ENGEO Limited

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**Project Number 20190.000.001**

# Geotechnical Investigation Report

Forestry Road, Riverhead, Auckland

Submitted to:
Rangitoopuni Developments LP
C/- Avant Group Limited
[redacted]

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# Executive Summary

This site is considered to be geotechnically suitable for the proposed development provided the Auckland Council Code of Practice (ACCoP) v.2.0 guidelines and the recommendations of this report are followed.

The table below provides a summary of geotechnical design parameters appropriate for this site. These should not be taken in isolation but instead considered in context of the recommendations outlined in the relevant sections referenced below.

### Geotechnical Design Parameters

| Relevance | Parameter | | Design Value | Report Section |
|---|---|---|---|---|
| Shallow Foundations | Preliminary Geotechnical ultimate bearing capacity | | 300 kPa | 14.2 |
| | Preliminary Expansive Site Class (NZS3064) | | H1 (High) | 14.1 |
| | Seismic Site Class (NZS1170) | | C | 9.1.3 |
| Retaining Wall Design / Lateral Loads | Engineered Cohesive Fill | Undrained shear strength (kPa) | 110 | 14.4 |
| | | Friction Angle (degrees) | 32 | |
| | | Unit weight (kN/m$^3$) | 18 | |
| | Albany Conglomerate Soils | Undrained shear strength (kPa) | 75 | 14.4 |
| | | Friction Angle (degrees) | 28 | |
| | | Unit weight (kN/m$^3$) | 18 | |
| | Takaanini Formation Alluvium | Undrained shear strength (kPa) | 90 | 14.4 |
| | | Friction Angle (degrees) | 29 | |
| | | Unit weight (kN/m$^3$) | 17 | |
| | ECBF Residual Soils | Undrained shear strength (kPa) | **100** | 14.4 |
| | | Friction Angle (degrees) | 30 | |
| | | Unit weight (kN/m$^3$) | 18 | |
| Slope Stability | Slope Stabilising Measures | | MSE Engineered Slopes & Inground Palisade walls | 12.0 |
| Pavement Design | California Bearing Ratio (CBR) | | 2 – 3 % | 14.3 |

Geotechnical hazards pertinent to this site, earthworks and the proposed lightweight dwellings are summarised in the Table below.

**Geotechnical Risk Matrix**

| Geotechnical / Geological Hazard | Risk Matrix Rating (Section 2.4.3 ACCoP) | |
|---|---|---|
| | Pre-Construction | Post Construction |
| Liquefaction | Likelihood = 3; Consequence = 1<br>**Risk = 3 (Low)** | Likelihood = 2; Consequence = 1<br>**Risk = 2 (Very low)** |
| Settlement and Compressible Soils | Likelihood = 4; Consequence = 3<br>**Risk = 12 (High)** | Likelihood = 2; Consequence = 2<br>**Risk = 4 (Low)** |
| Expansive Soils | Likelihood = 3; Consequence = 3<br>**Risk = 9 (Medium)** | Likelihood = 3; Consequence = 1<br>**Risk = 3 (Low)** |
| Sensitive Soils | Likelihood = 5; Consequence = 1<br>**Risk = 5 (Medium)** | Likelihood = 1; Consequence = 1<br>**Risk = 1 (Very low)** |
| Collapsible Soils | Likelihood = 1; Consequence = 1<br>**Risk = 1 (Very low)** | Likelihood = 1; Consequence = 1<br>**Risk = 1 (Very low)** |
| Landslide Susceptible Ground / Slope Stability / Slippage | Likelihood = 4; Consequence = 3<br>**Risk = 12 (High)** | Likelihood = 2; Consequence = 2<br>**Risk = 4 (Low)** |
| Soil Erosion | Likelihood = 4; Consequence = 2<br>**Risk = 8 (Medium)** | Likelihood = 3; Consequence = 1<br>**Risk = 3 (Low)** |
| Uncontrolled Fill | Likelihood = 2; Consequence = 3<br>**Risk = 6 (Medium)** | Likelihood = 1; Consequence = 2<br>**Risk = 2 (Very low)** |

The risks posed by the presence of expansive soils will be managed through design of foundations suitable to accommodate the design expansive site class. Sensitive soils will be managed through construction and will not pose a risk to site once developed. Existing uncontrolled fills will be managed during construction and will be either remediated or cut to waste.

Consolidation settlements will be managed by adopting the appropriate foundation solution for the proposed building loads / platforms.

The risk of slope instability will be managed through implementation of slope stabilising measures and separation from existing geomorphic features. This will be implemented through construction of in-ground palisade walls, drainage and MSE engineered slopes.

# 1 Introduction

ENGEO Ltd was requested by Rangitoopuni Developments LP C/- Avant Group Limited to prepare a Geotechnical Investigation Report for a proposed retirement village at Forestry Road, Riverhead, (herein referred to as 'the site', shown in Figure 1).

The purpose of this assessment is to support a fast-track Resource Consent application for the proposed retirement village within Stage 2 of the Riverhead Forest development. This work has been carried out in accordance with our signed agreement dated 04 December 2024 [P2022.000.641_06].

ENGEO completed a desktop study, geomorphological site walkover and intrusive investigation across the site from February to March 2025. The Investigation included drilling of hand auger boreholes, machine boreholes and Cone Penetration Testing.

The results of our assessment comprise a site-wide engineering geological model, geomorphology plans, slope stability analysis, preliminary settlement analysis, an assessment of liquefaction susceptibility and preliminary building development recommendations.

# 2 Site Description

The site is situated north of the Auckland townships of Huapai, Kumeū, and Riverhead. The proposed retirement village includes the development of 32.3 hectares of the overall 173-hectare land parcel located in the south-eastern extent of the portion of Riverhead Forest accessed by Forestry Road from the southwest (legal description LOT 2 DP 590677). The site was formerly covered by dense Pinus Radiata plantations cultivated for commercial forestry, which have recently been harvested and replanted. It is now largely covered by young trees and shrubs. Within the site boundaries there are several unnamed gravel access roads.

The site is bordered to the south by large (predominately), cleared land parcels that contain residential dwellings, to the west by Deacon Road (a gravel road), and to the east by dense residential neighbourhoods. The surrounding land to the north consists of Pinus radiata plantations and pockets of native forest.

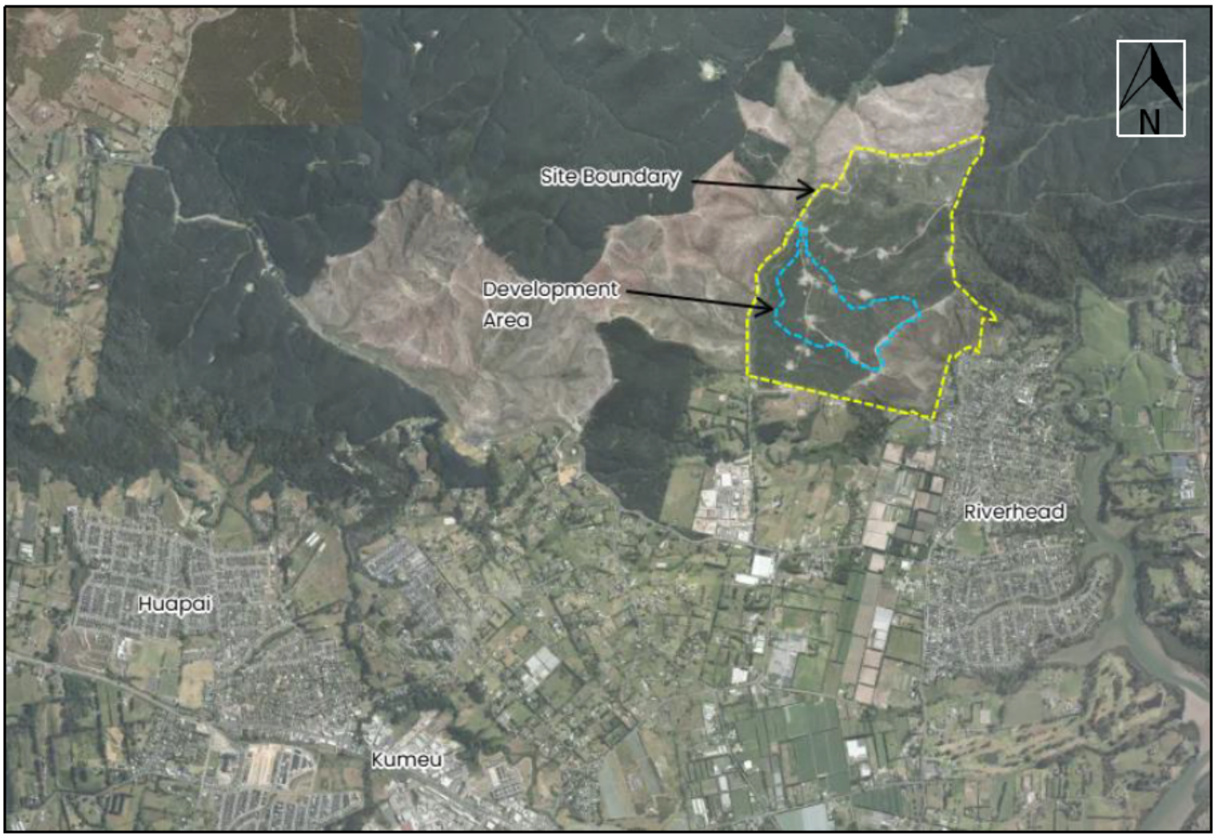
The development area is predominantly west facing gently sloping (4 to 10 degrees), ground. There are localised steeper slopes within the development area associated with overland flow paths orientated north to south and east to west. The slopes immediate adjacent to overland flow paths generally range from 10 to 26 degrees. Elevations across the development area (and to the west), range from 90 m RL at the highest point down to 25 m RL along the gully which runs parallel to Forestry Road. This then links up to Deacon Road providing access further north into the forest.

The eastern boundary of the development area is bounded by a break in slope in the opposite direction (grades down to the east). The upper elevated portion of this slope is generally steeper with a gradient ranging from 18 to 30 degrees for a vertical elevation change of 10 to 15 m. The lower portion of the slope grades range from 8 to 12 degrees over a vertical elevation change of 50 m. The elevation changes across this slope from 90 m to 10 m RL.

Auckland Council GeoMaps depict several tributaries across the site, which appear to be ephemeral at higher elevations, with permanent watercourses typically found in the lower-lying gulleys. These tributaries flow westward toward an unnamed stream, continue south and eventually drain into the Wautaiti Stream located east of the site.

A site location plan is presented in Figure 1.

**Figure 1: Site Location Plan**




Note: Image sourced from Linz NZ. Image not to scale. Yellow line depicts the site boundary, while the blue line depicts the approximate development boundary.

# 3 Previous Geotechnical Reporting

## 3.1 2019 Tonkin & Taylor Preliminary Geotechnical Assessment

Tonkin & Taylor Limited (T & T) prepared a Preliminary Geotechnical Assessment for the wider Rangitopuni Riverhead Development project (ref: 1008462.v1) dated April 2019. The assessment involved a desktop study of the general geotechnical characteristics of the site and the potential geohazards that may affect land development. While specific development plans were not provided, it was understood that the site was being considered for housing, commercial, business and industrial activities.

The preliminary report identified potential geohazards including slope instability, compressible soils, liquefaction and expansive soils, with slope instability considered likely to impose the greatest impact. The preliminary assessment concluded that the site is geotechnically feasible for land development, though varying degrees of engineering controls will be required to mitigate the risks posed by geohazards across the site.

## 3.2 2022 ENGEO Geotechnical Investigation Assessment

ENGEO prepared a Geotechnical Due Diligence Assessment (ref: 20190.000.001_01) dated 20 April 2022 for the wider Rangitopuni Riverhead site, which includes Lot 2 DP 138519, Lot 1 DP 138520 and Lot 1 and 2 DP 138521. The assessment assumed residential development of the site based on conceptual land parcels shown on Boffa Miskell Draft Land Use Plans (unreferenced) provided to ENGEO.

The assessment included a desk-based preliminary review of the site's geomorphology. Based on these findings, the site was generally considered suitable for the proposed development. The assessment also identified key geotechnical constraints pertinent to development of the site, which included slope instability, soft compressible ground, waterways, and areas of stream incision.

The site was separated into geotechnical hazard Zones A, B and C on a Preliminary Geotechnical Zoning Plan. The areas characterised by flat to gently sloping terrain were typically assigned to Zone A ("few geotechnical constraints"). The areas with moderate slope gradients, often near incised gulleys, were generally assigned to Zone B ("some geotechnical constraints, land considered feasible for land development, but some engineering control may be required"). Finally, areas with steep to very steep slopes and incised flow paths were generally assigned to Zone C ("significant geotechnical constraints, steep sloping terrain and geomorphology of the site will require extensive engineering control for land development").

# 4 Proposed Development

ENGEO have been provided with the following documentation which details the retirement village development proposed for the site. These plans are included in Appendix 1.

