from 168 m<sup>2</sup> and 5041 m<sup>2</sup>.



As outlined in the Vecta master plans for Stage 4 of the Cannon Point Development, earthworks / land modification is required to achieve a suitable site level for development. Across the site, a total earthwork area of 65,400 m<sup>2</sup> is proposed, which is comprised of 26,948 m<sup>2</sup> of cut and 38,452 m<sup>2</sup> of fill. Within these areas, volumes of 85,150 m<sup>3</sup> of cut and 94,222 m<sup>3</sup> of fill are proposed; therefore, leaving an excess of 9,072 m<sup>3</sup> requiring offsite disposal.

Based on the Vecta plans, no retaining walls are required on the site; however, based on the presence of vertical cuts up to 8 m high, such as those displayed in cross section B-B, retaining walls will be required.

A series of batters are proposed which will be constructed by cutting into the slope above the site, and filling against the boundary of properties in Tōtara Park. Fill batters are proposed to grade from cut in the north, to approximately 6 m high at the southern extent. Cuts of up to 17 m are proposed at the toe slope.

The current service plan indicates that stormwater and sewer will be separated and diverge approximately one quarter from the south-western extent of the site. The southern half of the site will drain all services to Colorado Drive, while the northern extent will drain to the established mains on Larchmont Grove.

The site will have to work around the established electrical pylons and cable in accordance with the restrictions imposed by the operators of such services.

## 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 spread across the Hutt Valley flood plain and an elevated terrace, which sits tens of metres above the surrounding alluvial flood plain, and the Hutt River. A large portion of the site is located approximately equal in height with the Tōtara Park subdivision to the southeast. The site is generally flat with minor undulation at the south-western extent of the site where a couple of minor abandoned stream channel are present.

Steep, densely vegetated native bush of the Akatawara Forest is located to the west of the site, where a series of northwest to southeast ridgelines separates four subcatchments. The larger of the two subcatchments is sizable (~200,000 m<sup>2</sup>) and has incised into the elevated terrace. A debris fan is present at the base of the channel.

In the centre of the site, a large undulating southeast facing slope is present which extends across the terrace to the lower flood plain.

Some minor landform modification, in the form of farm tracks, has taken place to enable access around the site during its previous land uses. Additionally, two high-voltage transmission towers are present on the site.

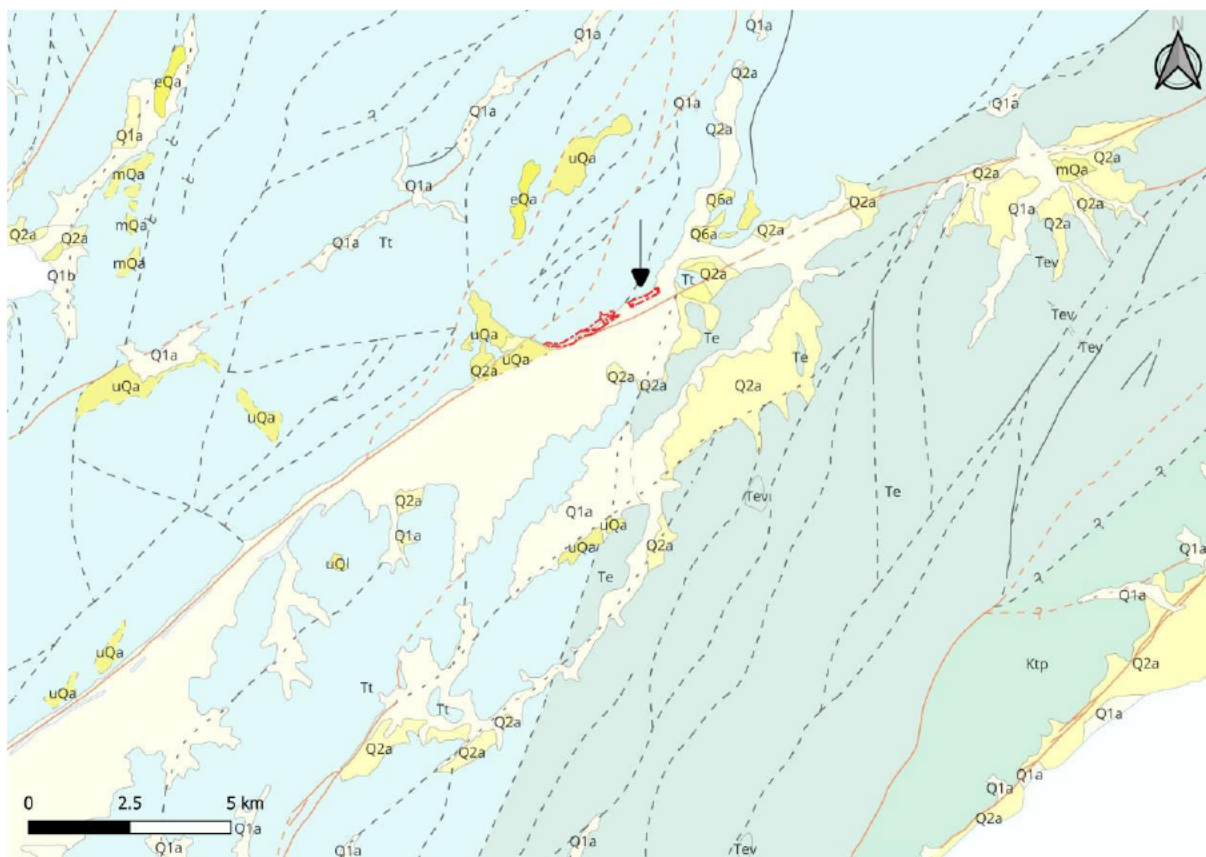
## 4.2 Geology and Geomorphology

The site is located in the Upper Hutt Valley, on the northern edge of the valley wall, between the Akatarawa Forest, a part of the Tararua Range to the north, and the Rimutaka Range foothills to the south. Both ranges as indicated in Figure 2 are predominantly Greywacke Sandstone. The Hutt River, which drains from the Tararua Range catchment, has been actively meandering through the Hutt Valley for tens if not thousands of years degrading and aggrading terraces throughout the valley.

The generalised geology of the site (Figure 2) has been obtained from the 1:250,000 scale geological map of New Zealand (GNS Science Geological Map 10). The site is underlain by the 'Q1a' Holocene Alluvial Deposits, which is comprised of well sorted gravels, and 'Q2' Alluvium which comprise poorly to moderately sorted gravel with minor sand and silt underlying terraces.

Based on our understanding of the previous land use and prior investigations, localised areas of fill may be present.

**Figure 2: Generalised GNS Science Geological Map of Wellington**



Produced using QGIS, with information sourced from GNS Science Web Map and Web Services. Red polygon and arrow indicating site location. Arrow indicating Stage 4 of Cannon Point.

## 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 (at Tōtara Park) along the length of the site by Van Dissen et al., (2005) and others. The Wellington Fault, like many faults in the southern North Island is an oblique dextral reverse fault, with strike-slip motion as 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). The annual slip rate is estimated to be on the order of 6 mm/yr, while the horizontal net single displacement event has been estimated at 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 / rivers and terraces), following 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 sense 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 of up to 2.7 m.

Several other faults not listed in Table 3.6 of NZS 1170.5 are within 20 km of the site, and include the Akatarawa, Moonshine, and Ohariu Fault. 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 in QGIS. Pink around fault indicates the location of the fault is well constrained.



#### 4.4 Historical Aerial Photography Review

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

- **1942:** The site is largely covered by grassland. Some larger growth is present on the terrace riser and tread. A large lobe-like structure is present at the base of the slope immediately north of the Subcatchment 1 Gully, with no vegetation occupying the undulating slope; however the terrace riser is covered with some semi-mature vegetation. The terrace tread and riser has been incised into, a result of the catchment upstream. Three power poles transect the long axis of the site. No development has occurred at Tōtara Park during this period.
- **1951:** The site remains in grasslands and appears to be actively farmed. Vegetation is further established on the terrace riser, with additional vegetation on the lobe-like structure in the centre of the site. Additional farm tracks are established across the site, and some sort of agricultural planting has taken place on the terrace tread southwest of the Subcatchment 1 Gully. Some white objects are present at the base of most slopes and are likely to represent rockfall debris.
- **1962:** Little has changed on the site. Agricultural planting is no longer present on the terrace tread at the south-western end of the site. Tōtara Park continues to be farmed with some agricultural crops present. The slopes above the site continue to become more vegetated.
- **1974:** The site remains largely unchanged. The three power poles that were previously present on the site have been removed and two high-voltage transmission lines / towers have been established. One transmission line is present on the lobe-like structure and another at the northeastern end of the site. Tōtara Park has begun being established.
- **1980:** Tōtara Park has continued its north-westerly expansion and now buttresses the southeastern boundary of the site. The terrace riser, tread, slopes above the site, and lobe structures are now significantly more vegetated.
- **1988:** The site has become more vegetated; the lower flood plain area of the site remains as grassland.
- **2003:** Some of the vegetated areas at the northern end of the site have been reduced to increase the grassland areas on top of the terrace tread. Tōtara Park is established as it remains today. The site remains today largely as it is displayed here.

#### 4.5 Hazard Mapping

The Greater Wellington Regional Council maps indicate that the site has both a 'moderate' and 'high' combined earthquake hazard rating.

The 'moderate' rating is derived from a 'low' slope failure hazard, a 'low' liquefaction potential, and a 'low to moderate' ground shaking hazard.

The 'high' rating is derived from a 'moderate' slope failure hazard, a 'low' liquefaction potential, and a 'low to moderate' ground shaking hazard.

The Greater Wellington Regional Council (GWRC) Maps display that the site is located well away from the tsunami evacuation zone.

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

#### 4.6 New Zealand GNS Landslide Database

The New Zealand Landslide Database (Morgenstern et al., 2025) shows three recorded landslides occurred in proximity to the Cannon Point Development; however, no landslides have been registered above or immediately surrounding the proposed Stage 4 area.

#### 4.7 Select Land Use Register

The GWRC's Select 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 minimal investigations have been uploaded to the site. ENGEO has been provided with investigation data from the site and surrounding area for use in their report which is summarised as follows (logs attached in Appendix 3):

- CCL, using Pro Drill Limited services, undertook five boreholes in the Stage 5 area. Four were completed on the elevated terrace tread, one was completed on the flood plain, and achieved a maximum depth of 12 m.
- CCL additionally undertook three test-pit excavations which achieved a maximum depth of 3 m, and 14 window sample boreholes, which achieved depths between 0.2 and 2.2 m.
- CCL undertook 40 Scala penetrometers across the site, achieving depths between 0.2 and 4.0 m.

### 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 4 area.

#### 5.1 Site 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 4.

- At entrance to the site, off Colorado Drive, two sheds occupy a relatively small, fenced paddock area.
- The grasslands along the south-eastern half of the site were generally flat, with minor undulation. At the south-western end of the site, immediately northeast of the sheds, incision of a minor stream has taken place, a culvert is present at the southeast boundary.