- Crosson Architects, Concept Design Plan for Resource Consent, Rangitoopuni Lifestyle Village, dated 29 April 2025 (unreferenced).
- Maven Associates Limited Proposed Earthworks and Overview Plan – Retirement Village, dated March 2025 (ref: 147007 rev. A, drawing no. C210-1 to C220-2).
- Maven Associates Limited Proposed Sediment Control Plan and Details – Retirement Village, dated March 2025 (ref: 147016 rev. A, drawing no. C230-1 to C230-6 and C240 to C245).
- Maven Associates Limited Proposed Roading and Overview Plan – Retirement Village, dated March 2025 (ref: 147016 rev. A, drawing no. C300 to C300-16).
- Maven Associates Limited Accessway Long Sections – Retirement Village, dated March 2025 (ref: 147016 rev. A, drawing no. C320 to C328-4).
- Maven Associates Limited, Accessway Typical Cross Sections – Retirement Village, dated March 2025 (ref 147016 rev. A, drawing C340 to 344).
- Maven Associates Limited, Proposed Stormwater Overview Plan – Retirement Village, dated March 2025 (ref 147016 rev. A, drawing C400-0 to C400-14).
- Maven Associates Limited, Proposed Wastewater Overview Plan – Retirement Village, dated March 2025 (ref 147016 rev. A, drawing C500-0 to C500-7).
- Maven Associates Limited, Proposed Water Supply Overview Plan – Retirement Village, dated March 2025 (ref 147016 rev. A, drawing C600-0 to C600-6).

The Crosson Architects proposed scheme plan indicates that the site is to be developed to include 260 independent living units (villas) and 36 care units across four stages (Stages 1 to 4). The concept plan includes a proposed main 'spine' road that extends through the centre of the development. Several smaller arterial roads extend off and reconnect to the main road. Independent living units saddle both sides of the smaller arterial roads. The centre of the development includes a care building, amenities building and wellness centre. Several planted 'green' areas are proposed across the development area between the independent living units and the heads of existing overland flow paths.

The Maven Associates plans show that it is proposed to re-align Forestry Road west of the retirement village development area. This road is to be upgraded due to flood issues associated with the current alignment. Based on our discussions, we understand that the re-alignment will comprise minor excavations on the western boundary of the road to form permanent batters or engineered design retaining wall solutions depending on the existing topography.

The Maven Associates, Proposed Water Supply Plan (Drawing C600-1) shows that water supply treatment tanks are proposed in the north-western portion of the site adjacent to the main access way into the retirement village (eastern side). This is proposed to be constructed on level ground with upslope retaining supporting excavations.

The Maven Associates, Proposed Wastewater Plan (Drawing C500-1) shows that the wastewater treatment tanks are proposed in the north-western portion of the site, adjacent to the main accessway into the retirement village (western site). The wastewater treatment tanks are proposed at the top of the proposed fill slope, but at a lower elevation to the proposed independent living units. The vertical height difference will be supported by an engineered retaining wall.

The Maven Associates Limited Earthworks plan also shows that a landscape path is proposed east of the retirement village to link up the village to the Riverhead township.

## 4.1 Proposed Earthworks

According to the Maven Associates Limited proposed earthworks plans, development of the retirement village will involve bulk grading that includes significant cut and fills to form a development area that generally grades from east to west at a final grade of up to 4 degrees.

The northern and southern bounds of the development area will comprise permanent earth cut and fill batters of less than 24 degrees.

The earthworks cut and fill plan indicate the following:

- A total of 558,130 $m^3$ of cut and 739,250 $m^3$ of fill is proposed. The earthworks area is approximately 32.3 hectares. The earthworks volumes provided are unfactored bulk earthworks only and we understand that they do not include allowance for service trenches and / or undercuts that may be required. The earthworks volumes indicate that there is a net fill surplus of 181,120 $m^3$. A nominal topsoil depth of 300 mm has been modelled across the proposed earthworks area.

- Excavations are generally proposed in the elevated eastern portion of the site. Maximum excavations are proposed up to 10 m below existing ground level. These excavations decrease through the central portion of the site to depths generally ranging from 2.0 m to 4.0 m.

- Filling is generally proposed in the lower lying western, southern and the north-eastern portion of the development area. The earthworks filling across site is proposed to range from 2.0 m to 6.0 m in the west and 6.0 m to 8.0 m within the northern, southern and north-eastern portion of the site. As a result of the filling, large batters are proposed.

- Earthworks for the proposed Forestry Road upgrade comprise minor excavations on the western boundary of the road. It is proposed that permanent batters or engineered retaining wall solutions will be required depending on the existing topography. Earthworks fill is proposed on the eastern portion of the road alignment ranging 1.0 m to 2.0 m in height.

- Surplus cut material is to be filled north of the proposed retirement village. The Maven Associates Limited Cut and Fill plans indicate that the surplus fill area is to take some of the net fill of 181,120 $m^3$. This material is proposed to be placed at a thickness of 1.0 m to 2.0 m, with some isolated areas increasing to 3.0 m to 5.0 m in height.

- Minor earthworks are proposed on the large slope east of the retirement village area to form an on-grade landscape footpath to provide foot access from the retirement village to the Riverhead Forest township. The earthworks plan indicates < 1.0 m in cut and fill and follows the existing topography of the slope.

- Filling in the order of 2.0 m is proposed to form the new accessway into the retirement village. However, we envision that this will likely be closer to 6.0 m due to the presence of existing fill material that will require undercut and replacing. We note that the accessway extends over an existing overland flow path and stream.

- Minor excavations are proposed below the retirement village at the western boundary of the site with the formation of a 4.0 m high earth bun. The earthworks are proposed to form an attenuation pond to reduce the effects of downslope flooding for the 100-year events.

## 4.2 Proposed Stormwater and Wastewater

As there is no existing stormwater network in the vicinity of the development area, a new stormwater network will be constructed to support the development. The retirement village will be provided with a dual stormwater network. Roof caught water will be retained in a separate network from JOALs and surface runoff.

Water collected from the roof will be channeled through a sealed piping system into a collection tank for re-use on-site. The water will be contained in a U-PVC or PE network and will be conveyed to storage tanks, before being treated and reticulated within the village. This system is to be designed by GWE Consulting Engineers.

A separate stormwater network will be provided for surface runoff. The networks will discharge surface runoff via specifically design outfalls to existing streams and / or OLFP's. These outfalls will be designed to include rock riprap and erosion controls at the point of discharge.

There is no reticulated wastewater network within the site and the site is not contained within the urban extent of Auckland. The wastewater design for all lots has been completed by GWE Consulting Engineers. The wastewater is to be supported by a private wastewater treatment plant that will provide treatment for all wastewater before disposal into the ground, through a series of dripper irrigation lines. The design and location of the disposal area have been undertaken by GWE Consulting Engineers.

A gravity wastewater network will be constructed within the development, and this will provide lot connection to each of the villas and communal buildings within the village. Three private pumpstations with rising mains have been proposed where lots are located on peninsulas that would not be able to be serviced by gravity feed. These pumpstations will outlet into a discharge manhole connected to the overall gravity network. This gravity network conveys all flows to the plan system before being treated and disposed to the ground.

# 5 Desktop Study

## 5.1 Regional Geology

GNS Science maps the site and nearby area as being underlain by a number of geological units as presented in Figure 2.

Based on the GNS map (1:250,000), the site is underlain by East Coast Bays Formation (ECBF) of the Warkworth Subgroup (Waitematā Group). ECBF typically comprises alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. The upper horizons of the ECBF typically comprise residually weathered plastic silts and clays which gradually increase in strength with depth. Commonly a 'Transition Zone' comprising of dense sand and hard silts / clays separates the residually weathered soils and competent bedrock. The weathering profile and layering thickness of the residually weathered soils can vary significantly depending on the geomorphological setting of the site.

Albany Conglomerate, also forming part of the Waitematā Group, is mapped approximately 950 m northeast of the site. Albany Conglomerate typically comprises a well-cemented mixture of hard pebbles, cobbles and boulders of igneous and metamorphic origin in highly lenticular beds. This geological unit commonly has a similar weathering profile to that of the ECBF (described above), however, based on its geological deposition it is typically less extensive and confined to 'narrow' lenses.

Several alluvial derived geological units are mapped near the site ranging from Holocene to late Pliocene age are mapped near the site as shown in Figure 2. Based on site geomorphology, Holocene alluvial deposits are expected to be present within low lying areas of the site adjacent to overland flow paths, streams and / or rivers. Older middle Pleistocene to late Pliocene deposits may also be present within the site, however, these are likely to be at higher elevations relative to the younger alluvium.

These alluvial formations typically comprise pumiceous mud, sand and gravel with muddy peat and lignite. However, the Holocene alluvium is typically more compressible and consist of soft silts and clays and may include layers of organic materials.

This geological unit a has been described in later sections of this report as Undifferentiated Takaanini Formation consistent with the revised GNS stratigraphic framework dated March 2021

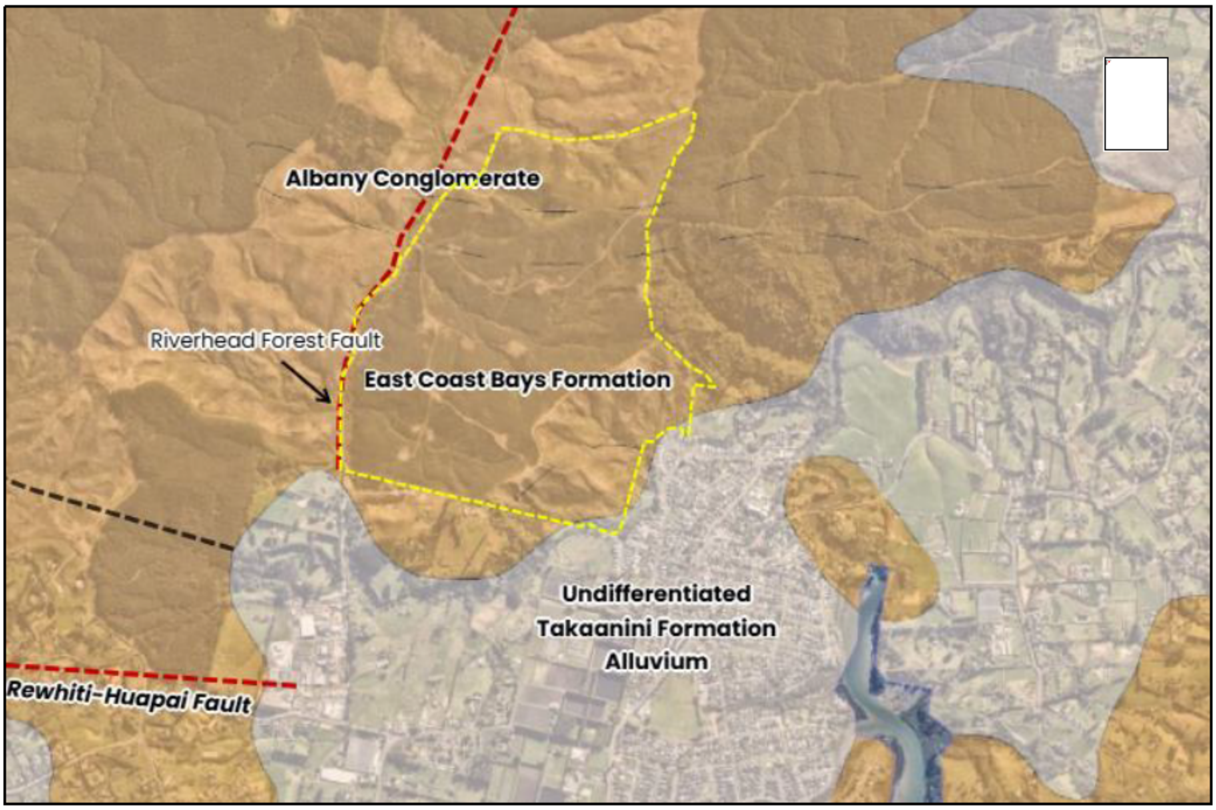
## 5.2 Seismicity

The NZ Active Fault Database indicates that there are no known faults intersecting the site. The nearest active fault mapped by the NZ Active Fault Database is the Waikapua Fault located approximately 45 km southeast of the site. Two inferred inactive faults are mapped within the vicinity of the site (refer to Figure 2) and are summarised as follows:

- The *Rewhiti-Huapai Fault* is mapped as an east-west striking normal fault with a down-throw to the south, estimated to be in the order of tens of meters along its length. The fault is mapped approximately 1.3 km to the south of the site.

- The *Riverhead Forest Fault* is mapped as a northeast-southwest striking normal fault extending along Forestry Road, Riverhead. The Riverhead Forest Fault was recently discovered, and as such, limited information is available with respect to its sense of movement and offset. The fault is mapped approximately 100 to 200 m west of the site.

**Figure 2: Regional Published Geology 1:250,000**




Note: Image sourced from GNS WebMaps overlay [accessed 08/03/2025].

## 5.3 Historical Aerial Photography Review and Commentary

Historical aerial photographs of the site dating from 1940 to 2024 have been reviewed to assess historical site land use and landform modifications, where photographic quality and vegetative cover permit. These photographs were retrieved from the Alexander Turnbull Library, Auckland Council GeoMaps, the Retrolens online portal (retrolens.nz) licensed by Land Information New Zealand (LINZ) and Nearmap New Zealand.

Since the early 1900s, the site and broader Riverhead Forest area have undergone periodic pine tree harvesting, occasional re-routing of forestry roads, and have been developed with biking and walking trails to enhance the recreational use of the area. Evidence of localised instability is observed within some aerial images shortly following forestry clearing; however, this was difficult to see as the site has been densely vegetated for a significant portion of time. This is further discussed in the Geomorphological Study in Section 6.1. The historical aerial imagery indicates that between 2022 and 2023, the Pinus radiata plantations covering the site were harvested. The site is now vacant and largely covered by shrubs and woody debris ('forestry slash').