- Several farm tracks are present across the sites, some of which traverse the steep terrace risers and are cut into the toe of the slopes above the site.
- An elevated terrace is present along the northeast to southwest trend of the site against toe of the north-western slopes. The terrace riser (sloped area) varies along its length of the site. In the south-western extent the riser is steep, and considerably higher, when compared to the north-eastern end which is shallow slope and only several metres high.
- At the south-western extent of the terrace riser a couple of shallow head scarps were noted along the crest of the terrace riser.
- In the centre of the site the steep terrace riser geometry is replaced by an undulating shallow slope, which continues to a greater height on the slopes above the site than the surrounding terrace riser. This terrane generally contains a lower level of semi-mature vegetation compared to the surrounding terrane. The toe of the undulating slope is lobate extending away from the slope suggestive of the toe of a historic landslide.
- Located immediately southwest of the historic landslide is a large stream channel which has incised through the terrace tread and rise. The stream has a significant volume of smaller angular rock debris present at its base; however, the banks are well vegetated, and the catchment is vegetated to the point that access is limited. No obvious debris puncture marks are present on the semi-mature vegetation within the stream channel.
- At the mouth of the stream, which coincides with the terrace riser, a gentle slope radiates out in a semi-circular shape and is likely a debris flow fan from previous debris flow events.
- Some larger rounded cobbles and boulders debris was present in the stream channel, which looked as though it has been dumped off the top of the terrace tread to the north, possibly in an effort to control the direction of the stream.
- Groundwater was observed to be seeping out of the terrace riser immediately south of the stream apparently along the soil to rock boundary. The ground was saturated in the area immediately surrounding the seepage area; however, this was also near the stream channel.
- Rock outcrops were observed in several locations along the length of the site, particularly at towards the toe and crest of the terrace riser. Rock was also observed in the slope above the site at several points.
- Across the site, Greywacke boulders were observed at the toe of and across the face of the terrace riser, along the top of the terrace tread, and along the toe of the slopes above the site, which were covered by the tree canopy. Boulders across the grassland and terrace tread were sparse and isolated, while those at the toe of the slopes above the site were abundant and clustered.
- Rock and boulder debris was variable in size, with isolated boulders having long-axis lengths varying between 0.4 to 2.6 m. While clustered areas of rockfall had comparatively smaller long-axis lengths of between 0.1 and 0.3 m, with one longer boulder was measured at greater than 2.0 m (however this specimen had a tree growing over the top of it, indicating that it had been there for a long time).



- The rockfall at the toe of the slope was commonly more angular, while the debris across the grassland area was commonly more subrounded, indicating that the boulders on the grassland area are possibly due to alluvial drop out (i.e., when the stream was flowing across the floodplain, rather than falling from the slopes above, which would generate angular debris).
- Most rockfall debris had moss and lichen growing on it and was surrounded by established vegetation. There was some debris which was not vegetated or with moss, indicating that there had been some recent rockfall.
- Much of the rockfall debris was no more than 10 m away from the track at the toe of the slope. The largest boulder observed outside of the bush line was 2.6 m in length and may have originated from an exposed rock face in the slope above. However, it is unclear however if the boulder had been moved to its current position or if it naturally fell to this point.
- Two large transmission lines are present on the site, one located at the north-eastern end of the site, another is located centrally to the site atop the terrace tread. A telecommunications tower is also located in the northernmost corner of the site. Additionally, a series of traditional concrete power poles are located along the grassland area of the site.

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

**Figure 4: Site Walkover and Geomorphic Observations**



Photo 1: North-eastern extent of terrace riser.

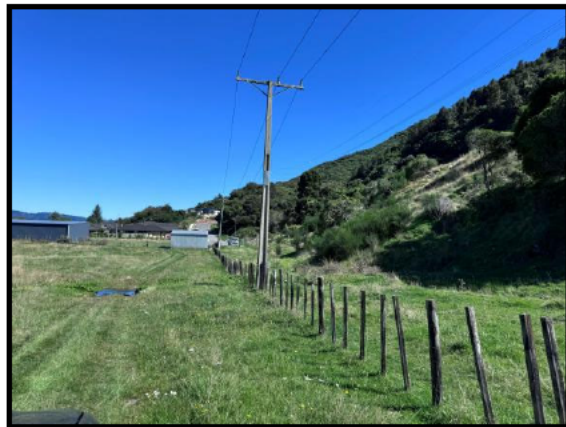


Photo 2: Steeper south-western extent of terrace riser





Photo 3: Photo looking down across undulating ground of historical landslide.



Photo 4: Gentle lobe-like structure from apex of Subcatchment 1.



Photo 5: Rock observations where groundwater seepage was observed.



Photo 6: Example of boulder on terrace riser.



Photo 7: Rockfall debris observed at the base of slopes. Largely moss and lichen covered.



Photo 8: Transmission line on top of land slide debris above the site

## 5.2 Investigations Completed

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

Twelve test-pits with associated Scala penetrometer and shear vanes were undertaken across the site and achieved depths of 1.4 to 2.5 m. A test location plan and the geological logs are displayed in Appendix 5. Test pits are labelled STG4-TP01 through STG4-TP12. Material was logged in accordance with the 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 Soil Profile**

| Material / Description                              | Description                                                                          | Consistency / Density      | Depth to top of layer (m) | Depth to bottom of layer (m) |
|-----------------------------------------------------|--------------------------------------------------------------------------------------|----------------------------|---------------------------|------------------------------|
| Topsoil                                             | Gravelly SILT with some sand and silty fine to medium SAND                           | Firm to medium dense       | 0                         | 0.05 – 0.3                   |
| Fan deposits / Colluvium                            | Sandy fine to coarse GRAVEL with some silt                                           | Still to very stiff        | 0.05 – 0.3                | 0.8                          |
| Landslide Debris <sup>1</sup>                       | Cobbly fine to coarse GRAVEL with some sand and minor silt                           | Medium dense to dense      | 0.2                       | 2.3                          |
| Alluvium                                            | Sandy / Silty fine to coarse GRAVEL and Cobbly fine to coarse GRAVEL with some sand. | Medium dense to very dense | 0.05 – 0.8                | 2.3                          |
| Slightly Weathered Greywacke Sandstone <sup>2</sup> | Greywacke Sandstone                                                                  | Very dense                 | 2.0                       | N/A                          |

<sup>1</sup> STG4-TP01 was undertaken within the large undulating slope in the centre of the site thought to be a historic landslide.

<sup>2</sup> Greywacke Sandstone was only encountered in STG4-TP02, however, was observed at the ground's surface.

## 6 Engineering Geological Model

An Engineering Geological Model (EGM) has been developed to provide a conceptual framework for understanding the site's geological and geomorphological conditions and the processes including them. The EGM integrated 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 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

### Alluvial Terraces

The site is proposed to be partly located across an alluvial flood plain and an elevated terrace, however, the flood plain the site contains is slightly elevated to the surrounding area, likely due to up-lift on the Wellington Fault, which is located southeast of the site.

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 from similar aged terraces found to the north, along with visual observations, the alluvial gravels typically overlay Greywacke Sandstone. Since the formation of the terrace which the site partially 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. The Hutt River today sits many metres below the site level.

A later event, likely during the Holocene, diverted the Hutt River further east, away from the slope, such that the river now does not affect the site. There is evidence in the historical aerial images to suggest that there have been periods when the Hutt River has directed its course back towards the site, particularly during flood events, however, it does not appear to have resulted in any significant erosion.

### Colluvium / Landslide Debris

Following, and likely during the aggregation of the primary terrace, the slopes above the site appear to have been actively distributing material into the terrace. The increasing steepness of topography away from the toe of the major slopes is an indication colluvial debris has failed and accumulated at the toe of the slopes, however, there is no recent evidence of shallow failure observed during ENGEO's site walkover.

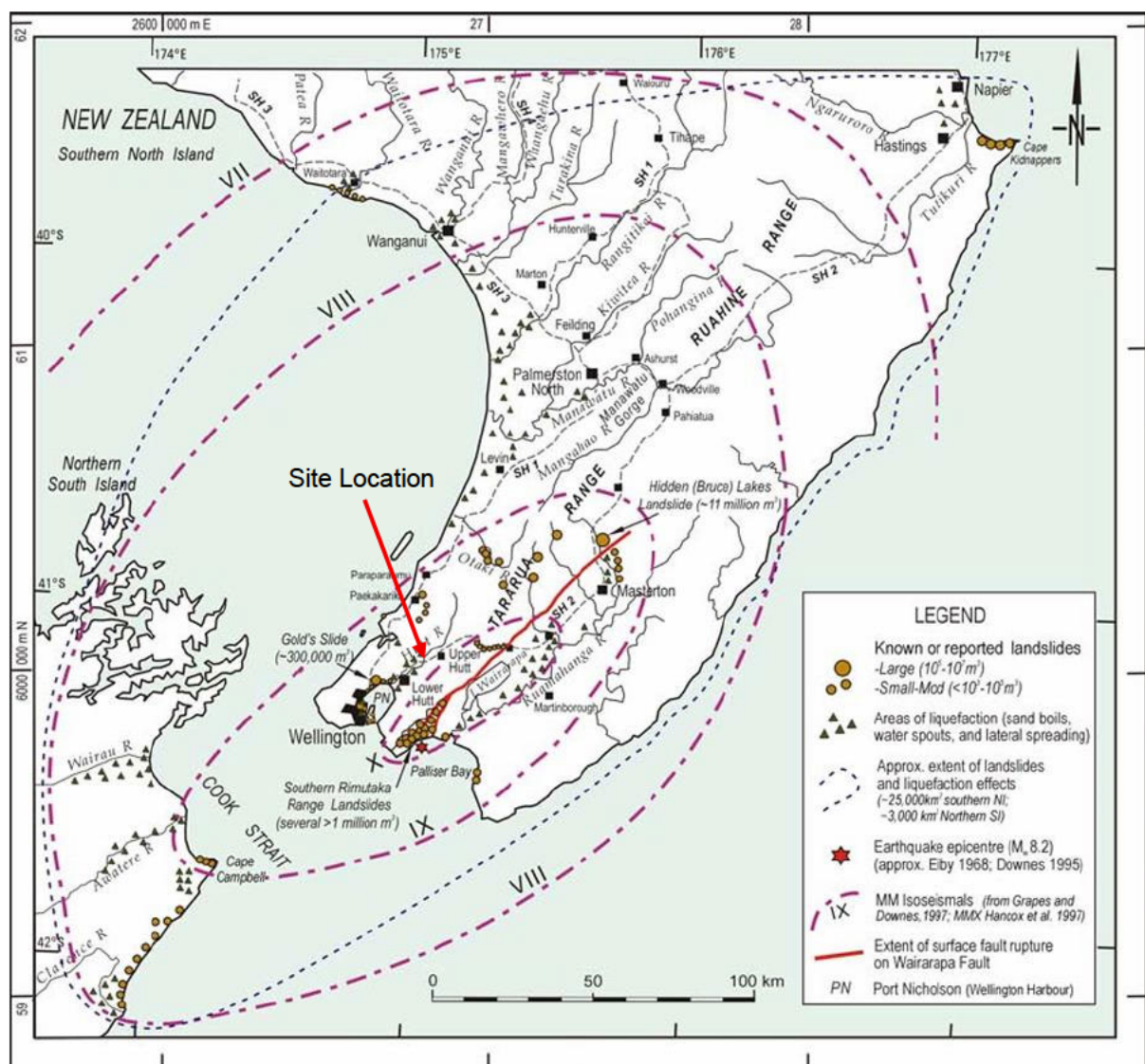
However, the undulating lobe-like structure that overprints the terrace in the centre of the site, based on our investigation and further review of historical aerial photography, is interpreted to represent a possible relic mass movement feature. Based on the scale of the landslide, we interpret that it is likely the result of a catastrophic event.