The site and wider Riverhead Forest area were historically covered by native forest largely consisting of kauri trees. During the Colonial Era of New Zealand dating from the mid-1800s to early 1900s, the area was logged, and kauri gum was extracted from the soil. In the 1920s, Pinus radiata plantations were cultivated for commercial forestry, which was facilitated by the construction of gravel road networks across the site prior to 1940.

### 5.4 New Zealand Geotechnical Database

ENGEO have reviewed the New Zealand Geotechnical Database (NZGD) for historical investigations completed by others near to, or across the site.

Based on our review, there are no historical investigations within the site or development area. Several historical investigations are mapped to the south of the site within Riverhead township and surrounding land. These comprise hand auger boreholes and rotary cored machine boreholes.

We have completed a high-level review of a representative sample of investigations within the wider area and note that the area is generally underlain by alluvial soils from ground level or existing fill. The alluvial soil generally increases in depth towards areas of low relief and proximity to the Rangitopuni Stream and estuarine areas to the south and east of Riverhead township. Underlying the alluvial soils are the residually weathered East Coast Bays Formation Soils. These soils generally increase in strength with depth to the extremely weak to very weak siltstone / sandstone bedrock.

Although the historical investigation and geological findings presented on the New Zealand Geotechnical Database (NZGD) align with the mapped geology in the local area, we haven't included any of these on our Investigation Location Plan (Appendix 2) as they are typically located well away from our site and development area.

## 6 Site Walkover Assessment

### 6.1 Geomorphological Study

ENGEO visited site in December 2024 and February 2025 to undertake geomorphological mapping of the Stage 2 Retirement Village. Due to areas of the site being covered by low shrubs and forestry 'slash' at the time of our walkover, this geomorphological assessment has been supplemented by reviewing available historical aerial photographs and digital elevation models. The interpreted geomorphology is presented on Geomorphology Plans (Appendix 2), and the key observations summarised as follows:

#### Retirement Village

- The retirement village site is defined by gently sloping ground orientated east to west with an elevation change from 90 m RL to 25 m RL from the eastern to western site boundary.

- An extensive network of overland flow paths is present within the site. These features are ephemeral and typically orientated south-east to north-west, and east to west. The overland flow paths drain into east to west trending permanent stream features to the north and south of the site, which in turn drain into a stream tributary that runs parallel to Forestry Road and Deacon Road. Isolated steeper slopes are present adjacent to the ephemeral overland flow paths within the site boundary and typically range 10 to 26 degrees. No obvious evidence of significant scour, erosion or instability was observed within these features.

- Evidence of localised 'shallow instability' can be observed within the permanent stream banks to the north, south and east of the site. The historical failures typically consist of shallow circular slides / slumping associated with toe softening or erosion. These features are largely situated outside of the development boundary except for isolated areas adjacent to the north-western and south-western corners of the site.

- Several access tracks have been constructed within the site. These appear to have been largely constructed 'at grade', however, where necessary, localised fills have been completed to infill gully and overland flow path features. Based on our site walkover and investigation, fills in the order of 9 m have been completed to form some of the stream crossing / accessways. There is a high possibility that the historical filling overlies potentially 'soft' or organic alluvial soils that maybe susceptible to consolidation settlement.

- Laterally extensive southeast aspect head scarps are located adjacent to the eastern site boundary at the head of an extensive network of overland flow paths and gully features. The upper extents of the head scarps are characterised by steep to very steeply sloping grounds and gradually decrease in gradient with a decrease in elevation within the gully network. These features have likely formed over extended time frames as a result of successive shallow soil failures or 'slumping' caused by soil saturation and concentrated surface runoff. Over time these failures result in a loss of toe support and gradually extend further upslope creating the arcuate head scarp features observed today. This failure mechanism is supported by our observations identifying hummocky ground and debris lobes within the landslide 'body'.

# 7 Site Investigation

ENGEO have completed an intrusive investigation across the proposed retirement village from January to March 2025. The investigation comprised the following:

- Twenty hand auger boreholes, denoted HA01 to HA20, drilled down to depths ranging from 1.5 m to 5.0 m below ground level. The hand auger boreholes were typically drilled within areas of low relief (overland flow paths), access road or previously cleared areas used for forestry operations.

- Seven machine boreholes, denoted MBH01 to MBH07, drilled down to depths ranging from 16.5 m and 42.0 m below ground level. The machine boreholes were completed across the site on existing gravel roads that provided access.

    - Standpipe piezometers were installed within MBH02, MBH03, MBH05 and MBH06 to facilitate monitoring of static groundwater levels.

- Five Cone Penetration Tests, denoted CPT01 to CPT05 were advanced across the site area by McMillan drilling group Limited on 7 and 8 February 2025. The CPTs were advanced within the vicinity of the gravel roads across the site. The CPTs were advanced to depths ranging from 11.65 m to 21.78 m depth.

The investigations are shown on our Investigation Location Plan in Appendix 3 and full borehole logs and machine borehole core photos are in Appendix 4. These logs have been prepared in general accordance with the New Zealand Geotechnical Society Guideline for the field classification and description of soil and rock for engineering purposes (NZGS, 2005).

The raw CPT results prepared by McMillan Drilling Group Limited are included in Appendix 5.

## 7.1 Ground Conditions

### Topsoil

Topsoil was encountered within hand auger boreholes HA02, HA07, HA09, HA10, HA13 and HA15 to HA17. The topsoil was encountered to a maximum depth of 0.2 m below ground level.

Topsoil was not encountered in other investigation boreholes; however, these were largely completed on or very close to access roads and we expect that existing topsoil was stripped when they were formed.

### Existing fill

Existing fill was encountered within all machine borehole investigations except for MBH02. The existing fill comprised of clayey silt with minor organics and fine to coarse gravel with trace cobbles and rootlets. The existing fill was encountered from ground level to a maximum depth of 1.0 m below ground level. The machine boreholes that were undertaken within proximity to existing forestry roads encountered fill to a maximum depth of 0.45 m below ground level and was gravel in nature, whereas the existing fill encountered within boreholes outside of the road corridors encountered cohesive soils (clayey silts) with variable organic content. Strength testing was not undertaken across the existing fill within the machine boreholes.

Existing fill was also encountered within hand auger boreholes HA01, HA03, HA04, HA05, HA06, HA08, HA11, HA13, HA14, HA18, HA19 and HA20. The existing fill within the hand auger boreholes generally comprised clayey silt and silt with variable gravel inclusions and organic content. This was consistent with the machine borehole findings. The existing fill was generally less than 1.0 m thick except for HA13 and HA19 where it was encountered up to 2.9 m bgl.

### Alluvium

Takaanini Formation Alluvium was encountered within MBH04 from ground level to a maximum depth of 7.15 m below ground level. The alluvial soils comprised hard sandy silty and clayey silt from ground level to 3.0 m below ground level.  Strength testing resulted in a single SPT N-value of 7 at 1.5 m below ground level, whilst the shear vane was unable to penetrate (UTP).

From 3.0 m to 7.5 m below ground level the soils encountered comprise firm sandy silt and silty clay with some amorphous organics. SPT N-values across this soil layer ranged from 0 to 3, whilst undrained shear strength testing ranged from 32 to 37 kPa.

Hand auger boreholes HA01, HA02, HA04, HA08, HA12, HA15, HA16 and HA20 encountered alluvial soils. The alluvial soils within the hand auger boreholes generally comprised very stiff to hard clayey silt and silty clay with variable sand content. Undrained shear strength testing was generally greater than 100 kPa except for isolated layers of organic silt which resulted in shear strengths ranging 60 to 80 kPa.

Borehole HA12 is considered an outlier but represents alluvial soils immediately within the overland flow paths. This hand auger borehole encountered firm to stiff organic silt and clayey silt with minor organics. A groundwater elevation of approximately 0.8 m below ground level was measured immediately following drilling. Undrained shear strength testing ranged 22 to 59 kPa across this soil layer. However, a sharp increase in strength was noted and the borehole met practical refusal at 2.3 m bgl, indicating hard material is present under this soil layer. This inference was further supported in HA08 close by, where alluvial soils were underlain by hard sandy silt of the East Coast Bays formation underlying alluvial soils.

### East Coast Bays Formation

The East Coast Bays Formation of the Waitemata Group is divided into subordinate layers of soils and rock.

#### Soils

East Coast Bays Formation soils were encountered within all investigation's boreholes underlying existing fill, Alluvial soils and Albany Conglomerate. The residually weathered soils across the site can be described as containing two layers

The upper soil layer consists of stiff to very stiff silty clay and clayey silt with some sand. These soils were typically 10 to 12 m thick across the western and central portion of the site but increase to approximately 20 m thick within the eastern elevated portion of the site. Average SPT N-values across the boreholes within these soils ranged 7 to 11, while undrained shear strength testing was generally greater than 70 kPa within the machine boreholes.

The lower soil layer consists of moderately dense to very dense silty sand with interbedded layers of hard sandy silt and clayey silt. This layer was generally less than 5.0 m thick in the low-lying gullies, whilst this layer was 15 to 20 m thick within the central and eastern elevated portions of the site. Average SPT N-values across the boreholes within these soils ranged 18 to 25. It should be noted that SPT testing was undertaken to maximum depths of 30 m below ground level due to drilling rig ability.

The lower unit has been described as transitional zone soil in our geological ground models and stability modelling.

All hand auger boreholes encountered East Coast Bays Formation soils, except for HA01, HA02, HA04, HA12, HA15. The soils encountered within the hand auger boreholes aligned with the machine borehole findings to termination depth and typically comprised stiff to hard clayey silt and silty clay with variable sand content. Undrained shear strength testing across the boreholes within these soils were typically greater than 120 kPa.

### Albany Conglomerate

Albany conglomerate of the Waitemata Group was encountered underlying fill within MBH02. This soil was described as comprising silty clay and clayey silt with minor sand content. Isolated layers of silty fine to coarse sand and sub grounded fine to coarse gravel were observed within the soil matrix. SPT N-values ranged 3 to 7 across this soil layer.

Albany conglomerate was not encountered within other investigation boreholes, however it's likely that the deposit extends laterally beyond MBH02 between investigation location points.

#### Rock

Underlying the residually weathered soils of the East Coast Bays Formation comprised extremely weak to very weak interbedded sequences of siltstone and sandstone.

Within the west and lower lying portions of the site (MBH02 and MBH04), moderately weathered very weak rock was encountered at depths ranging 11.5 m to 23.0 m below ground level (28.1 m RL to 32.7 m RL).

Within the central portion of the site MBH03 encountered highly weathered extremely weak sandstone from 36.5 m below ground level to the base of the borehole 37.5 m below ground level (26.35 m RL to 25.2 m RL). Very weak rock was not encountered within this borehole.

MBH05 drilled north of MBH03 within a slightly lower elevation in a gully feature encountered extremely weak sandstone from 12.8 m to 16.5 m below ground level (44.4 m RL to 40.7 m RL). Very weak interbedded layers of siltstone and sandstone was encountered form 16.5 m below ground level to the base of the borehole (40.7 m RL to 37.5 m RL).

Within the elevated eastern portion of the site MBH01 and MBH06 encountered extremely weak to very weak rock at depths ranging 33 to 38 m below ground level (top of layer varying 37.8 m RL at MBH06 and 52.75 m RL at MBH01).

MBH07 was undertaken east of the proposed development area across the middle of the large slope. The machine borehole was advanced from 49.6 m RL to assess depth to bedrock at the middle of the slope. Extremely weak sandstone was encountered form 34.3 m below ground level to the base of the borehole at 40.5 m bgl (20.7 m RL to 14.5 m RL). Very weak rock was not encountered within this borehole.

SPT N-values that were completed within the extremely weak to very weak rock was greater than 50.

We note that the rock head across the site has a variable weathering profile which is likely due variable topography across the site and presence of streams / overland flow paths, where the shallow soils have been subject to erosion and scouring due to surface water.

## 7.2 Instrumentation and Groundwater Monitoring

Standpipe piezometers were installed within boreholes MBH02, MBH03, MBH05 and MBH06. The piezometer screen depths ranged from 6.5 m to 14.0 m below ground level in MBH02, 2.0 m to 8.0 m below ground level in MBH03, 2.0 m to 9.5 m below ground level in MBH05, and 2.0 m to 12.0 m below ground level within MBH06. The piezometer construction details are presented on the machine borehole logs in Appendix 4.

Table 1 presents the manual groundwater dips completed for all machine boreholes shortly after drilling and during the monitoring period. Groundwater monitoring plots for MBH02, MBH03 and MBH06 are included in Appendix 6. The groundwater monitoring was continually assessed with electronic data loggers installed within the piezometers. We were unable to retrieve the data from MBH05, thus, this is not presented.