Given the age of the terrace that is seen to be overprinted around 1942, and the approximate age of the surrounding vegetation, which is not present on the undulating terrane, it is likely that the landslide occurred following the establishment of vegetation on the terrace riser. While this does not constrain the age of this landslide very well, the 1855 Wairarapa Earthquake is considered one possible contributing mechanism; however, the precise origin and timing of the feature remains uncertain.

There is a vast quantity of literature documenting the prolific landsliding around the Greater Wellington Region that occurred co-seismically with the Wairarapa Fault rupture. Hancox (2005) has mapped the distribution and size of hundreds of reported landslides that occurred following the Wairarapa earthquake, with sizes ranging from 100 m<sup>3</sup> to 11,000,000 m<sup>3</sup>. Many small to moderately sized landslides occurred within central Wellington and the Hutt Valley, with the famous Gold's slide, an ~300,000 m<sup>3</sup> landslide, occurring between Ngauranga Gorge and the Petone Esplanade.

The morphology of the hill slopes and general fracturing and weakness present in the geology of the western valley wall of the fault-angle basin has largely been caused by the Wellington Fault. When considering all of the factors outlined above, it is the Wairarapa Earthquake that is most likely candidate to have caused the proposed landslide in our opinion.

**Figure 5: Distribution of Reported Landslides Following the 1855 Wairarapa Earthquake (Hancox, 2005)**



Map taken from Hancox, (1855).



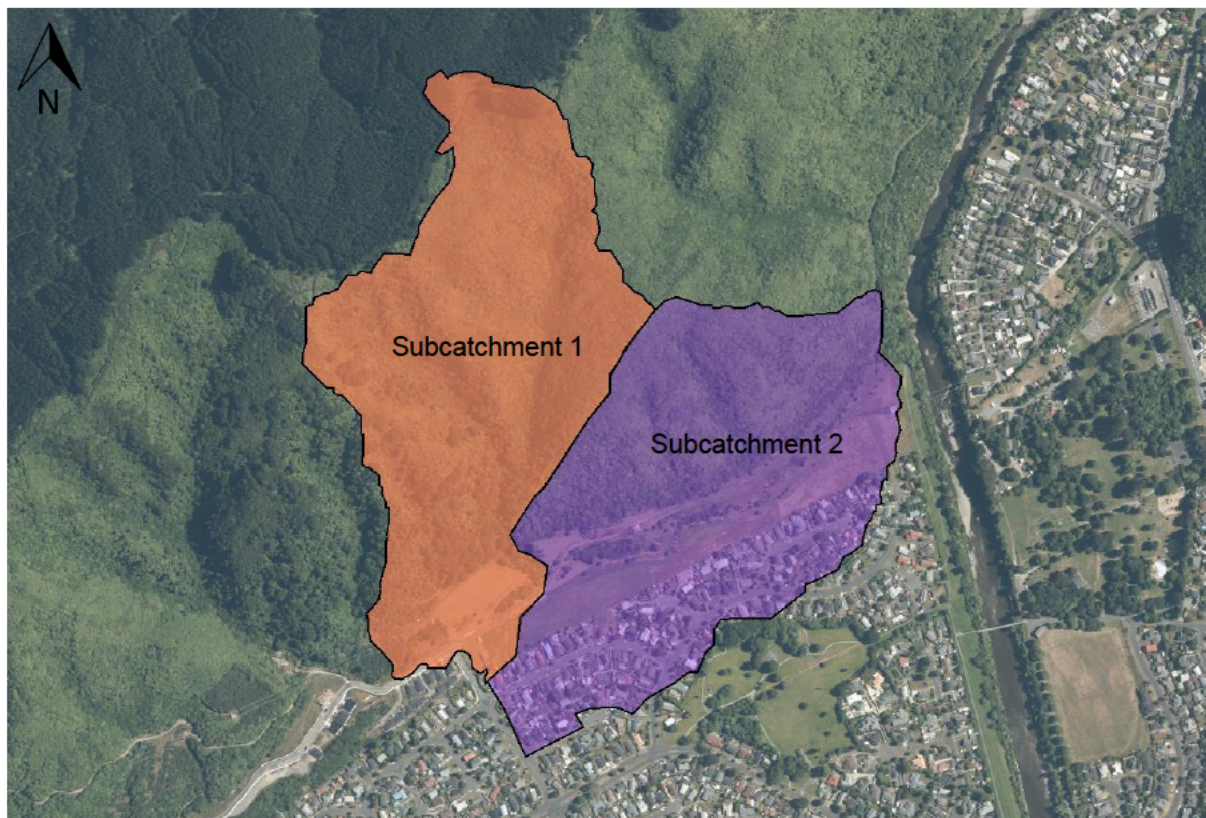
### Debris Flow / Streams

The site is largely contained within the limits of two primary catchment areas (Figure 5). Subcatchment 1 has the largest surface area and is the only perennial stream on the site; meanwhile, Subcatchment 2 contains four minor southeast facing gullies which are ephemeral (only flowing some of the time, and usually briefly).

At the base of Subcatchment 1, a gentle slope radiates from the apex of the channel in the semi-circular shape and has been inferred to be indicative of debris flows having previously occurred.

Based on field evidence, we were unable to determine whether there was a discrete event or whether a series of events had previously occurred, but investigations near the apex of the gully revealed approximately 1.3 m of medium dense gravels which were brown in colour when compared to the bluish-grey gravels seen below.

**Figure 6: Catchment Areas Located Above Stage 4**



### 6.4 Identified Geohazards

The site walkover and EGM identified that, debris flow, slope instability, and rockfall processes are geohazard that the site is exposed to, and are characteristic of some Wellington hill slope terrain.

These hazards are discussed in greater detail in the following sections, and levels of risk in according with the National Policy Statement for Natural Hazards are outlined in Section 0.

## 6.5 Assumptions and Uncertainties

Given the lateral and vertical elevation of the site, we have assumed that the flood plain and terrace have both been formed by similar alluvial / geomorphological processes. We have assumed that the terrace present on the site was established at a similar period as those formed throughout the Upper Hutt Valley. Based on observations and investigations undertaken on the site, and documentation of terraces of similar age, we have assumed that the alluvial gravel material underlying the terraces was deposited directly overlying Greywacke Sandstone.

There remains some uncertainty regarding the timing and origin of the event that initiated the large landslide referred to in Section 6.2. Our best estimate is that it was initiated by the 1855 Wairarapa Earthquake; however, it may predate this (it is unlikely to post-date it). Having stated this, the geomorphology of the landslide appears to be relatively fresh, suggesting that it is unlikely to be thousands of years old.

Due to the densely vegetated slopes immediately above the site, it was not possible to access, or view, the slopes above the site, which limited direct observation of upslope terrain and therefore necessitated conservative assumptions within the assessment. Additionally, the thickness of the landslide debris at the centre of the site is unknown. One test pit (STG4-TP01) was undertaken within the landslide debris which reached 2 m approximately halfway up the slope and did not reach the underlying lithologies. Further targeted subsurface investigation and slope assessment may refine the Engineering Geological Model and reduce conservatism within the current analyses.

## 6.6 Engineering Geological Units

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

**Table 2: Engineering Geological Units**

| EGM Unit                         | Test ID                        | Description                                                                    | In Situ Testing SPT N, DCP |                                                                  |
|----------------------------------|--------------------------------|--------------------------------------------------------------------------------|----------------------------|------------------------------------------------------------------|
|                                  |                                |                                                                                | m bgl                      | In Situ Results                                                  |
| Holocene Debris flow Material    | STG4-TP09                      | Sandy fine to coarse GRAVEL with some silt.                                    | 0.15 – 1.5                 | DCP (5 – 11 blows /100 mm)                                       |
| Landslide Debris <sup>1</sup>    | STG4-TP01                      | Sandy fine to coarse (angular) GRAVEL with minor silt.                         | 0.2 – 2.5                  | DCP (2 – 15+ blows /100 mm)                                      |
| Pleistocene / Holocene Alluvium  | STG4-TP12<br>MBH04 (CCL)       | Silty and sandy fine to coarse gravels with some / minor cobbles and boulders. | 0.05 – 2.3                 | DCP (1 – 15+ blows /100 mm)<br>SPT (N values between 16 and 50+) |
| Greywacke Sandstone <sup>2</sup> | STG4-TP02<br>BH02 & BH05 (CCL) | Slightly weathered Greywacke Sandstone                                         | 2.0 – N/A                  | SPT (N values ≥50)                                               |

<sup>1</sup> ENGEO only encountered this material in one investigation locations.

<sup>2</sup> The only *in situ* test data for Greywacke Sandstone has been taken from CCL geotechnical borehole logs.



## 7 Analysis

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

- Seismic Hazard.
- Liquefaction and Lateral Spreading.
- Undocumented fills and historical landform modification.
- Slope instability and rockfall.
- Earthworks requirements including preliminary foundation recommendations as well as erosion and sediment control.

Each of these aspects is discussed in the following sections.

### 7.1 Seismic Hazard

Potential seismic hazards resulting from nearby moderate to major earthquakes can generally be classified as primary and secondary. The primary effect is ground rupture, also 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, especially given the high degree of certainty in the mapped extent of the fault; however, 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 the site will likely experience strong seismic shaking during major regional earthquake events, consistent with wider Wellington Region.

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

Through discussions with the client, we understand the potential site development includes Importance Level 2 (IL2) structures. Peak horizontal ground accelerations ( $a_{max}$ ) for use in geotechnical assessments are provided in Table 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 subsurface profile at the site typically consists of gravels and other well graded materials, which are typically moderately dense to very dense from approximately 0.5 m depth. Importantly, the water table at the site is possibly up to 5 m depth, with isolated locations of slightly shallowing depth to water table (~1.5 m and as indicated by groundwater seepage). We therefore consider the likelihood of liquefaction and lateral spreading to be low.

Additionally, the free-face of the Hutt River is ~40 m from the boundary of the site, with rock seen to be outcropping along the stream bank, further minimising the likelihood of lateral spreading.

## 7.3 Upslope Stability Assessment

Based on our desktop study, review of historical aerial imagery, literature, site observations and investigations, and review of the proposed development plans, we consider that slope instability from the slopes above the site is likely to be a hazard at the site for future development. Given that large amounts of cut material are proposed to be undertaken across the terrace riser and tread, the slope instability risk from the terrace itself is generally decreasing; however, lots are still proposed to be developed in proximity to the slopes above the site.

We evaluated the potential stability of the slopes above the 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 4 development with the locations displayed in Appendix 6. Sections were analysed as they are proposed to be developed in accordance with the most relevant earthworks drawings (26014 – Cannon Point Fast Track-EB, dated 27 February 2026). Batter design and height will be required to be revisited at the detailed design stage.

**Section A:** Located at the south-western end of the development spanning the slopes above the site, and through the terrace riser.

**Section B:** Located further north and perpendicularly intersection the terrace riser.

**Section C:** Oriented approximately north to south intersection the large landslide and continuing to the slope above the site.

**Section D:** Located at the northern end of the development, approximately northwest to southeast, and intersects a similar slope face to that which has already failed in a catastrophic landslide.

### 7.3.2 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 typical Greywacke Sandstone weathering profile from completely weathered rock to moderately / slightly weathered rock.

### 7.3.3 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.

### 7.3.4 Geotechnical Material Parameters

The following geotechnical material parameters (Table 4) 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 Hutt 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 <sup>3</sup> ) | Strength Type | Effective Stress Parameters |         | USC (kPa) | GSI | mi |
|------------------------------------------|----------------------------------|---------------|-----------------------------|---------|-----------|-----|----|
|                                          |                                  |               | Ø' (°)                      | c (kPa) |           |     |    |
| Fill Material                            | 18                               | Mohr-Coulomb  | 32                          | 2       | N/A       |     |    |
| Saturated Fill Material                  | 20                               | Mohr-Coulomb  | 31                          | 0       |           |     |    |
| Historic Landslide Debris                | 17                               | Mohr-Coulomb  | 32                          | 0       |           |     |    |
| Saturated Historic Landslide Debris      | 20                               | Mohr-Coulomb  | 29                          | 0       |           |     |    |
| Colluvium / Debris Flow                  | 17                               | Mohr-Coulomb  | 30                          | 5       |           |     |    |
| Saturated Colluvium                      | 18                               | Mohr-Coulomb  | 30                          | 1       |           |     |    |
| Alluvial Gravels                         | 21                               | Mohr-Coulomb  | 35                          | 40      |           |     |    |
| 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  |

### 7.3.5 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                              |                            |
| Design Ground Acceleration (PGA)                       | 0.13 g                           | 0.68 g                     |

### 7.3.6 Analysis Results

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

Discussion of the slope stability results is provided below.

**Table 6: Slope Stability Results for Stage 4 Slopes**

| Section ID   | Scenario                        | Required FoS | Calculated FoS | Acceptable? |
|--------------|---------------------------------|--------------|----------------|-------------|
| Section A-A' | Static                          | 1.5          | 1.07           | No          |
|              | Raised Ground Water + Wet Front | 1.2          | 0.73           | No          |
|              | SLS                             | 1.2          | 0.88           | No          |
|              | ULS                             | 1.0          | 0.38           | No          |
| Section B-B' | Static                          | 1.5          | 0.94           | No          |
|              | Raised Ground Water + Wet Front | 1.2          | 0.74           | No          |
|              | SLS                             | 1.2          | 0.75           | No          |
|              | ULS                             | 1.0          | 0.33           | No          |

|              |                                 |     |      |    |
|--------------|---------------------------------|-----|------|----|
| Section C-C' | Static                          | 1.5 | 0.83 | No |
|              | Raised Ground Water + Wet Front | 1.2 | 0.53 | No |
|              | SLS                             | 1.2 | 0.67 | No |
|              | ULS                             | 1.0 | 0.34 | No |
| Section D-D' | Static                          | 1.5 | 0.86 | No |
|              | Raised Ground Water + Wet Front | 1.2 | 0.59 | No |
|              | SLS                             | 1.2 | 0.71 | No |
|              | ULS                             | 1.0 | 0.31 | No |

#### Static Conditions

In Sections A, B, C, and D, a Factor of Safety (FoS) around or below the 1.0 target was achieved under static conditions, indicating that the slope is marginally stable under long-term static conditions. However, the minimum FoS for long-term stability (1.5) has not been achieved. During our site walkover ENGEO did not identify any obvious signs of soil creeping or recent instability near the toe of the slope, as the upslope regions were unable to be accessed. However, given that no obvious instability was noted on-site, or during our review of the recent historical aerials it would indicate that the slope is able to maintain its existing slope angle.

The central and up slope regions have marginally higher FoS values.

In Section C, where the 1V:2H batters are proposed to be cut (within the historical landslide debris), FoS values are between 1.2 and 1.5. The material as it is currently present is loose and uncompacted and therefore requires further work and individual assessment to determine its suitability in permanent cuts. For cut and fill slopes, these analyses should be interpreted as a tool for design refinement.

#### Raised Groundwater / Wetted Front

Our analysis of the slopes above the site under “raised groundwater” conditions, show that failure planes are localised within the surficial 1–2 m of saturated material (wetted front). Calculated FoS values are less than the minimum requirement for saturated conditions scenario of 1.2.

In Section C, the FoS of the proposed batters just falls short of the minimum requirement for the raised groundwater scenario achieving values between 1.12 and 1.18. As per our previous comment however, these will require further work during earthworks to improve their suitability for permanent batter construction.

In cases where saturation induced failures occur, we consider that the most likely scenario is that failures 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.

## Seismic Conditions

Our analysis indicates that the minimum recommended FoS values were not achieved on the slopes above the site, or the proposed batters beneath the site under an Ultimate Limit State (ULS) seismic event.

Above the site all sections analysed achieved FoS values around 0.3–0.4 in the slopes above the site, which is below the minimum recommended FoS for ULS scenarios. Failure planes are typically 1.5 – 3 m in depth and extend along the length of the slope above the site, taking advantage of the highly weathered to moderately weathered Greywacke Sandstone boundary. This accords well with the observations that we have made about the historic landslide that is present on the site.

Additionally, Section C displays that the proposed batters which are to be cut into the historic landslide would require additional mitigation and refinement to satisfy ULS performance objectives, and only achieve a FoS of between 0.37 and 0.6. Failure planes within the proposed landslide batter extend back into the slope by approximately 7 m.

The fill batter located along the boundary of the site additionally displays a FoS value of less than 1.0, and therefore not meeting the minimum requirements. Both batters can likely be improved through refinement of geometry and / or engineered reinforcement measures to make them viable under ULS conditions.

A global failure that starts above the site and extends beneath the development into Tōtara Park is indicated to have a FoS value of 0.96, however, this is considered to be unlikely. Notwithstanding the above, the site presents geotechnical and natural hazard considerations typical of Wellington hill developments.

### 7.3.7 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, and then displacement may occur, the degree of which depends on the materials involved and their strength properties.

To assess the likely co-seismic displacement, we have made several assumptions based on the information available to us, these are detailed in Section 7.3.2. 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 Vs30 of site – 600 m/s (for slopes above the site) and 400 m/s (for the terrace and flood plains). Vs30 is a parameter that represents the average shear-wave velocity in the top 30 m of the Earth's surface.
- Moment Magnitude (MW) of 7.8 obtained from National Seismic Hazard Model (GNS Science) website, for 1/500-year earthquake event for Wellington with a Vs30 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 7), and the cut / fill batters below the site (Table 8).

**Table 7: ULS Estimated Displacement for Slopes Above the Site**

| Section   ULS | K <sub>y</sub> | Indicative Displacement Estimate | Expected Failure Mass Material Type <sup>1</sup> | Categorial Slope Performance <sup>2, 3</sup> |
|---------------|----------------|----------------------------------|--------------------------------------------------|----------------------------------------------|
| Section A     | 0.04           | 1000+ mm                         | Brittle                                          | Very High                                    |
| Section B     | 0.00           | 1000+ mm                         | Brittle                                          | Very High                                    |
| Section C     | 0.00           | 1000+ mm                         | Brittle                                          | Very High                                    |
| Section D     | 0.00           | 1000+ mm                         | Brittle                                          | Very High                                    |

<sup>1</sup> Brittle material tends to fracture with little or no plastic movement, hence large failures can occur with low levels of displacement.

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

<sup>3</sup> Categorial Slope Performance is not an consideration of the risk that the slope posed but s instead a criterion for determining the displacement behaviour for a given seismic event.

Pseudo-static and displacement modelling methods are intentionally conservative and may overpredict deformation behaviour in complex natural slopes.

**Table 8: ULS Estimated Displacements for Cut and Fill Batters**

| Section   ULS | Batter Type                            | K <sub>y</sub> | Indicative Displacement Estimate | Expected Failure Mass Material Type <sup>1</sup> | Categorial Slope Performance <sup>2</sup> |
|---------------|----------------------------------------|----------------|----------------------------------|--------------------------------------------------|-------------------------------------------|
| Section A     | Cut                                    | 0.98           | Less than 50 mm                  | Ductile                                          | Negligible Slope Movement                 |
| Section B     | Cut                                    | 0.20           | 400 – 450 mm                     | Ductile                                          | Major Slope Movement                      |
| Section C     | Fill                                   | 0.28           | 250 mm                           | Ductile                                          | Large Slope Movement                      |
|               | Cut                                    | 0.08           | 1000+ mm                         | Ductile                                          | Severe Slope Movement                     |
| Section D     | N/A – No cut or fill batters proposed. |                |                                  |                                                  |                                           |

<sup>1</sup> Brittle material tends to fracture with little or no plastic movement, hence large failures can occur with low levels of displacement.

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

Our analysis indicates that earthquake-induced failures above the site, are likely to deform in a brittle manner, and are estimated to result in large displacement events at peak ground accelerations lower than the 1/500-year seismic event. Based on Table 18 of Slope Stability Guidance Unit 3, High and Very High Landslide hazards are described as failures that could plausibly result in evacuable failure, with a >33% likelihood of failure for displacement >100 mm. During the 2016 Kaikōura earthquake, a seismometer located above the site registered 0.073 g (10% of ULS, and 57% of SLS), which provides useful real-world evidence of slope resilience under seismic loading without inducing failure. However, the historic landslide on the site, suspected to result from the 1855 Wairarapa Earthquake was subject to much higher ground accelerations, likely in the order of 0.6–0.7g, as discussed in Dowrick (2005).

Cut and fill batters are estimated to display 'Large', 'Major', and 'Severe' Slope movements. These are defined in Unit 3 as slides that are likely to accommodate slope movement without further mobilisation, slides that are likely to result in further mobilisation, and significant slope destabilisation is likely respectively. The failure planes indicated in our analysis range in depth between 0.3 m and 4 m.

Based on the findings above, we consider that multiple engineering mitigation pathways are available to improve slope performance and reduce risk to tolerable levels. For the cut and fill batters that are proposed. Fill batters that are proposed to be located against the south-eastern boundary should consider reinforcing the slopes with geogrids. While the cut slopes above can be reduced to a lower angle or reinforced to improve their stability.