**Table 1: Groundwater Monitoring**

| Borehole ID | Groundwater depth (m bgl) [m RL] | | |
|---|---|---|---|
| | Variable date[1] | 12/03/2025 | 21/03/2025 |
| MBH01 | 1.5 [84.5] | N/A[2] | N/A |
| MBH02 | 8.8 [42.2] | 8.7 [42.3] | 8.7 [42.3] |
| MBH03 | 4.4 [58.4] | 4.7 [58.1] | 4.8 [58.1] |
| MBH04 | 3.3 [41.0] | N/A[2] | N/A[2] |
| MBH05 | 3.4 [53.8] | 6.3 [50.9] | 6.5 [50.7] |
| MBH06 | 6.3 [70.3] | 4.1 [72.3] | 4.2 [72.2] |
| MBH07 | 6.1 [48.9] | N/A[2] | N/A[2] |

1 Groundwater levels were measured shortly after completion of drilling, however the drilling of the machine boreholes was completed over February and March 2025.

2 N/A = Not assessed. These boreholes did not have piezometers installed.

# 8 Engineering Geological Model

## 8.1 Model Development

An observational engineering model for the site has been developed incorporating the findings of our desktop study, geomorphology mapping and our intrusive testing. This information has been combined with the topographic surface contour data to create a 3D geology ground model using the Leapfrog Works software. The creation of the model allows for visualisation of the borehole data in both 3D and cutting 2D cross sections to support the analysis and design. Three geological cross sections have been cut to present a general ground profile for the critical slopes across the proposed Retirement Village area.

These sections along with their locations are presented in Appendix 7.

## 8.2 Ground Model

### Elevated Portion of the Site

Our field observations and review of historical and recent subsurface investigation data indicate that the elevated portion of the site comprise residually weathered stiff to hard silts and clays, and medium dense to dense sands of the ECBF. The soils overlay completely weathered, extremely weak and very weak interbedded sequences of sandstone and siltstone was encountered.

No obvious evidence of 'disturbed soil' was identified in the boreholes drilled on the intact ridge.

### Slope Regression Features

A slope regression feature is present adjacent to the eastern boundary of the development area that is largely controlled by a large slope. Based on our investigation and site observations, the bottom portion features likely contain surficial colluvial soils typically comprising variable strength (firm to very stiff) and \ cohesive soils. The composition of these soils is expected to vary across the slope, dependent on the parent unit of the historic slope failure. As encountered within MBH07 the shallow soils consist of competent East Coast Bays Formation soils that increase in strength with depth to the underlying bedrock.

Based on the site geomorphology, the slope regression features are likely from shallow 'slumping' of soil because of extreme saturation or concentrated surface water runoff. These failures result in an over steepened crest of slope that causes head scarp regression.

### Gully and Overland Flow Paths

Takaanini Formation alluvial soils were generally encountered in the gully and overland flow path features present throughout the site. These soils were identified to be of variable strength ranging from firm to hard and were typically weakest closest to the surface water features, especially as the moisture content within the soil increases and the presence of shallow groundwater.

Soft potentially compressible soils and organic lenses were encountered immediately adjacent to or within several of the overland flow path features.

## 8.3 Assumptions and Uncertainties

- All ground models are an interpretation and simplification of site conditions to facilitate understanding and decision-making. Ground models require interpolation between discrete data points and an understanding of site history to be as accurate as possible.
- Further refinement is recommended through subsequent investigations, design and construction phases where specific items affect the viability of the proposed development.

# 9 Geohazard Assessment

The following sections outline the key geotechnical hazards that have been assessed for the site and proposed development.

## 9.1 Seismic Hazards

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

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

### 9.1.1 Preliminary Site Seismic Subsoil Classification

For the purposes of preliminary structural design, a site soil classification of 'Class C – Shallow Soil Site' as per NZS 1170.5:2004 is appropriate for the site based on soil strength materials to the base of the investigations and our understanding of the geological setting.

### 9.1.2 Ground Rupture

As cited in Section 5.2, the site is located approximately 45 km from the nearest mapped active fault. At this distance, fault related ground rupture and ground lurching are considered unlikely.

However, given the tectonic setting, regional subsidence may be experienced following a large earthquake on faults in the area. In seismically active areas there is always a risk that unmapped active faults may be present, particularly when the faults may have a low recurrence interval and may be buried under younger alluvial sediments. The risk of surface damage from unmapped faults is no different at this site the surrounding land.

### 9.1.3 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 drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

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

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

**Table 2: Design Liquefaction Parameters**

| Limit State | Return Period | $a_{max}$ | Magnitude |
|---|---|---|---|
| SLS | 25 years | 0.05 g | 5.9 |
| ULS | 500 years | 0.19 g | 6.5 |

### 9.1.4 Liquefaction assessment

We have carried out a computational liquefaction analysis using the CPT test data (CPT01 to CPT05) to assess the liquefaction risk to the development following bulk earthworks. We assessed Serviceability Limit State (SLS) and Ultimate Limit State (ULS) scenarios based on magnitudes and accelerations outlined in Section 9.1.3.

The liquefaction potential assessments have been carried out with computer software (GeoLogismiki Cliq v.2.3.1.15) using Boulanger & Idriss (2014) methodology for liquefaction triggering.

Based on our review of the earthwork's plans, earthworks cut of 1.0 m, 3.0 m and 10.0 m have been adopted to CPT01, CPT03 and CPT04 respectively, whilst an earthworks fill of 5.0 m has been adopted for CPT02. CPT05 has been advanced outside the earthworks are adjacent to an existing gully / overland flow path feature. We have assessed liquefaction risk on existing conditions for CPT05.

Groundwater depths presented in Section 7.2. However, for the purposes of our modelling a groundwater level 8.0 m bgl has been adopted to CPT01, 6.0 m bgl for CPT02, 5.0 m bgl for CPT03 and CPT04 and 3.5 m bgl for CPT05 for existing conditions. Groundwater levels have been corrected for CPTs within areas of proposed filling (earthquake GWT).

Serviceability limit state (SLS) and Ultimate Limit state (ULS) results are summarised in Table 3. The detailed results for both cases are included in Appendix 8.

**Table 3: Post Earthwork Liquefaction Assessment**

| Parameter | Performance Summary | |
|---|---|---|
| | SLS | ULS |
| Total Settlement | 0 mm | 0 to 29 mm |
| Index Settlement | 0 mm | 0 to 29 mm |
| LSN | 0 | 0 to 3.6 |
| LPI | 0 | 0 to 0.3 |
| Crust Assessment | Greater than 3.0 m | 3.0 m |

1. Index settlements refer to calculated liquefaction induced settlements in the upper 10 m
2. LPI and LSN performance have been determined in accordance with MBIE Module 3
3. Crust assessment based on Bowen and Jacka without consideration of specific foundation width and embedment.

### 9.1.5 Liquefaction Analysis Discussion

Our analysis indicates that liquefaction is not triggered at a SLS seismic event.

Under ULS seismic conditions, liquefaction induced ground settlements were estimated to be between 0 and 29 mm.

Table 3 presents the Liquefaction Potential Index (LPI) and the Liquefaction Severity Number (LSN) for the SLS and ULS cases. For the ULS case, the CPT locations have a LPI below 0.3, indicating a low risk of triggering. For the LSN values, the CPT locations have values below 3.6, indicating insignificant (LO) liquefaction induced ground damage on foundation performance. Index settlements were estimated up to 29 mm for CPT05, whilst the remaining CPTs were less than 9 mm.

In the ULS case, predicted settlements were generally estimated to occur at a depth greater than 3.0 m below ground level (considering earthworks).

Based on our liquefaction assessment, we consider the risk of liquefaction induced settlement to the site to be low. The Structural Engineer will need to consider if total settlements up to 29 mm are acceptable for buildings within development area of the site in the ULS case.

We recommend a suite of additional cone penetration tests is undertaken once the site is cleared to refine this assessment.

### 9.1.6 Lateral Spread Analysis Discussion

Based on the site's geological setting, the cohesive nature of the soils encountered on-site, lack of laterally continuous layers of potentially liquefiable soils and the height of theoretical free face across the slopes, we consider lateral spreading to be a low risk for the site development.

However, we expect that during an ultimate limit state event, localised co-seismic slope instability may occur immediately adjacent to locally incised gully features within the low-lying portion of the site. We expect that this will be in the form of shallow stream bank collapse.

## 9.2 Settlement

We have carried out a preliminary assessment of potential fill induced elastic settlement at each of the four CPT locations. Preliminary settlement analysis has been undertaken using computer software CPeT-IT (v2.2.1.1). The analysis has been undertaken using soil parameters derived from the CPT tests.

As previously discussed in Section 4.1, filling is generally proposed in the lower lying western, southern and the north-eastern portion of the development area. Earthworks filling across site is proposed to range from 2.0 m to 6.0 m in the western and southern portion of the site and 6.0 to 8.0 m within the northern and north-eastern portion of the site.

For the purposes of our preliminary settlement analysis, we have made the following assumptions:

- A unit weight of 18 kN/m$^3$ for cohesive engineered fill.
- Proposed fill area 40 m x 40 m (approximate) across part of the proposed building platforms.

### 9.2.1 Settlement Discussion

For earthworks fill up to 6.0 m in height in the western portion of the site, elastic settlements in the order of 45 mm to 95 mm are estimated.

For earthworks fill from 6.0 m to 8.0 m in height in the western portion of the site, elastic settlements in the order of 95 mm to greater than 120 mm are estimated.

The majority of the proposed fills are over soils that we expect to comprise a thin veneer of alluvium underlain by residually weathered East Coast Bays Formation soils.

The Over Consolidation Ratio (OCR) of the residually weathered East Coast Bays Formation soils are typically between 4 and 10, suggesting that the soils are over consolidated. The Cone Penetration tests have largely been completed within these residually weathered soils, except for CPT01 and CPT05, that were undertaken adjacent to overland flow paths and gully features. Estimated OCR values were still greater than 4 for these soils.

The rate of settlement within these soils may occur at a slow rate given their plastic and cohesive nature. As such, we recommend that a series of settlement monitoring plates are placed after site stripping and prior to filling in areas of fill greater than 4.0 m in height. Further discussion on mitigation measures to manage fill induced settlement is included in Section 11.6.

## 9.3 Expansive Soils

Expansive clay and silt soils are common in Auckland and tend to shrink and swell particularly with seasonal fluctuations of water content. Shrink-swell behaviour has implications for foundation design and surface structures and will need to be addressed during foundation design of buildings. Investigations within the site encountered cohesive soils within the shallow soil profile, which generally comprised clayey silts with some silty clays also encountered. We have provided a preliminary expansive site class in Section 14.1 of this report.

## 9.4 Slope Stability Assessment

### 9.4.1 General

We have undertaken a slope stability assessment to evaluate the risk of instability to the development due to the existing topography and permanent batters as result of the proposed bulk earthworks. The appended stability analysis reflects the analysis of representative critical sections through the development area for the retirement village.

Cross section alignments were selected using critical sections from the leapfrog geological model, proposed earthworks plan and geomorphological features observed in our assessment.

To address post earthworks and development stability, we have assessed eleven cross sections (denoted Section 1 to 12).

The section locations are included below in Figure 3 and the slope stability modelling outputs are included in Appendix 9.

**Figure 3: Retirement Village Slope Stability Modelling Cross Sections**

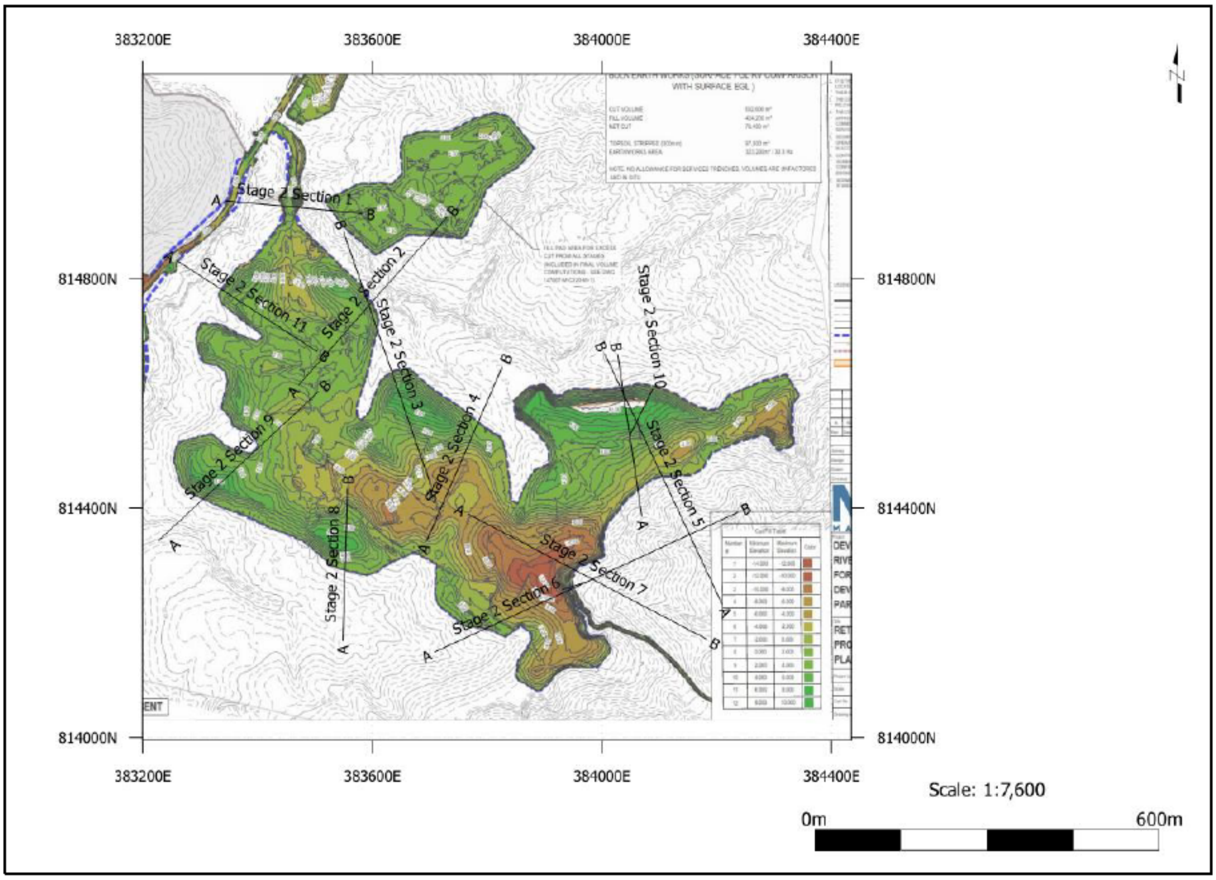



Image produced from Leapfrog. Image not to scale (refer to Appendix 9 for full sheet)

### 9.4.2 Assumptions and Design Parameters

The following assumptions and design parameters have been adopted to support the stability analysis:

- Stability analysis was undertaken assuming noncircular failure mechanisms using optimised Cuckoo search with software Slide 2.

- Normal (static) groundwater conditions have been modelled using the existing groundwater data. For the transient case (worst credible), a conservative groundwater estimation has been adopted on a case-by-case basis using the existing groundwater data, cross section topography, lithology and resultant changes due to the proposed earthworks.
- Proposed building platforms and roads have been modelled with a uniformly distributed load of 20 kPa.
- Importance Level 2 has been assumed for the undeveloped lots in accordance with NZS1170.5:2004. However, this may need to be revised depending on the future development proposals. On this basis, peak horizontal ground accelerations (amax) have been calculated in accordance with MBIE / NZGS Module 1 (2021) Pga = 0.19 for the 1/500-year return period case (ULS).
- Lateral co-seismic displacements were assessed for sections where yield accelerations causing a FOS < 1.0 were estimated for the ULS seismic case (i.e. 0.19 g). This was calculated in accordance with Jibson 2007. This method is empirical and simplified to estimate co-seismic displacements based on Newmark's sliding block model.

The Factor of Safety (FoS) for limit equilibrium analysis (as is used in the slope stability analysis) is a ratio of the forces resisting failure to the forces driving failure.

Factor of Safety = Resisting Forces / Driving Forces

A FoS of less than 1.0 is typically considered to represent likely slope instability, whilst a FoS of greater than 1.5 is usually targeted in design to mitigate against slope instability issues. Table 4 presents the FoS that are required for residential development in Auckland. These are in keeping with the requirements of Section 2 of the Auckland Council Code of Practice for Land Development and Subdivision July 2022 (Version 2.0).

**Table 4: Minimum Global Factors of Safety for Stability**

| Residential Subdivision / Development | |
|---|---|
| **Conditions** | **Factor of Safety Required** |
| Normal groundwater conditions (Static) | 1.5 |
| Extreme (worst credible) groundwater condition (Transient) | 1.3 |
| Seismic condition with a 500-year event (ULS) | 1.0 |

### 9.4.3 Design Ground Model

The cross sections to support the analysis have been taken from our 3D ground model development engineering on Leapfrog. The soil parameters presented in Table 5 have been used for stability analysis. These parameters have been adopted following a review of the existing reports, completion of ground investigations and local experience in the area.

**Table 5: Slope Stability Analysis Soil Parameters**

| Material | Unit Weight (kN/m3) | Effective Cohesion, c' (kPa) | Effective Angle of Friction, φ' (°) | Undrained Shear Strength (kPa) |
|---|---|---|---|---|
| Engineered (cohesive fill) | 18 | 7 | 30 | 110 |
| Colluvium | 16 | 2 | 28 | 90 |
| Undifferentiated Takaanini Formation Alluvium | 17 | 3 | 28 | 80 to 100 |
| Albany Conglomerate | 18 | 5 | 28 | 75 |
| East Coast Bays Formation Residual Soils | 18 | 6 | 30 | 100 to 120 |
| East Coast Bays Formation Transition Zone | 18 | 7 | 34 | 200 |
| East Coast Bays Formation Rock | 19 | 50 | 36 | N/A |

### 9.4.4 Slope Stability Results and Discussion

Table 6 presents a summary of the stability analysis results with discussion below. Slope stability modelling outputs are included in Appendix 9.

**Table 6: Slope Stability Analysis Results**

| Cross Section Location | Estimated Global FOS | | |
|---|---|---|---|
| | Static | Transient | Seismic (0.19 Pga) |
| 1[2] (embankment) | 1.59 | 1.41 | 2.45 |
| 2 | 1.62 | 1.35 | 1.35 |
| 3 | 1.61 | 1.45 | 1.93 |
| 4 | 1.66 | 1.47 | 1.41 |
| 5 (eastern slope) | 1.81 | 1.55 | 0.80[1] |
| 5 (northern fill slope)[2] | 1.52 | 1.30 | 0.94[1] |
| 6 | 2.21 | 2.21 | 0.87[1] |
| 7 | 2.19 | 1.90 | 0.93[1] |
| 8 | 2.29 | 1.76 | 1.29 |
| 9 | 1.85 | 1.53 | 2.02 |
| 10[2] | 1.50 | 1.30 | 1.45 |
| 11 | 1.66 | 1.43 | 1.25 |
| 12 (attenuation pond) | 1.92 | 1.75 (1.07[2]) | 1.56 |
| RT-Section | 1.61 | 1.36 | 1.26 |

1 For the Seismic Ultimate Limit State cases where the minimum FOS were not met (i.e. greater than 1.0) a co-seismic displacement assessment was undertaken to estimate lateral displacement from the yield acceleration to the ultimate co-seismic peak ground acceleration of 0.19 g.

2 These slopes require engineered MSE walls. This is further described in Section 12.0.

3 Extreme groundwater (post flood case) and a saturated fill berm.

Most of the slope stability modelling indicated that the minimum global of factor for slope stability is met for the proposed development provided that engineered solutions are adopted. Based on the results of our modelling, we make the following comments:

- Section 1 is located within the northern portion of the site and is cut through the proposed fill embankment to provide access to the slope. To achieve minimum global factors of safety we have modelled a MSE geo-grid reinforced slope. Conceptually, this will comprise of 6.0 m long RE560 geo-grid at 0.6 m vertical spacing. Detailed designs will need to be undertaken to co-ordinate the location of culverts and how the fill ties in with the existing road alignment and topography.

- Sections 2 to 4 indicated that minimum global factors of safety for stability were met. As such, we do not consider that the engineered fills will require geo-grid reinforcement to provide stability and support to distributed building loads of 20 kPa. The existing and permanent fill slopes are approximately 22 to 24 degrees. If planting on the face is proposed, then geogrid lengths within the slope face and strata web geocells will be required to support landscaping and face stability. Conceptually these geo-grid lengths are expected to be 3 to 4 m long with strata web (or equivalent) to provide stability of the slope face for landscaping. The first geo-grid layer will likely be lower in the slope face to allow the construction of a trench and wire ropes at the top of the slope to build the strata web.

- Sections 5 (north) and 10 were assessed across the north-eastern boundary of the site. To achieve acceptable global factors of safety an MSE earth fill (or equivalent) is required.

- Sections 5 (east), 6 and 7 were assessed within the eastern portion of the site which contains a large slope exhibiting existing geomorphology features - including over steepened gully heads and mid-level benches indicating signs of localised instability and gully regression. The stability analysis for these sections indicated that global stability for the static and transient cases was met. However, in the seismic case yield accelerations below 0.19 g were estimated to cause instability. A co-seismic displacement assessment was undertaken and estimated displacements ranged from 1 to 14 mm for the 16% probability of exceedance case. We consider that these will be within acceptable ranges for an ultimate limit seismic event. We have included in-ground palisade wall advice in Section 12 to provide building protection from localised instability and future regression at the head of the gullies.

- Section 8 was assessed within the central southern portion of the site within the proposed fill area and permanent batter. The fill batter within this portion of the site is approximately 14 degrees. Based on our slope stability assessment, the minimum factors of safety for the static, transient and seismic case were met.

- Sections 9 and 11 were assessed for the western portion of the site which generally comprise fill heights in the order of 2.0 m to 8.0 m in height. The slopes were approximately 20 to 24 degrees in grade. Based on our slope stability assessment, the minimum factors of safety for the static, transient and seismic case were met.

- Section 12 includes the proposed stormwater attenuation pond. This includes excavations upslope to form a level pond base. A fill berm approximately 4.0 m high is proposed with slope grades of 1V:2H. The slope stability modelling indicates that the minimum factors of safety for the static, transient and seismic case were met. In the post flooding (extreme groundwater case) failure surfaces below 1.3 were estimated to occur across the downslope stream bank. However, the berm was offset by 7.5 m from the estimated failure surface. Because of our stability modelling, we do not consider that stability measures are required. However, additional design co-ordination will be required with the civil engineer.

Although our slope stability modelling indicated that the majority of the proposed boundary slopes meet Auckland Council factor of safety requirements, further co-ordination on locations of buildings post earthworks will be required to refine the extent of engineered fill slopes during detailed design.

Additional geotechnical investigations along the proposed fill alignments are also recommended to refine the ground model and analysis during detailed design. This will comprise additional machine boreholes, cone penetration tests and specific laboratory testing. The leapfrog ground model will be further developed as these investigations are completed.

# 10 Assessment Against the Section 106 RMA

We do not consider the proposed development to be subject to significant subsidence (including liquefaction), falling debris or inundation by soil or rock in accordance with the provision of Section 106 of the Resource Management Act 1991.

Provided the recommendations presented in this report and future design level reports are followed, we consider it unlikely that future erosion or slippage could pose a risk to structures and infrastructure will occur.

We do not consider that future development and use of land is likely to accelerate, worsen or result in material damage to the land provided that proper engineering practices are following during the proposed development.

# 11 Geotechnical Earthworks Recommendations

## 11.1 General

ENGEO have been provided with the Maven Associates Limited Proposed Earthworks Cut/Fill Plan – Retirement Village, dated March 2025 (ref: 147016 rev. A, drawing no. C220- RV 1). The plan shows the proposed lot boundaries along with cut and fill areas. The plans show a total cut volume of 502,600 $m^3$ and a total fill volume of 424,170 $m^3$.

Based on our desktop review and site investigations, we consider that the site is generally suitable for the proposed earthworks provided that the works are carried out in accordance with the following site-specific recommendation, our earthworks specification (yet to be released), Auckland Council's Code of Practice and other appropriate standards.

## 11.2 General Site Works

### 11.2.1 Initial Site Preparation

The site has historically been used for forestry related activities, with multiple planting and harvesting events. As such, the removal of tree stumps, topsoil and tree debris 'slash' will need particular attention by the earthwork's contractor at the commencement of earthworks. Where stumps and root systems remain, it is important that all tree stumps and large roots (greater than 25 mm) are completed removed from the building platforms / road subgrade and immediate surroundings, and that the excavations created are filled with compacted hardfill or engineered site won cohesive soils to suitable standards.

It is likely that a substantial volume of this material will need to go off-site as it will be unsuitable for use, unless there are locations for it to remain that are outside of overland flow paths and / or flood zones etc. Should unsuitable soil be proposed to be retained on-site, then the geotechnical engineer should be informed to provide comment on the location due to the sites sensitivity to fill loads / slope instability.

Should low strength natural soils be encountered under topsoil / debris be encountered on-site, these should also be undercut and replaced with engineered fill to provide uniform ground conditions for future foundations and pavement subgrades.

## 11.3 Material Suitability

Excavations are proposed through the central and eastern portion of the development area. The excavations are generally in the order of 2.0 m to 4.0 m below ground level across most of the proposed excavations, except for the eastern portion of the site where they increase to a maximum depth of 10.0 m below ground level. Based on our groundwater monitoring, we expect that most of the excavation between 2.0 m to 4.0 m below ground level will not intercept groundwater. However, for excavations greater than 4.0 m in height, we expect that groundwater will be intercepted.

We expect that excavations in the upper 4.0 m soil profile will comprise silty clay and clayey silt with variable sand content. This material is considered suitable for use as engineered fill provided that sufficient conditioning is completed. For wet material, this will likely require discing and mixing during periods of dry weather. Moisture contents are expected to increase with depth and the soils will require additional moisture conditioning so that they can achieve suitable compaction.

For excavations greater than 4.0 m (i.e. large cuts within the eastern portion of the site), we expect that this material will comprise sandy silt and clayey silt with variable sand content. We expect that the sand content will increase with depth as well as the moisture content. As there is a net cut excess of approximately 80,000 $m^3$, we recommend that wet soils won from the deeper excavations are not utilised for cut to fill operations.

Given that these excavations are expected to intersect groundwater, we recommend that the earthworks contractor considers the type of machinery to be used in this area of the site. Conventional scrapers may not be suitable in the larger excavations where wet soils of sandy composition may be encountered. Thus, excavators and dump trucks may be required to excavate the deeper portions of the bulk excavations.