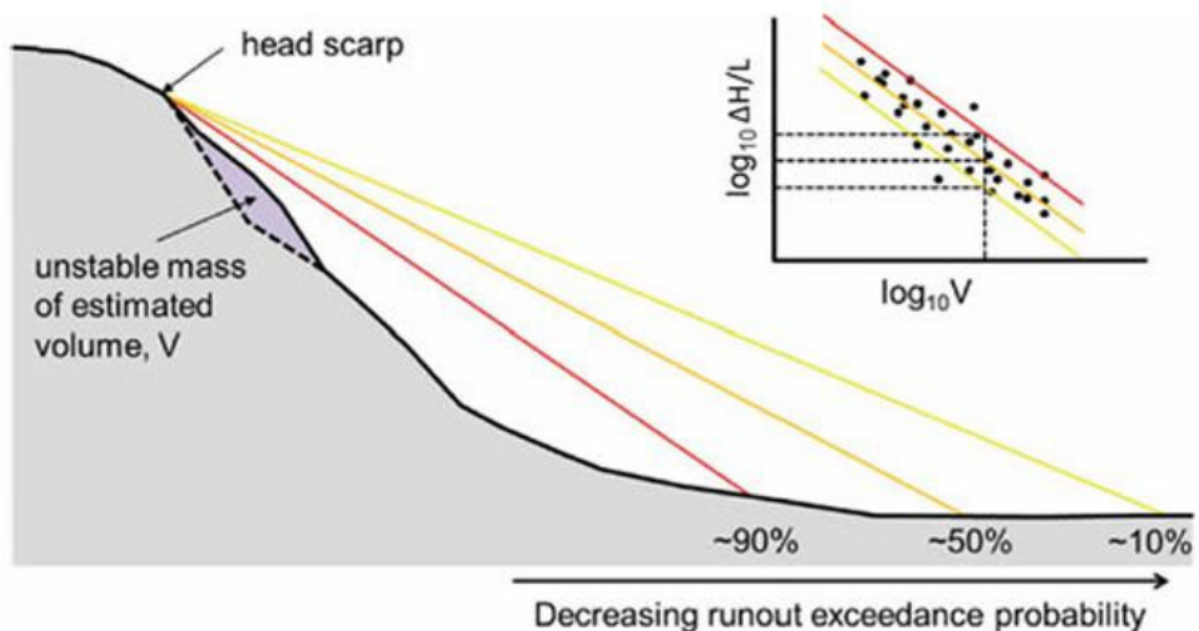
Due to the steep slopes above the site, the previous large failure on the site, predicted saturation induced failures, and the high levels of displacements estimated under seismic conditions, an analysis of the possible runout distances is required to determine if the proposed lots are at risk.

#### **7.3.8 Upslope Landslide / Debris Flow Runout Assessment**

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.

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 Method for Determining Debris Runout (Brideau et al., 2020)**



In our analysis we searched for the maximum credible volume of material that the slopes located to the northwest of the site are capable of generating<sup>(Note 1)</sup>, and given the uncertainty in the upslope soil profile we have assumed a uniform thickness of 1.0 m of colluvium is present on the face of the slopes before intercepting rock. It is however likely that some amount of rock underlying the colluvium will be residual soil or completely weathered rock. Based on the assumptions above three angles have been projected from the slopes above the site:

1. A slope angle of 29.5° which reflects the maximum runout length observed within the extent of the historic landslide.
2. Varying angle displayed in Table 9, for a 25% probability of exceedance assuming that the material is partially saturated.
3. Varying angle displayed in Table 9, for a 25% probability of exceedance given the material is completely saturated.

**Table 9: Shadow Angles for Completely and Partially Saturated Debris Flows at Maximum Credible Volumes**

| Section ID    | Maximum Volumes (m <sup>3</sup> ) | Partial Saturation | Complete Saturation |
|---------------|-----------------------------------|--------------------|---------------------|
| Section B & D | 3,000                             | 23.1°              | 28.3°               |
| Section E & F | 5,000                             | 22.3°              | 27.7°               |
| Section A     | 7,000                             | 21.9°              | 27.3°               |

The results of the shadow angle runout analysis are displayed in Appendix 7.



Lots located within the purple polygons of Appendix 7 are expected to be completely inundated by debris if a seismically induced landslide which comprises of all the available debris occurs. Extending further southeast of the slope, we anticipate that 25% of partially saturated material will exceed the area (dark blue polygon) displayed in Appendix 7. Lots that are located within this area (where 75% of debris is likely to terminate) may be effectively mitigated through engineered catch fences, debris barriers, and earth bunds. The affected lots are 10, 11, 12, 13, 49, 50, 51, 52, 53, 54, 57, 58, 59, 60, 61, 74, 75, 83, 96.

The remaining lots, excluding lots 2, 3, 4, 5, 6, 16, 17, 18, 19, 27, 28, 36, 37, 38, 39, 92, and 93, are located within the 25% probability of exceedance area for a completely saturated material, and will require some form of mitigation measure.

#### 7.4 Rockfall

During ENGEO's site walkover and geomorphic mapping, evidence of rockfall was identified at the base of multiple slopes along the north-western boundary of the proposed development and are displayed in Appendix 4.

The observed debris can be broadly classified into two groups. The first comprises abundant, clustered debris of relatively small size (approximately 50–250 mm). The second consists of larger debris, up to 2 m in size, which is more sparsely distributed.

Debris was typically covered in moss and lichen, indicating that some of the rockfall, from both groups, had been sitting at the base of the slopes for some time. Furthermore, some of the larger boulders had young trees growing around and over the top of the boulders. Some fresh rockfall debris was present with had minimal moss and lichen indicating that these areas within the bush line are still being supplied from active source areas.

The two large boulders that were observed outside of the bush line, the largest of which measured 2.6 x 1.8 x 1.4 m, were observed near the terrace crest, and were grouped next to each other. Due to the land modification that has occurred during the farming of the land since the mid-1900s, it is possible that these boulders were moved and placed together to allow for the creation of a fence line along the toe of the slope. However, there is a rock outcrop directly upslope of the debris, and an f-angle estimate undertaken while conducting the site walkover indicated a runout angle of 26°.

Several smaller boulders were noted along the terrace riser, with sizes maximum sizes between 1.2–1.8 m. The origins of these boulders is uncertain, some were rounded and some were angular. However, due to some rock outcrops being present along the crest of the terrace is possible that some of these boulders rolled down the riser. If this is the case, the proposed earthworks will reduce the terrace riser to such a level that this will no longer be a risk.

Considering the above, rockfall at the toe of the slope along the north-western boundary appears to be where the most frequent rock fall is occurring. Our observations highlight that the rockfall that appears to have occurred most recently is typically smaller in nature and is not running out past the edge of the tree line. Furthermore, the slopes are well vegetated and the most recent development plans indicate that a green space will be located along the toe of the slope, and therefore provide an additional buffer to the proposed lots from rockfall.

Based on the observations outlined above, we consider that the likelihood of debris impacting the proposed development is 'low', and that the consequences of if debris is to reach the proposed subdivision, that the consequence of impact is likely to be 'low' to 'medium' – as the active debris size is relatively small, and the proposed lots are offset further than the existing vegetation from the site.

## 7.5 Debris Flow Analysis

As discussed in Section 6.2, a gentle slope radiates from Subcatchment 1 in the semi-circular shape and has been inferred to be indicative of debris flows having previously occurred. The deposit is most likely derived from repeated, small-scale 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 Centre 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 worse 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 10.

**Table 10: 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 8; 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 1.0 m and 1 m/s, respectively. Exceptions are included in Table 11.

**Table 11: Debris Flow Scenarios - 30% Volumetric Solids Concentration Scenario**

| Lot | Approximate Inundation Percentage | Maximum Inundation Depth (m) | Maximum Debris Flow Velocity (m/s) |
|-----|-----------------------------------|------------------------------|------------------------------------|
| 62  | 50%                               | 0.35                         | 1.60                               |
| 68  | 30%                               | 1.15                         | 0.85                               |
| 72  | 20%                               | 0.60                         | 1.20                               |
| 73  | 50%                               | 0.85                         | 1.15                               |
| 75  | 60%                               | 1.10                         | 0.90                               |
| 76  | 90%                               | 1.60                         | 1.50                               |
| 77  | 90%                               | 1.55                         | 0.90                               |
| 78  | 100%                              | 1.55                         | 0.85                               |
| 90  | 40%                               | 0.35                         | 1.90                               |
| 91  | 25%                               | 0.80                         | 1.25                               |



### 60% Volumetric Solids Concentration Scenario

In general, the inundation depths and velocities reported for the 60% volumetric solids concentration scenario were limited to 1.0 m and 1.0 m/s, respectively. Exceptions are included in Table 12.

**Table 12: Debris Flow Results - 60% Volumetric Solids Concentration Scenario**

| Lot | Approximate Inundation Percentage | Maximum Inundation Depth (m) | Maximum Debris Flow Velocity (m/s) |
|-----|-----------------------------------|------------------------------|------------------------------------|
| 2   | 50%                               | 1.2                          | 0.70                               |
| 28  | 100%                              | 1.15                         | 0.35                               |
| 62  | 75%                               | 0.55                         | 1.10                               |
| 67  | 100%                              | 1.10                         | 0.20                               |
| 68  | 30%                               | 1.35                         | 0.15                               |
| 74  | 60%                               | 1.10                         | 0.30                               |
| 75  | 75%                               | 1.30                         | 0.30                               |
| 76  | 90%                               | 1.75                         | 0.50                               |
| 77  | 90%                               | 1.70                         | 0.25                               |
| 78  | 100%                              | 1.70                         | 0.20                               |
| 86  | 80%                               | 0.30                         | 2.00                               |
| 88  | 100%                              | 0.55                         | 1.15                               |
| 89  | 100%                              | 0.50                         | 1.35                               |
| 90  | 100%                              | 0.70                         | 1.25                               |

### 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 $I_{DF}$<br>(note 1) |
|--------------------------------------------|-------------------------------------------------------------------------------------------------------------------|------------------------------|
| <b>Class I – Some Sedimentation</b>        | Sediment-laden water ingresses buildings main floor or basement, requiring renovation. Up to 25% insured loss     | Less than 1                  |
| <b>Class II – Some structural damage</b>   | Some supporting elements damaged and could be repaired with major effort; 25 – 75% insured loss                   | 1 - 10                       |
| <b>Class III – Major structural damage</b> | Damage to foundation piles, pillars and will likely to require complete building reconstruction >75% insured loss | 10 - 100                     |
| <b>Class IV – Complete destruction</b>     | 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 |     |                   |                                  |                                  |                  |
|--------------------------------------------------------|-----|-------------------|----------------------------------|----------------------------------|------------------|
| <b>Class IV – Complete destruction</b>                 | 0   | 6                 | 25                               | 67                               | 100              |
| <b>Class III – Major structural damage</b>             | 0   | 22                | 38                               | 28                               | 0                |
| <b>Class II – Some structural damage</b>               | 30  | 50                | 37                               | 5                                | 0                |
| <b>Class I – Some Sedimentation</b>                    | 70  | 22                | 0                                | 0                                | 0                |
| <b><math>I_{DF}</math></b>                             | 0-1 | 1-10 <sup>1</sup> | 10 <sup>1</sup> -10 <sup>2</sup> | 10 <sup>2</sup> -10 <sup>3</sup> | >10 <sup>3</sup> |

The IDF values were calculated for each lot (Appendix 9). IDF values from both scenarios in exceedance of 0.4 are shown in Table 13. If the IDF value exceeds 0.4 in both scenarios, the greatest value was selected.

**Table 13: Debris Flow Scenarios**

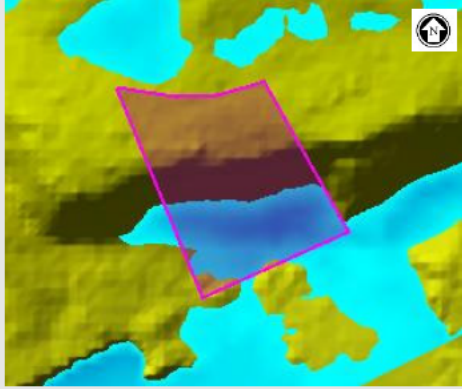
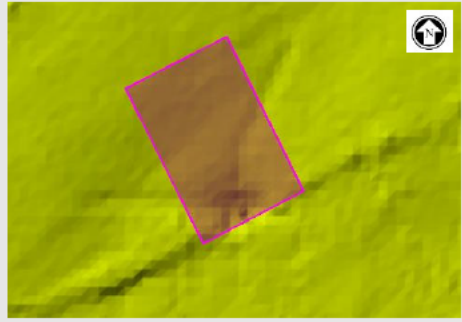
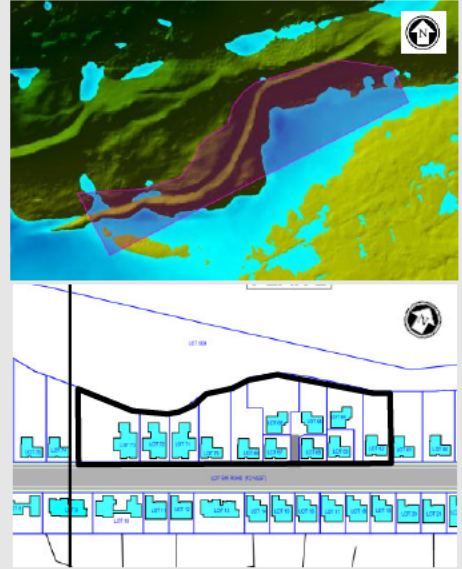
| Lot | IDF  | Damage Class                      | Damage Class Probability                                                                                               |
|-----|------|-----------------------------------|------------------------------------------------------------------------------------------------------------------------|
| 2   | 0.59 | Class I – Some Sedimentation      | 70% - Class I – Some Sedimentation<br>30% - Class II – Some Structural Damage                                          |
| 28  | 0.56 |                                   |                                                                                                                        |
| 62  | 0.90 |                                   |                                                                                                                        |
| 68  | 0.83 |                                   |                                                                                                                        |
| 72  | 0.86 |                                   |                                                                                                                        |
| 73  | 1.12 | Class II – Some Structural Damage | 22% - Class I – Some Sedimentation<br>50% - Class II – Some Structural Damage<br>22% Class 3 – Major Structural Damage |
| 74  | 0.47 | Class I – Some Sedimentation      | 70% - Class I – Some Sedimentation<br>30% - Class II – Some Structural Damage                                          |
| 75  | 0.89 |                                   |                                                                                                                        |
| 76  | 3.60 | Class II – Some Structural Damage | 22% - Class I – Some Sedimentation<br>50% - Class II – Some Structural Damage<br>22% Class 3 – Major Structural Damage |
| 77  | 1.26 |                                   |                                                                                                                        |
| 78  | 1.12 |                                   |                                                                                                                        |
| 86  | 1.20 |                                   |                                                                                                                        |
| 88  | 0.73 | Class I – Some Sedimentation      | 70% - Class I – Some Sedimentation<br>30% - Class II – Some Structural Damage                                          |
| 89  | 0.91 |                                   |                                                                                                                        |
| 90  | 1.26 | Class II – Some Structural Damage | 22% - Class I – Some Sedimentation<br>50% - Class II – Some Structural Damage<br>22% Class 3 – Major Structural Damage |
| 91  | 1.25 |                                   |                                                                                                                        |

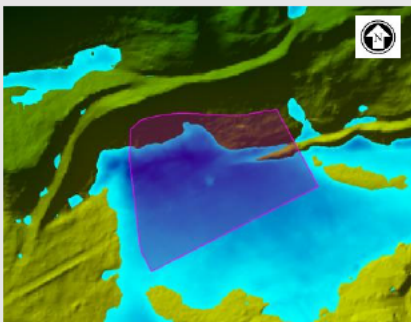
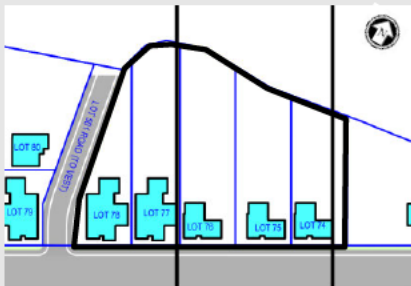
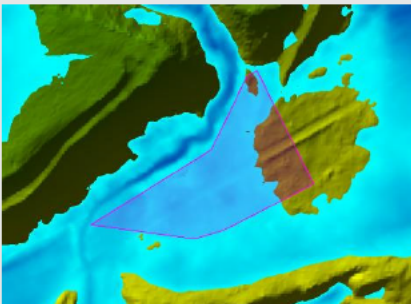
#### 7.5.6 Risk Assessment

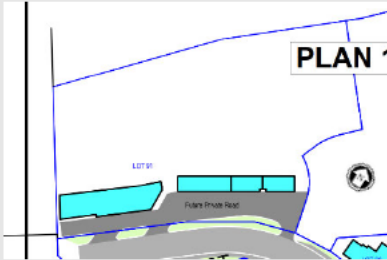
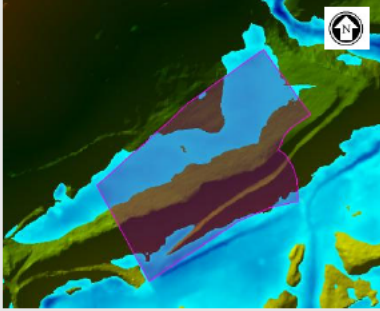
The Damage Class and Damage Class Probability were used as a starting point for risk assessment for debris flow. It was concluded that lots with an IDF below 0.40 are firmly within the Class I range; additional assessment was not considered necessary. Lots with IDF values above 0.40 are included in Table 14. The architectural plans (dated 15 June 2026) were used to provide additional context for risk assessment.



Table 14: Risk Assessment Discussion

| Lot | IDF  | Location                                                                            | Discussion                                                                                                                                                                                                                                                                                                                                                                                                                                                                          |
|-----|------|-------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 2   | 0.59 |    | <p>Lot 2 contains a steep drop off from an upper terrace to a lower terrace. Following earthworks, we understand the lot will not contain a steep gradient.</p> <p>The existing civil plans show stormwater intakes in the road north of the site.</p> <p><b>The final civil cut / fill plan and stormwater infrastructure design will need to be reviewed to determine if additional debris flow mitigation measures are required.</b></p>                                         |
| 28  | 0.56 |   | <p>Ponding at Lot 28 was significant because of the terrain variation and low point. Following earthworks, the lot is likely to be relatively flat</p>                                                                                                                                                                                                                                                                                                                              |
| 62  | 0.90 |  | <p>Lots 62, 68, 72, and 73 back up to the toe of the existing mountain slope. The terrain indicates that a natural channel may be present within the site running parallel to the northern boundary.</p> <p>Following earthworks, we understand the lot will not contain a steep gradient.</p> <p><b>The final civil cut / fill plan and stormwater infrastructure design will need to be reviewed to determine if additional debris flow mitigation measures are required.</b></p> |
| 68  | 0.83 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |
| 72  | 0.86 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |
| 73  | 1.12 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                     |

|    |      |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
|----|------|-------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 74 | 0.47 |    | <p>The source of the inundation to the building platforms is runoff from the terrace north of the lots. Once the stormwater reaches the proposed housing platforms, it becomes largely stationary and occupies a low region in the existing terrain. Extensive cut / fill works are planned for this area and will impact the flow of stormwater.</p> <p><b>The final civil cut / fill plan and stormwater infrastructure design will need to be reviewed to determine if additional debris flow mitigation measures are required.</b></p> |
| 75 | 0.89 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 76 | 3.60 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 77 | 1.26 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 78 | 1.12 |    |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 86 | 1.20 |  | <p>Review of the time series data in the debris flow model indicates that inundation occurs at Lots 86, 88, 89, and Lot 90 when the creek north of the sites breaches its banks. Construction of a culvert (blue arrow) is proposed to carry stormwater from the creek, which will significantly reduce inundation risk.</p>                                                                                                                                                                                                               |
| 88 | 0.73 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 89 | 0.91 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
| 90 | 1.26 |                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |

|    |      |                                                                                   |                                                                                                                                                                                                                                                                                                                                                                      |
|----|------|-----------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
|    |      |  | <p>The majority of inundation at Lot 117 occurs in the northern portion of the lot, which will remain green space. Construction of a culvert (blue arrow in image above) is proposed to carry stormwater from the creek, which will significantly reduce inundation risk to the southern portion of the site.</p>                                                    |
| 91 | 1.25 |  | <p>The majority of Lot 91 is green space. Inundation to the building platform is limited and the majority of inundation is a result of ponding on the existing access road.</p> <p><b>The final civil cut / fill plan and stormwater infrastructure design will need to be reviewed to determine if additional debris flow mitigation measures are required.</b></p> |

## 8 Recommendations

### 8.1 Preliminary Foundation Recommendations

Based on our analysis, most of the site will not require any Specific Engineered Design (SED) for foundations. However, the crest of the fill batter that runs the length of the south-eastern boundary is estimated to displace by up to 250 mm and extend back 2.0 m from the crest of the batter during a ULS seismic event and this area should be avoided for foundation placement.

Any lot that is proposed to be established within the footprint of the historical landslide should be subject to Specific Engineering Design (SED) given the weak / loose nature of the material.

We recommend that each lot have a suitably qualified geotechnical engineer undertake an assessment of the ground conditions 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 0.3 and 1.2 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.

### 8.3 Earthworks

Across the site, a total earthwork area of 69,030 m<sup>2</sup> is proposed, which is comprised of 30,770 m<sup>2</sup> of cut and 38,260 m<sup>2</sup> of fill. Within these areas 110,600 m<sup>3</sup> of cut is proposed and 94,680 m<sup>3</sup> of fill is proposed (site-won material); therefore, leaving an excess of 15,920 m<sup>3</sup>.

ENGEO has reviewed the design, with analysis being undertaken on some key elements, such as the cut and fill batters proposed. Failure in the cut and fill batters is predicted to occur under high levels of seismic-induced ground shaking. We recommend undertaking an individual assessment and design of the proposed batters by a suitably qualified Geotechnical Engineer.



### 8.3.1 Topsoil and 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 should be protected from overland water flow by the additional 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.

ENGEO should be given the opportunity to review any changes in the final earthworks plans prior to work 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. 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 5 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 erosion 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.

Cut batters located beneath high-voltage transmission lines will need specific engineering design and must be done in collaboration with the asset owner, and in compliance with relevant standards such as NZECP34.

#### 8.4 Retaining Walls

At the south-western extent of the site, a 14 m deep cut is proposed to be undertaken to reduce the height of the terrace batter, this cut will provide material to be used in the cut to fill. However, at the toe of the batters seven to eight metre high near vertical batters are illustrated in the Vecta drawings. No comment is present in the Vecta drawings, however, given the height of these batters and the structures which are proposed to be established at the base of the slope, significant retaining walls will be required.