## 11.4 Fill Placement

The earthworks plans indicate significant filling is proposed adjacent to low lying gully features and overland flow paths. The ground investigations completed indicate that the gully features are underlain by firm to stiff alluvial sits and clays with soft to firm organic silts and clays. Following stripping within the proposed fill areas, localised undercuts may be required to remove the remaining soft material and form a stiff subgrade suitable to receive filling. We expect that this will be localised to the flanks of overland flow paths and mapped riparian margins.

If it is uneconomical / or not feasible to undercut down to the underlying competent soils, then we recommend that a bridging layer comprising a layer of Bidim A29 geocloth (or equivalent) with approximately 500 mm of Soft Pit Run (SPR) is placed and compacted prior to undertaking bulk filling with cohesive soils. ENGEO will need to undertake an assessment of the subgrade following stripping to provide advice during earthworks. Additional remedial measures may be required as described in Section 11.6

## 11.5 Subsoil Drainage

As noted in Section 7.2, groundwater was measured at depths ranging from 4.0 m to 8.0 m across the site. For most of the earthworks, we do not expect that groundwater will be intercepted and subsoil drains are unlikely to be required.

For excavations in the eastern portion of the site greater than 5 m below ground level. We expect that groundwater will be intercepted. A series of subsoils drains at the new subgrade elevation may be required to manage groundwater. Conceptually, these will comprise a grid of 150 mm high density perforated flexible drain pipes within stormwater service trench lines to manage groundwater. We envision that these will be discharged into the base of riparian margins and / or overland flow paths. Discharge of the subsoil drains will be controlled onto a rip-rap swale outlet.

For bulk fill areas, subsoil drains are recommended at the interface between native subgrade and fill batters on the boundary of the site. The subsoil drains will likely comprise 150 mm perforated high density socked flexible drain pipes benched into the slope with drainage metal prior to the placement of engineered fill and construction of engineered designed slopes. The subsoil drains will discharge on designed outlet points on the natural overland flow path and provide groundwater control for the engineered designed fill slopes.

## 11.6 Fill Induced Settlement

As described in Section 9.2, proposed earthworks will induce consolidation settlement of the underlying native soils. Following placement of filling, monitoring of the settlement plates are recommended for such a time that settlement has sufficiently attenuated. Based on our experience of similar soils, the settlements may take 12 to 18 months within the larger fill areas due to the cohesive nature and low permeability of the underlying soils.

Undercutting of weaker near surface soils can be undertaken during initial stripping to limit total settlements. Site observation by a ground engineering professional is recommended to observe the subgrade conditions and provide advice on undercut requirements prior to the placement of fill.

Alternatively, if the project programme does not allow, then a ground improvement solution could be implemented below the proposed earthworks to reduce total settlements. Conceptually, this would likely comprise of either timber or reinforced concrete piles down to underlying extremely weak siltstone and sandstone to transfer fill loads to competent stiff strata. A load transfer platform comprising geogrid reinforced hardfill (indicatively 500 mm thick) subject to the size of piles and spacing.

Clearing of areas of the forest is required so that a suite of additional geotechnical investigation is undertaken within the area of the maximum proposed filling to refine our assessment. This will comprise combination of machine boreholes, cone penetration testing and specific laboratory testing. We would then prepare a geotechnical design report that incorporates a detailed fill induced settlement analysis in conjunction with detailed engineered earthworks fill slopes (MSE slopes, in ground walls and / or shear keys) for building consent.

## 11.7 Forestry Road Upgrade

The Maven Associates Limited cross section plan shows that it is proposed to upgrade Forestry Road west of the proposed retirement village due to flooding issues. The proposal includes a re-alignment of approximately 1.5 km of Forestry Road to the west of its current alignment. Earthworks fill is proposed on the eastern edge of the road alignment, while excavation and permanent batters are proposed on the western edge of the road alignment. We understand that where suitable batters cannot be formed, engineered retaining walls will be undertaken to support the proposed excavations.

The proposed fill is recommended to be benched into the existing soils. Given that some of the filling may be on the existing road alignment, it is likely that undercuts to remove undocumented fills will be required prior to the placement of filling. Permanent 1V:3H fill batters are proposed along the eastern edge of the road. We consider that these permanent batters will be suitable for total heights of up to 1.5 m. If fill heights are greater than 1.5 m then we recommend that geo-grid reinforcement is incorporated into the fill heights so that long term stability is met.

# 12 Slope Stabilising Measures

As discussed in Section 9.4, slope stability measures are required for the earthwork fill slopes on the northern boundary of the site and the existing slope on the eastern boundary of the site. A plan providing an indication on the length and extent of the proposed remedial solutions is included in Appendix 10. Further discussion on conceptual remedial solutions is included below, however these solutions will require detailed design suitable to support a building consent application prior to construction.

## 12.1 MSE Stabilised Slopes

Geo-grid reinforced earth fill slopes are required along the boundary of the site and for the proposed crossing on the access road into the site.

Conceptually, we have modelled 18 m long RE560 geogrid at 0.6 m vertical spacing to provide adequate factors of safety for the large fill slopes within the development area.

For the proposed accessway and bulk filling, 6 m long RE560 geogrids at 0.6 m vertical spacing will be required to provide adequate factors of safety for global stability. Due to the angle of the proposed slope facing a Terra mesh (or equivalent) solution will be required to support the face. Further co-ordination will be required during detailed design between the civil engineer for the location of structural elements (i.e. culverts) and how they are incorporated in the fill slope.

Benching into the existing native soils will be required to tie in the engineered fill to the existing slope at the commencement of filling. As outlined in section 11.4 and 11.5, depending on the competency of the underlying soils, undercuts to form a shear key including subsoil drains may be required where earth fills are within proximity to riparian margins or overland flow paths.

If it is not considered economically feasible, then a load transfer platform, comprising a grid of driven timber piles or reinforced concrete piles with a reinforced gravel raft between the top of piles and earthworks fill may be required.

Further investigations alongside the alignment of proposed fill are recommended once access to these areas is provided. These investigations will be undertaken to determine what is the best option for engineered fill slopes to support stability while also addressing settlement.

## 12.2 Inground Palisade Walls

Although the global factors of safety were met for the eastern portion of the site, the geomorphology assessment indicated that the crest of the slope can be defined as containing several over steepened gullies separated by narrow spurs. We note that these features were not captured on the publicly available contours. The head of these gullies are noted to be susceptible to localised instability due to concentrated surface water and localised increased gradients. Undercutting of the spurs resulting in progressive narrowing indicating the gullies are widening as they regress.

The over steepened portions of the slope at risk of instability were noted to be approximately 10 m high. The near surface shallow soils in this area of the site comprise stiff to very stiff clayey silt and silty clay with variable sand content. The sand content within the soil matrix and strength generally increases in depth.

Thus, we have considered a regression angle of 30 degrees across the 10.0 m high head of the gullies for the future stability of the buildings. The architectural masterplan indicates that proposed building platforms are approximately 8.0 m to 20.0 m from the crest of slope and head of the gullies. As such, buildings located within 20.0 m of these over steepened portions of the slope will require an in-ground palisade wall to improve local stability. An indicative length of the proposed in-ground wall is included in Appendix 10. However, we consider that this is conservative and will likely be reduced during detailed design when building platform locations are further detailed.

The size of the in-inground palisade walls will largely be governed by proximity to the crest of the slope. Based on the proposed building platform alignments, the in-ground palisade wall will be located approximately 5.0 m to 20.0 m from the crest of the slope and the effective design retained height will range from 1.0 m to 5.0 m in height and will likely comprise a cantilever timber (350 to 450 mm diameter) or reinforced concrete retaining wall (500 to 750 mm diameter) spaced at 1.2 m to 1.5 m centers depending on final location and alignment. Detailed design of these in-ground palisade walls will be required to support a building consent application.

## 13 Permitted Activity Assessment - Auckland Unitary Plan

The Maven Associates Earthworks Plan indicates that excavations up to 10.0 m are proposed within the eastern portion of the development area. The plan indicates that post earthworks levels are proposed at approximately 78 m RL. Based on our groundwater monitoring undertaken proximal to the proposed excavations within MBH06, a conservative groundwater depth of 4.2 m below ground level at the highest point of the site has been adopted for the purposes of this assessment.

Auckland Council require an assessment against the Auckland Unitary Plan (AUP): Operative in Part (Table E7), where proposed excavations may extend below the groundwater table within the site. The tables in Appendix 11 present an assessment against Standards E7.6.1.6 and E7.6.1.10

Based on the assessment against the Auckland Unitary Plan, a consent for active dewatering, impeding groundwater and groundwater drawdown will be required for the proposed development. However, we do not consider that a detailed assessment of effects is required as the excavations and expected groundwater drawdown will occur in a rural setting surrounded by forest that is not considered to be subject to adverse effects. Consequently, a ground settlement monitoring contingency plan (GSMC) is also not required.

## 14 Preliminary Development Recommendations

The following preliminary development recommendations can be applied to the retirement village development area. These recommendations are preliminary only and should only be used for concept design, discussion and feasibility studies. Further geotechnical assessments will be required to support detailed design and applications for building consent.

## 14.1 Preliminary Expansive Soil Classification

As we have not completed laboratory testing as part of this phase of work, we recommend that a preliminary expansive soil class of H1 (High) in accordance with AS 2870 is considered for foundation design.

Following earthworks, we recommend that a suite of soil samples is retrieved from building platforms to undertake expansive soil laboratory testing to provide lot specific recommendations.

## 14.2 Preliminary Foundations

We expect that the independent living units / villas will be one to two storeys in height.. As such, a preliminary geotechnical ultimate bearing capacity of 300 kPa should generally be available for waffle raft and / or shallow strip and pad foundations constructed on engineered fill and on undisturbed natural ground.

However, due to variations in measured shear strengths within the natural ground profile and the variable depths of cut and earthworks proposed across the site, a preliminary geotechnical ultimate bearing capacity less than 300 kPa may be specified for shallow pad and strip foundations in some areas of deeper cut. This is to be confirmed following completion of earthworks.

For larger buildings (three stories or more), shallow foundations may be suitable depending on building loads and location within site. Alternatively piled foundations into underlying transitional zone soil or bedrock of the East Coast Bays formation are considered an alternative solution. Specific advice on piles can be provided as required.

Some building platforms may extend across cut-to-fill transitions. For cuts and fills (less than 1.0 to 3.0 m), this is not expected to pose an issue. However, for significant cuts greater than 4.0 m, the subgrade will have varying stiffness. Structural design of foundations and slabs on grade will need to account for potential differential stiffness and include structural joints. This should be addressed in detailed design and will largely depend on the building foundation locations relative to the proposed earthworks.

## 14.3 Roading and Pavements

An inferred preliminary California Bearing Ratio (CBR) of approximately 4% is considered appropriate for the inorganic natural silt and clay soils at the site. Higher values will likely be achievable in the fill areas. Specific *in situ* and laboratory testing of the exposed subgrade is recommended following earthworks and prior to finalising pavement designs.

It should be noted that actual CBR values can be highly affected by moisture content (i.e. exposure to the elements) and trafficking (yielding the subgrade, resulting in weaving) and we therefore recommend that the subgrade is only trimmed to final level immediately prior to placing base course. Vehicles should not traffic the subgrade prior to placing sufficient thickness of base course. Additional subgrade improvement requirements may be necessary to achieve council requirements. This may include undercut and replacements, and / or the use of triaxial geogrid.

An assessment for CBR values should be undertaken on the subgrade following excavation to design level, and prior to placement of roading elements. If the design CBR is not met, further excavation may be required.

## 14.4 Retaining Walls

We have not been provided with specific retaining wall plans at this stage; however, we understand that retaining walls may be required to retain cut and fill earthworks across the site.

The design parameters in Table 7 may be adopted for the preliminary design of retaining walls up to 1.5 m in height, assuming competent natural ground and / or engineer certified filled prevails. Walls with retained heights greater than this should be the subject of specific ground investigations.

**Table 7: Retaining Wall Design Parameters**

| Material Type | Unit Weight ($kN/m^3$) | Friction Angle (Degrees) | Cohesion (kPa) | Undrained Shear Strength (Su) – (kPa) |
|---|---|---|---|---|
| Engineered Cohesive Fill | 18 | 32 | 4 | 110 |
| Firm to stiff Takaanini Formation | 17 | 29 | 4 | 60 |
| Stiff to Very Stiff ECBF residual Soils | 18 | 30 | 6 | 80 |

These values are appropriate for the existing soils identified in our soil testing. However, if significant variation or zones of soft material are encountered during the site works, then the matter should be referred to ENGEO for review and comment, as necessary.

The retaining wall designer should consider all appropriate surcharge loadings, back and toe slope angles. If the walls are flexible, the soil may be assumed to be in the active state and the soil pressure may be calculated using active conditions ($K_a$). If no significant movement is acceptable at the SLS level, or if the wall can deflect less than 0.3% of its height, then the at-rest condition should be used ($K_o$). The designer should also determine whether deflections of the wall are acceptable and therefore whether 'active' (Ka) or 'at rest' (Ko) lateral earth pressure design should be used.

All retaining walls, including foundation walls, should be back drained to prevent the build-up of hydrostatic pressures. The back drain should discharge to an approved outlet.

## 14.5 Surface Water Management

During construction, appropriate measures shall be undertaken to control and treat stormwater runoff, with silt and erosion controls complying with local body guidelines for erosion and sediment control. This is particularly relevant for the site due to the proximity to a stormwater receptor, being the stream and reserve to the north. Surface cut-off drains or appropriate stormwater flow paths should be maintained outside of the proposed development area, both during and following construction. These drains and impervious surfaces will divert water away from any buildings and minimise possible movement in sensitive soils during and post construction.