Given the proposed height of the near vertical batters, steel posts or anchor shotcrete systems may be required to retain the slopes above the proposed development.

All retaining walls will require site specific investigations and detailed design to account for material composition, active wedge loading, and the geohazards outlined in Section 9.

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 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 stormwater network underlaying the proposed roadways. A culvert is present on the western portion of the subdivision, between Lots 90 and 91, which conveys stormwater from Subcatchment 1 through the subdivision to the south.

A rain garden is proposed for construction between of Lot 99 and 41 and will receive water from the subdivision stormwater system before discharging to the Hutt River. Stormwater channels are not depicted at the northern subdivision boundary at the toe of the hillside. It is recommended that stormwater channels are constructed on this boundary to protect the northern lots from stormwater and debris flow inundation.

#### 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 lot boundaries and areas with steep gradients. Stormwater and / or sediment-laden water should be captured at the toe of the slope and directed toward a culvert.

The final civil cut / fill plan and stormwater infrastructure design will need to be reviewed to determine if additional debris flow mitigation measures are required.

## 9 Safety in Design

ENGEO considers the items outlined in Table 15 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 15: Safety in Design**

| Risk                                    | Risk Rating | Potential Control Measures                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                          |
|-----------------------------------------|-------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Large Cuts > 6.0 m                      | Significant | <p>Select appropriate temporary cut angles to reduce risk of slope failure.</p> <p>Implement H&amp;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.</p> <p>Engineered temporary stability measures if other solutions are not viable.</p> <p>Implement safe construction methodology to lower risk to workers.</p> <p>Do not work above or below large cuts immediately following heavy rainfall (should consider geotechnical assessment following heavy rainfall).</p> |
| Steep Slopes (above and below the site) | Moderate    | <p>Contractor shall not carry out works on, immediately above, or immediately below slopes during or immediately after heavy rainfall.</p> <p>Complete planting work within embankment / batter faces during construction, rather than at the end.</p> <p>A geotechnical professional should assess the stability of unretained slopes on-site during construction.</p>                                                                                                                                                                                             |
| Streams and standing water              | Moderate    | <p>Silt fences constructed around stream channels and wetland areas to reduce thoroughfare.</p> <p>Stay away from channels during elevated rainfall.</p> <p>Complete a comprehensive water management plan prior to undertaking earthworks.</p>                                                                                                                                                                                                                                                                                                                     |

## 10 Sustainability

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 RMA 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 and the National Policy Statement for Natural Hazards 2025 (NPS-NH). 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**

|                   |              | Likelihood Level |             |           |          |            |               |           |
|-------------------|--------------|------------------|-------------|-----------|----------|------------|---------------|-----------|
|                   |              | Almost Certain   | Very Likely | Likely    | Possible | Unlikely   | Rare          | Very Rare |
| Consequence Level | 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 16, to determine based on the above-mentioned methodology, whether the development is likely to increase the likelihood of the hazards occurring.

Table 16: 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                                                                                                                                            |
|--------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------|-----------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Active Faults                              | Primary and secondary hazards due to proximity and unconstrained location of the fault relative to the site.<br><b>Low</b>                                                                                                                                                                                                                                                    | None                                                                                                                                                                                                                                                                                         | <b>Low</b>                                                 | No                                                                                                  | See Section 7.1 for further discussion.                                                                                                                        |
| Liquefaction                               | Ground settlement causing damage to dwellings and infrastructure<br><b>Low</b>                                                                                                                                                                                                                                                                                                | None                                                                                                                                                                                                                                                                                         | <b>Low</b>                                                 | 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.<br><b>Low</b>                                                                                                                                                                                                                                                                                               | Use of geogrid within the fill batters to increase stability of batter.<br>Use fill which is conducive to batter construction to 6 m in height.<br>Use lightweight foundations (Rib-Raft), or design foundations which can tolerate displacement.<br>Build away from the crest of the slope. | <b>Low</b>                                                 | No                                                                                                  | See discussion in Section 7.3 on slope stability.                                                                                                              |
| Slope instability above and below the site | <u>Lots Above Fill Batter</u> : Undermining foundations / collapse of building from slope below.<br>Seismic-induced differential settlement across foundations.<br><b>Low (given recurrence interval of Ultimate Limit State seismic event)</b>                                                                                                                               |                                                                                                                                                                                                                                                                                              |                                                            |                                                                                                     |                                                                                                                                                                |
|                                            | <u>Lots Below Slopes</u> :<br>Inundation / destruction of property from evacuated material from the slope above, injury and probable fatalities and / or critical injuries.<br><b>Medium (given the consequence and that lots are proposed to be established at the toe of the slope. Additionally, there is evidence that a failure has previously occurred on the site)</b> | Hard engineering solutions are not likely to provide suitable mitigation against maximum credible landslide volumes (>3,000 m <sup>3</sup> ) from the slope above.                                                                                                                           | <b>Medium (Considered tolerable - refer to Section 12)</b> | No – however if the toes of the slopes above the site are undermined and left unsupported then yes. | Stability of the slopes above and below the site are discussed in Section 7.3.<br>Runout analysis discussed in Section 7.3.8                                   |
| Rockfall                                   | Damage to property<br><b>Low</b>                                                                                                                                                                                                                                                                                                                                              | Not applicable / proposed green space acting as a buffer / rockfall catch fence.                                                                                                                                                                                                             | <b>Low</b>                                                 | No                                                                                                  | The average size of rockfall is small, and isolated to a couple of areas which are not located within any of the proposed lots (see discussion in Section 7.4) |
| Debris Flow                                | Sediment inundation of buildings and stormwater system. Structural damage is generally unlikely. Structural damage at Lots 73, 76, 77, 78, 86, 90 and 91 is possible.<br><b>Low</b>                                                                                                                                                                                           | Surface regrading and 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.                                                                                              | <b>Low</b>                                                 | No                                                                                                  | The apex of the gullies is well vegetated and display little recent evidence of debris flows (further discussion in Section 7.5)                               |

## 12 Societal Risk Assessment

As outlined in Section 11, the risk of a seismically induced landslide from above the subdivision is considered to be Medium under the NPS. This framework requires, for this level of risk, that the hazard must either be mitigated against or avoided. However, approximately 8% of the proposed lots are located in close proximity to the western hillslopes (refer Section 7.3.8). This has prompted further consideration of this hazard risk as a check on, or calibration of, the qualitative risk rating under the NPS-NH framework, specifically a seismic event triggering a comparable landslide to the one observed (Section 6.2) on the same or nearby slopes if development proceeds.

Societal risk considers the risk of loss of life within a population (rather than an individual) due to a specific hazard event, or category of hazard events (in this case, seismically induced landslides). It is generally defined as the product of the frequency (F) of occurrence of a specific hazard and the number of deaths (N) that are expected as a consequence of the hazard. Societal risk assessment is usually restricted to events potentially capable of causing large-scale loss of life.

### 12.1 Methodology and Assumptions

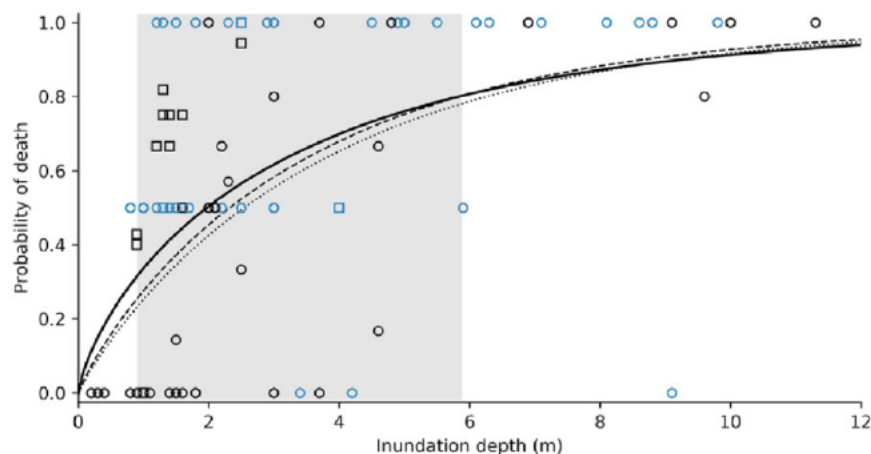
Societal risk for the lots located below the western hillslope was assessed as follows:

1. Consideration of three separate events, each with a varying level of intensity as outlined in the National Seismic Hazard Model (v1.0.4; released 2022). The three seismic ground accelerations tested were:
  - a. Serviceability Limit State equivalent event with a peak ground acceleration of 0.13 g,
  - b. An earthquake event approximately equivalent to the 1855 Wairarapa Earthquake, which is the possible initiating event for the landslide identified on the western hillslope (refer Section 4.3, and Section 6.2).
  - c. An extreme scenario of 1.6g (which was experienced in some areas following the 2016 Kaikōura Earthquake).
2. The probability of slope failure along the western hills of the Hutt Valley and Wellington has been assessed based on a series of published documented failures, along with undocumented failures, as a percentage of total hill length following the 1855 Wairarapa Earthquake; where the known sizes of small- to moderate- and large-scale landslides have been extrapolated to the other documented events. This value was adjusted based on the magnitude of the ground acceleration being analysed for a given ground acceleration scenario.
3. For each scenario, the probability of reaching the subdivision was assessed based on the measured runout angle (29.5°) of the existing landslide referred to above. In the SLS-equivalent scenario, the likelihood that material reaches the subdivision was assumed to be significantly lower, as we have had a series of 50% SLS events which have not elicited any obvious failures within the slopes above the site.