Stormwater from paved areas shall be taken in a piped system and disposed of into an approved stormwater system. Uncontrolled discharge onto land or uncontrolled disposal via in-ground systems must be avoided.

All service trenches should be capped with low permeability materials, so that excavations do not become points of entry for surface run-off.

### 14.6 Waste Water Management

GWE Consulting Engineers have completed the specific design of the proposed wastewater disposal fields. We have been provided with the GWE Consulting Engineers Proposed Retirement Village Wastewater Plan, dated 24 March 2025 (reference J6438-1 drawings 500 Revision 1). The plan presents the location of the proposed Wastewater disposal field. The drawings indicate that the fields are located a minimum of 15 m away from mapped overland flow paths and are on ground sloping 5 to 15 degrees. Based on the plan, the disposal fields are located well away from localised instability on the stream bank edges noted in our geomorphology study and we consider that these fields are unlikely to worsen or accelerate land instability.

We recommend that the on-site effluent treatment and disposal systems are designed in keeping with the relevant guidelines and standards and should avoid adding concentrated water to the steep slopes that have been identified to be at risk of slope stability. Based on site topography and proposed lot layout, it is likely that wastewater disposal fields will be positioned on or close to moderate sloping ground, particularly close to stream channels and incised gullies. We recommend that the wastewater designer confirms that the land application system is suitable for existing slope gradients and is suitably offset from overland flow paths.

In addition, system designers should take the following into account:

- Slope factors and potential effects of effluent loading on slope stability. Where fields are located on sloping ground or close to areas of potential instability, fields spacing should be increased where possible to avoid concentrated loading onto the slope.
- Soil and geology factors such as installing systems on old instability areas or on ridge spines with shallow depth to rock.
- Exposure to climatic elements; maximum exposure to wind and sun is recommended.
- Rainfall / seasonal water table – some sites will require groundwater cut-off drains and / or surface water diversion swales.
- Boundary, clean water, overland flow, wetland, and building setbacks.

## 15 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.

# 16 Future Work

We recommend that the following work is completed to support earthworks construction:

- Further investigations to support detailed design of engineered fill slopes and settlement analysis. This will consist of machine boreholes and cone penetration testing along with specific laboratory testing. The borehole investigations will target proposed locations of bulk filling. These investigations will be required in the previously inaccessible areas due to the presence of pine forest.

- Detailed design of engineered fill slopes (MSE walls or equivalent) suitable for building consent approval. This will include detailing geo-grid reinforcement and, if required, ground improvement options to limit settlement risk based on programme requirements.

- Preparation of an earthworks specification document to support contractor tenders. This document will also incorporate detailed engineered earthwork solutions.

- Construction Monitoring services (CM Level 3) to observe and undertaken compaction control testing during earthworks.

- Preparation of a geotechnical completion report and suitability statement at the end of earthworks construction prior to the commencement of building construction.

## 17 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, Rangitoopuni Developments LP C/- Avant Group 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. Although this is not a hearing before the Environment Court, I record that I have read and agree to comply with the Environment Court's Code of Conduct for Expert Witnesses as specified in the Environment Court's Practice Note 2023. I confirm that this report is within my area of expertise, except where I state that I rely upon the evidence or reports of other expert witnesses lodged forming part of the project's application material. I have not omitted to consider any material facts known to me that might alter or detract from the opinions expressed.

iv. 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.

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

vi. 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 (09) 972 2205 if you require any further information.

Report prepared by

[redacted]

[redacted]

Senior Geotechnical Engineer

[redacted]

[redacted]

Engineering Geologist

Report reviewed by

[redacted]

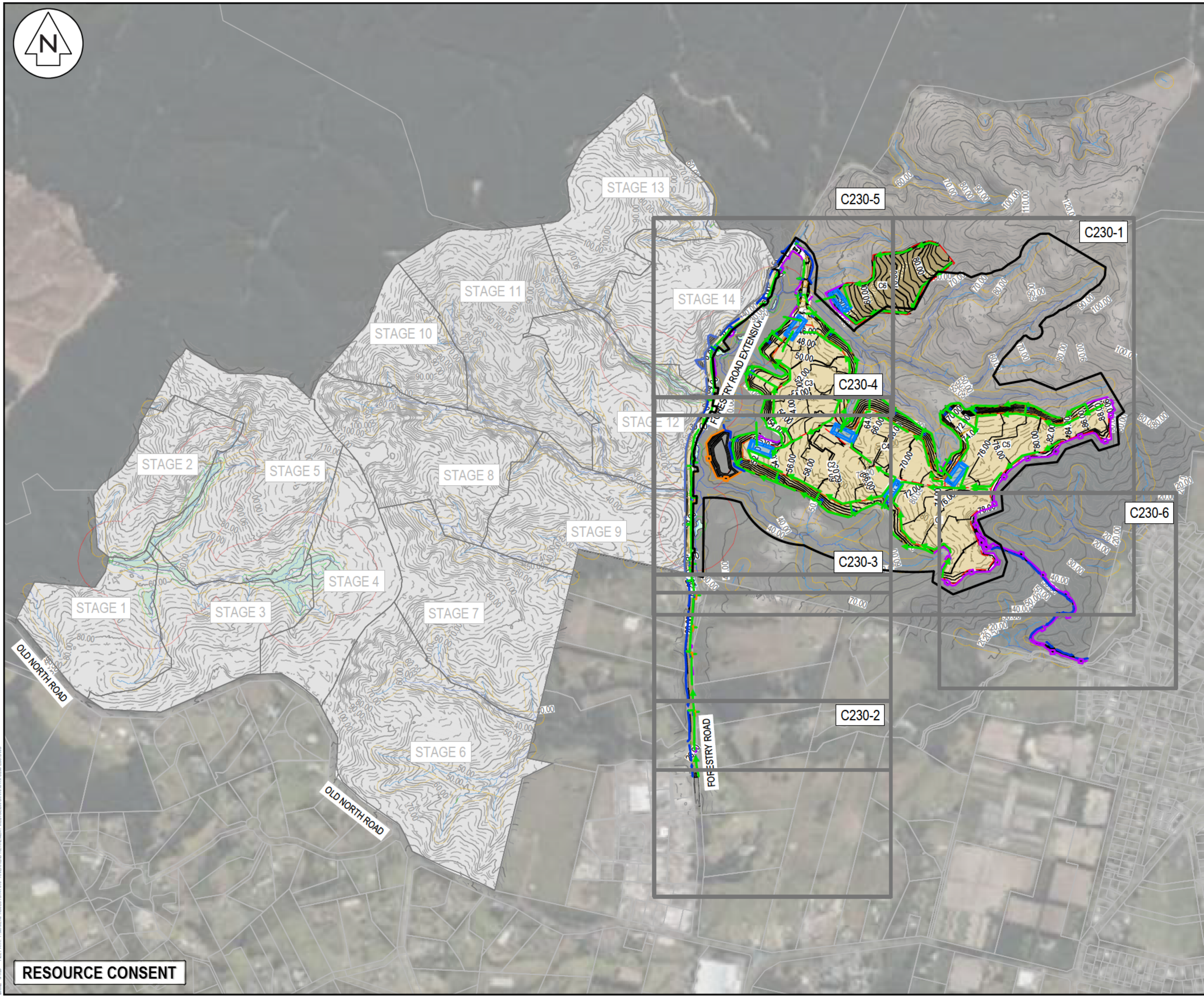
[redacted]

Principal Geotechnical Engineer

ENGEO

# APPENDIX 1:

Architectural and Civil Plans




NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000.
3. LEVELS IN TERMS OF THE AUCKLAND VERTICAL DATUM 1946.
4. ORIGIN OF LEVELS = SM XXXX SO XXXX(XXXX) PUBLISHED RL=XX.XX, SOURCED FROM THE LINZ DIGITAL GEODETIC DATABASE.
5. BOUNDARIES ARE SUBJECT TO FINAL SURVEY.

LEGEND
- EX BDY
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- EX MAJOR CONTOUR
- EX MINOR CONTOUR
- PR MAJOR CONTOUR
- PR MINOR CONTOUR
- PR EXTENT WORK
- PR CLEANWATER
- PR DIRTYWATER
- PR SILT FENCE
- PR SUPER SILT FENCE
- PR STOCKPILE
- PR SRP POND
- PR DEB POND
- PR DECANT BAR
- STAGE BOUNDARY
- EX STREAM
- EX WETLAND
- RIPARIAN YARD 20m SETBACK
- WETLAND AREA 100m SETBACK
- WETLAND AREA 10m SETBACK
- EARTHWORKS CATCHMENT AREA

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | HN | 03/2025 |

| | By | Date |
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| Survey | LIDAR | 03/2022 |
| Design | HN | 05/2022 |
| Drawn | HN | 03/2025 |
| Checked | RW/KH | 03/2025 |

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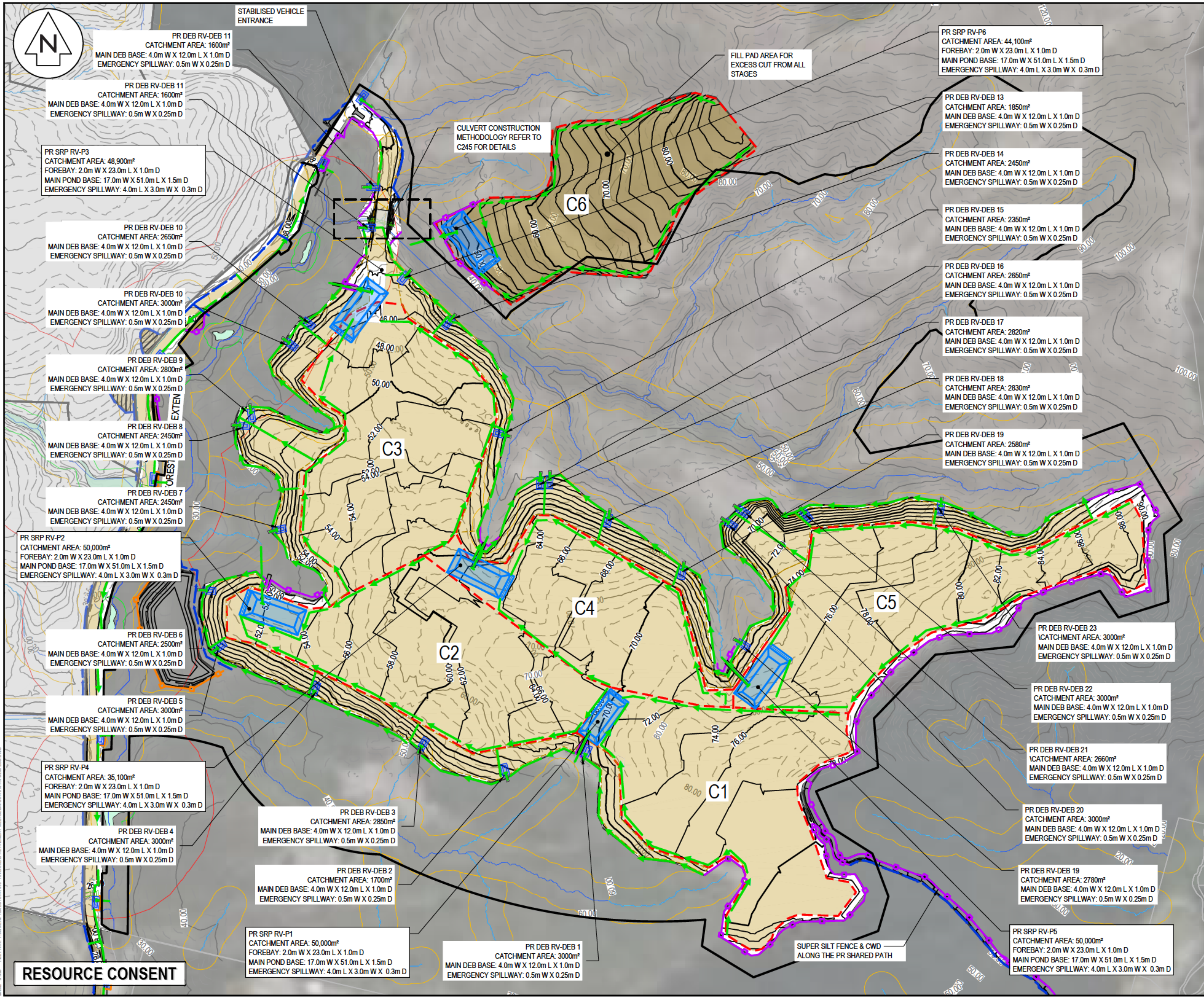
Project
DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title
RETIREMENT VILLAGE PROP. EROSION & SEDIMENT CONTROL OVERVIEW PLAN

| | |
|---|---|
| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230 |
| Rev | A |

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Title
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| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230-1 |
| Rev | A |