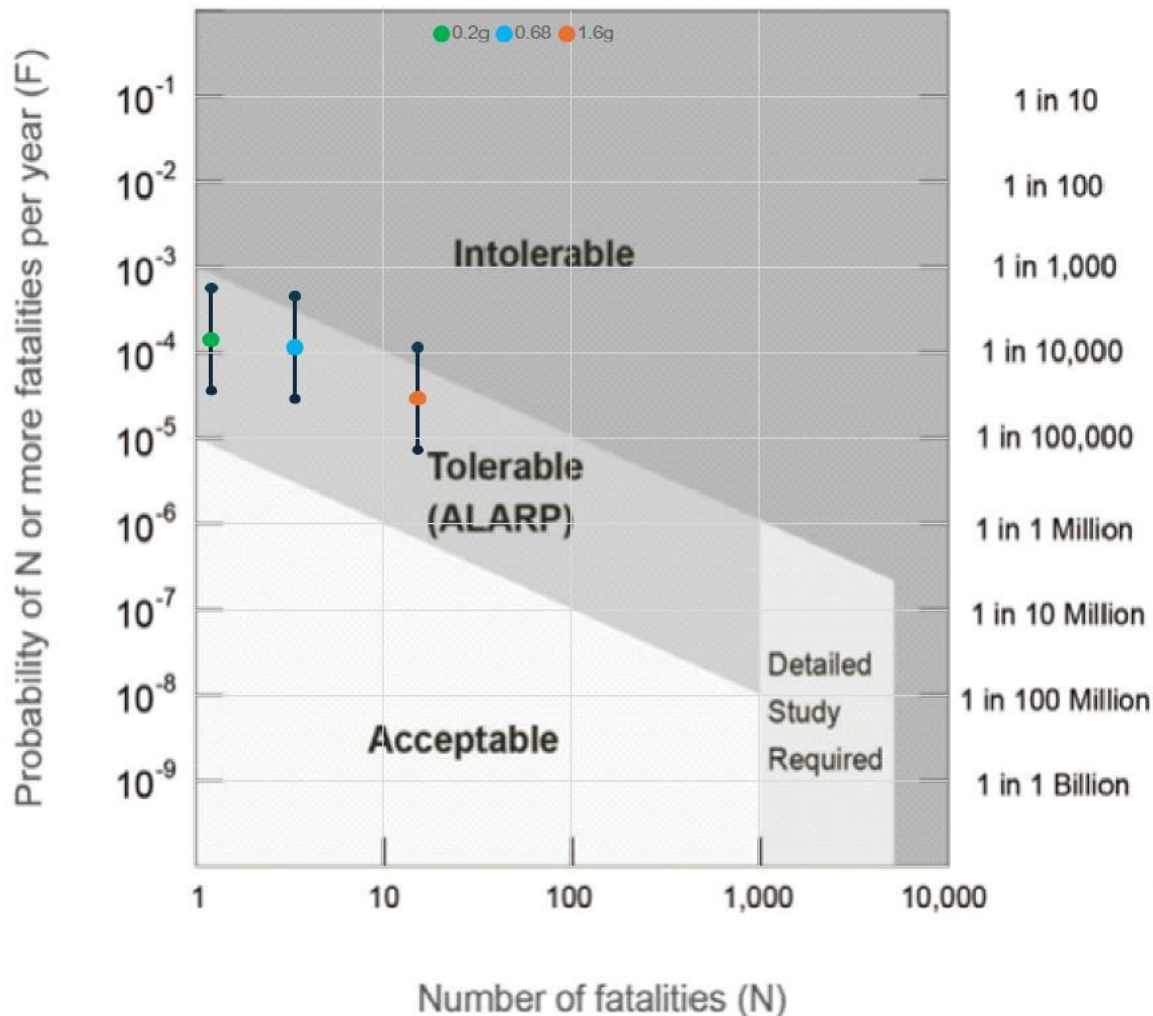
4. Whether a specific property is likely to partially or fully inundated has been determined on what percentage of the proposed construction location of the dwelling is to be located within the 29.5° runout area. If greater than 50% of the proposed construction area of a dwelling is located within the runout area, it was considered that this property will be fully inundated. If less than 50% of the lot area lies within the proposed runout area, then the property was considered to be partially inundated.
5. For properties where full inundation is modelled, we considered the likelihood that a fatality occurs based on Pollock and Wartman (2020). This paper considers human vulnerability to landslides based on a series of global case studies and determined a relationship between fatality probability and inundation depth (Figure 10). A depth of inundation of approximately 6.0 m has been assessed by cutting a series of cross sections through the historic landslide at the locations dwellings and comparing the ground elevation to the surrounding terrace level.

**Figure 10: Human Vulnerability Curve Based on Inundation Depth from Pollock and Wartman (2020)**



6. For lots where partial inundation is likely, a lower inundation depth was assumed, as demonstrated by the inundation height towards the edges of the lobe of the historical landslide. A probability of death of 0.3 was assumed.
7. Average occupancy rates have been assigned to properties depending on the intended number of bedrooms per property, with 2.5 occupancy for three-bed houses, and 3.5 for four-bed houses, in line with 2023 Census data. The total exposed population was then calculated by multiplying the number of partially and completely inundated lots by these occupancy rates.
8. This process was repeated for the three seismic scenarios, resulting in the F/N pairs displayed in Figure 11. Levels of societal risk have been calculated using event probability trees which are displayed in Appendix 9.

**Figure 11: F/N Curve for Proposed Lots Below the Slopes Above the Site Stage 4. (Societal tolerability framework from de Vilder et al., (2022))**



## 12.2 Uncertainty

All risk assessments are based on several assumptions and have inherent uncertainty. In quantitative risk (probability) this uncertainty can be communicated as confidence limits centered on the median (or best estimate).

Using the system developed by Paul and Miner (2025) and outlined in AGS/NZGS (in prep) an uncertainty index ( $I_e$ ) of 17 has been assessed. For a landslide of moderate complexity, this implies an error range between the upper and lower probability of loss of life of 1.2 orders of magnitude, or +/- 0.6 orders of magnitude from the best estimate. This calculated uncertainty is reflected in the error bars in Figure 11.

## 12.3 Risk Tolerability

There is no globally accepted criteria for assessing the tolerability of risk of loss of life to an individual or group of individuals. The tolerability framework presented in de Vilder et al., (2022), is one example based a compilation of societal risk tolerance criteria.

Based on the societal risk assessment presented in de Vilder et al (2022), the risk to persons in properties within the 29.5° area from the crest of the slopes above the site is tolerable. However, appropriate measures need to be in place to keep the risk level as low as reasonably practicable (ALARP).

Several options can be adopted to decrease the assessed levels of risk in accordance with the ALARP philosophy. Measures include:

- Increased level of engineered mitigations.
- Implementing structural and design considerations within houses to increase 'survivability' in line with Pollock and Wartman (2020). Measures here could include designing the upslope wall of houses to allow for impact load, limiting windows on the upslope side of houses, designing houses as multiple level structures with upstairs bedrooms etc

## 13 Conclusions

We conclude that, although there are considerable risks posed to the site, the risk to the proposed buildings from natural hazards may be considered to be at a tolerable level of risk once suitable mitigation measures, as described in this report, have been designed and implemented.

The site presents geotechnical and natural hazard considerations typical of Wellington hill developments. Subject to refined design, targeted mitigation measures, and detailed geotechnical input during subsequent design stages, we consider that the site may be capable of supporting the proposed development.

## 14 Future Work

The primary element of the site which requires further investigation is rainfall induced slope failure above the subdivision. We have estimated a thickness of soil which may be present above the rock surface.

The thickness of colluvial material heavily influences the possible volumes of material which can fail and our current analysis suggests that runout may extend across significant parts of the subdivision. Hand excavations undertaken in the slopes above the site would provide valuable information in the detailed design of landslide mitigations for lots proposed to be developed along the north-western boundary of the site.

It is likely that further borehole investigation will additionally be required as part of the detailed design process to (a) confirm subsurface conditions and (b) discount the possibility of liquefiable material at depth below the subdivision.

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.



- A suitably qualified Geotechnical Engineer should be consulted for the design of landslide runout mitigations along the northwest of the site.
- Material suitability testing for reusability should be undertaken.
- Stability assessment of cut and fill batters needs to be undertaken once a better understanding of the material properties has been obtained, or during lot specific investigations with the purpose of establishing foundation types.

## 15 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.

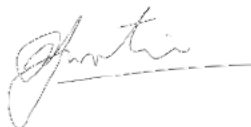
Report prepared by



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Engineering Geologist

Report reviewed by



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Principal Engineering Geologist

## 16 References

- AGS (2007a). "Guideline for landslide susceptibility, hazard and risk zoning for land use planning". Australian Geomechanics Society, Australian Geomechanics, Vol 42 No 1, March 2007.
- AGS (2007b). "Commentary on guideline for landslide susceptibility, hazard and risk zoning for land use planning". Australian Geomechanics Society, Australian Geomechanics, Vol 42 No 1, March 2007.
- AGS/NZGS (in preparation) Landslide Risk Management: Part 4: Guidelines for Landslide Risk Assessment
- Anderson, D. G., Martin, G. R., Lam, I., & Wang, J. N. (2008). Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments. Transportation Research Board, National Research Council.
- Baynes, F. J. and Parry, S. 2022. Guidelines for the development and application of engineering geological models on projects. International Association for Engineering Geology and the Environment (IAEG) Commission 25 Publication No. 1, 129 pp.
- Begg, J. G; Mazengarb, Colin; Institute of Geological & Nuclear Sciences Limited Lower Hutt, N.Z. : Institute of Geological and Nuclear Sciences, 1996
- Blake, T. F., Hollingworth, R. A., Stewart, J. P., Center, S. C. E., & Engineers, A. S. of C. (2002). Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California. Southern California Earthquake Center, University of Southern California.
- Brideau, M. A., de Vilder, S., Massey, C., Mitchell, A., McDougall, S., & Aaron, J. (2020, November). Empirical relationships to estimate the probability of runout exceedance for various landslide types. In *Workshop on World Landslide Forum* (pp. 321-327). Cham: Springer International Publishing.
- Cardno. 2019. Wellington Water's Reference Guide for Design Storm Hydrology.
- Chow, 1959. Manning's n for Channels.
- De Vilder SJ, Kelly SD, Buxton RB, Allan S, Glassey PJ. (2024) Landslide planning guidance: reducing landslide risk through land-use planning. Lower Hutt (NZ): GNS Science miscellaneous; 144. <https://doi.org/10.21420/R2X8-FJ49>
- Dowrick, D. J. (2005, September). Strong ground shaking in the 1855 Wairarapa Earthquake. In *The 1855 Wairarapa Earthquake Symposium* (p. 79).
- Hancox, G. T. (2005, September). Landslides and liquefaction effects caused by the 1855 Wairarapa earthquake: then and now. In *The 1855 Wairarapa earthquake symposium* (Vol. 150). Wellington: Greater Wellington Regional Council.
- LINZ. (2025). Retrolense. [Dataset]. Retrieved from <https://retrolens.co.nz/#/main>
- LINZ. (2025). New Zealand LiDAR 1 m Digital Elevation Model (DEM). <https://data.linz.govt.nz/data/>
- Little, T. A., & Rogers, D. W. (2005, September). Co-seismic slip during the 1855 earthquake, southern Wairarapa Fault, New Zealand. In *The 1855 Wairarapa Earthquake Symposium* (Vol. 8, No. 10). Greater Wellington Regional Council.



Morgenstern, R.; Rosser, B.J.; Lukovic, B.; Zoeller, A.; Scadden, P.G.; Cox, S.C. 2025 The New Zealand Landslide Database version 2: a data dictionary. Lower Hutt, N.Z.: *Earth Sciences New Zealand. GNS Science report 2024/49*. 42 p.; [doi: 10.21420/4Z6H-1F97](https://doi.org/10.21420/4Z6H-1F97)

New Zealand Geotechnical Society. (2025). *Slope Stability Geotechnical Guidance Series: Unit 3 Slope Stability Analysis*. Issued for Draft.

New Zealand Geotechnical Society. (2026). *Slope Stability Geotechnical Guidance Series: Unit 6 Debris Flow Assessment, Analysis and Mitigation*.

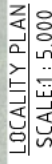
Paul, D; Miner, A (2025). *Practical communication of uncertainty in quantitative landslide risk assessment*. Australian Geomechanics Society, Vol 60, no 1. March 2025.

Van Dissen, R. J., Litchfield, N. J., & Begg, J. G. (2005). *Upper Hutt City fault trace project*. Institute of Geological & Nuclear Sciences.

## **Appendix 1:** Architectural Master Plans



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| Title | COVER SHEET | Classification | CIVIL           |             | Rev. |
|       |             |                | Master Plan No. | Drawing No. |      |
|       |             |                | 26014           | C000        | D    |

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