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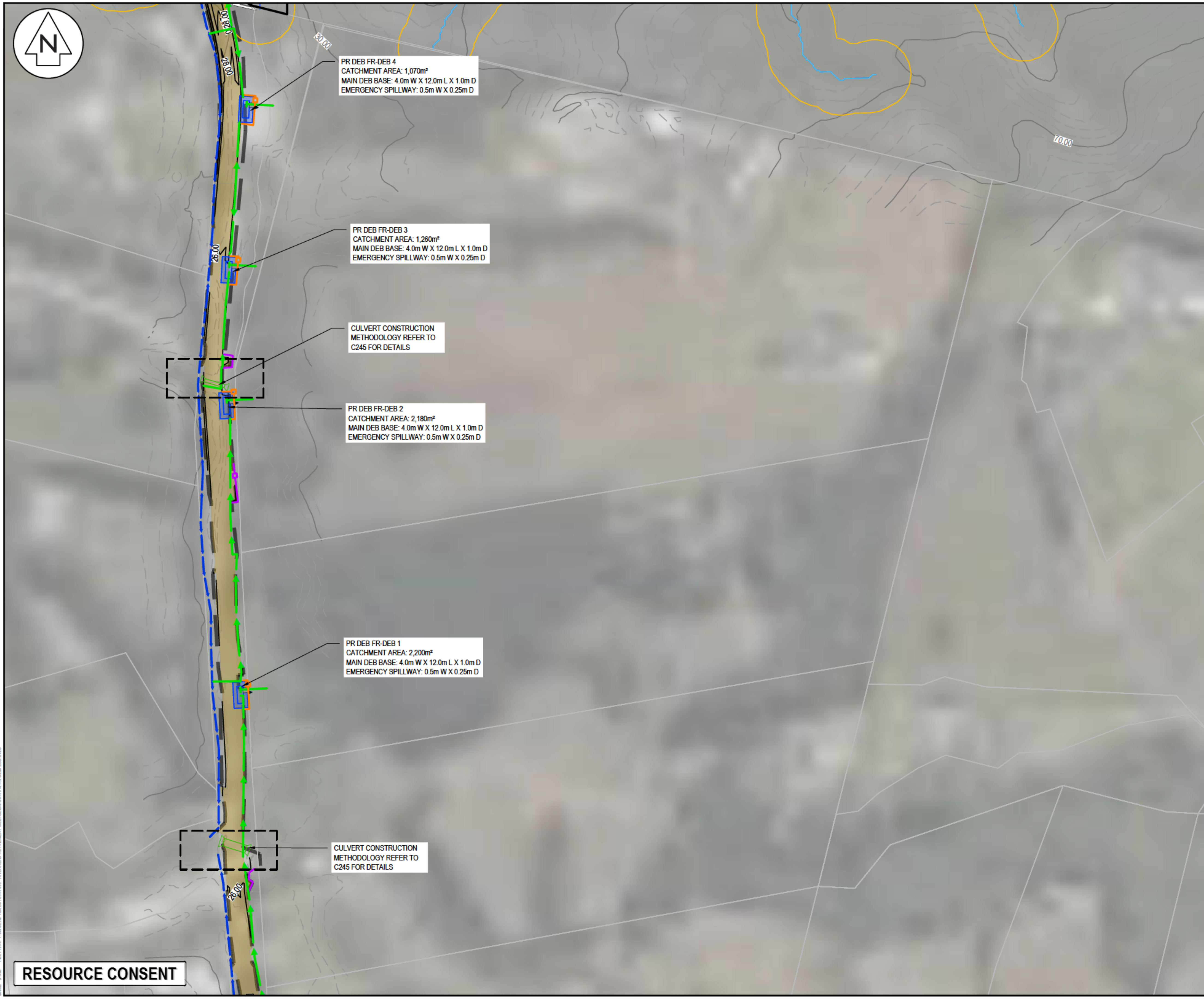
| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230-2 |
| Rev | A |

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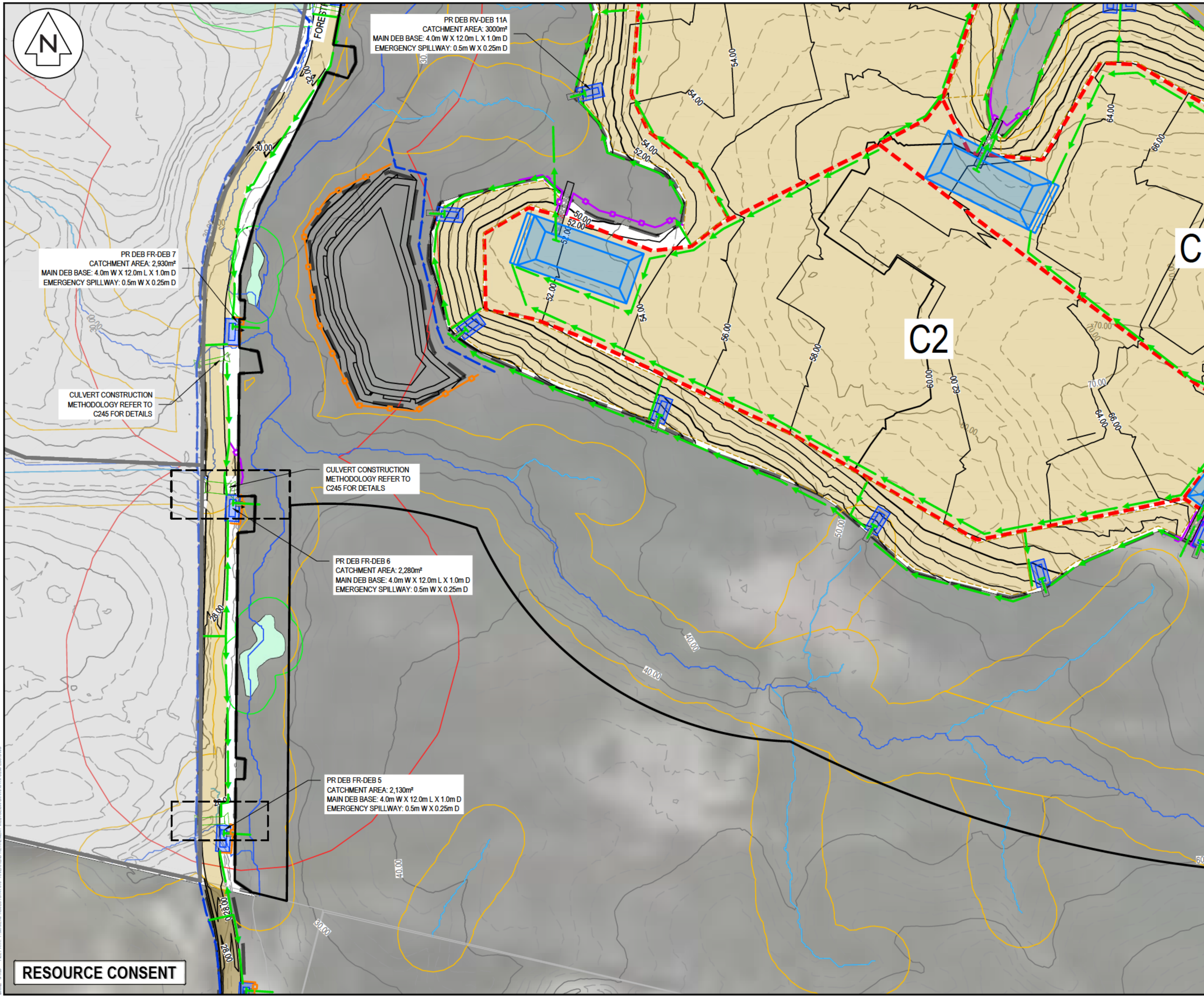
| Drawing no. | C230-3 |
| Rev | A |




**RESOURCE CONSENT**

NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000.
3. LEVELS IN TERMS OF THE AUCKLAND VERTICAL DATUM 1946.
4. ORIGIN OF LEVELS = SM XXXX SO XXXX(XXXX) PUBLISHED RL=XX.XX, SOURCED FROM THE LINZ DIGITAL GEODETIC DATABASE.
5. BOUNDARIES ARE SUBJECT TO FINAL SURVEY.

LEGEND

- EX BDY
- PR BDY
- EX MAJOR CONTOUR
- EX MINOR CONTOUR
- PR MAJOR CONTOUR
- PR MINOR CONTOUR
- PR EXTENT WORK
- PR CLEANWATER
- PR DIRTYWATER
- PR SILT FENCE
- PR SUPER SILT FENCE
- PR STOCKPILE
- PR SRP POND
- PR DEB POND
- PR DECANT BAR
- STAGE BOUNDARY
- EX STREAM
- EX WETLAND
- RIPARIAN YARD 20m SETBACK
- WETLAND AREA 100m SETBACK
- WETLAND AREA 10m SETBACK
- EARTHWORKS CATCHMENT AREA

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | HN | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | LIDAR | 03/2022 |
| Design | HN | 05/2022 |
| Drawn | HN | 03/2025 |
| Checked | RW/KH | 03/2025 |

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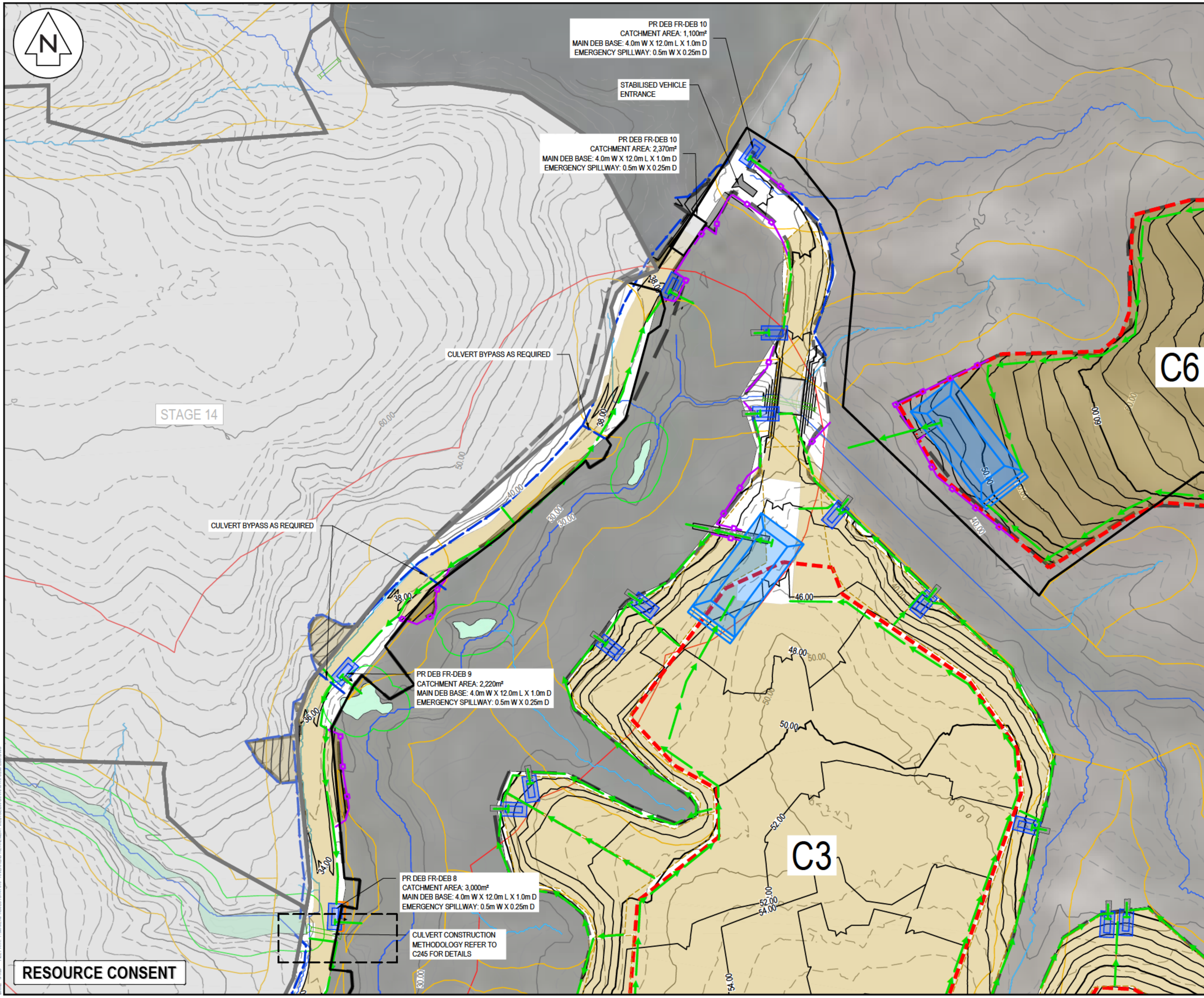
Project

**DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP**

Title

**RETIREMENT VILLAGE PROPOSED EROSION & SEDIMENT CONTROL PLAN**

| | |
|---|---|
| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230-4 |
| Rev | A |




RESOURCE CONSENT

NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000.
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LEGEND
EX BDY
PR BDY
EX MAJOR CONTOUR
EX MINOR CONTOUR
PR MAJOR CONTOUR
PR MINOR CONTOUR
PR EXTENT WORK
PR CLEANWATER
PR DIRTYWATER
PR SILT FENCE
PR SUPER SILT FENCE
PR STOCKPILE
PR SRP POND
PR DEB POND
PR DECANT BAR
STAGE BOUNDARY
EX STREAM
EX WETLAND
RIPARIAN YARD 20m SETBACK
WETLAND AREA 100m SETBACK
WETLAND AREA 10m SETBACK
EARTHWORKS CATCHMENT AREA

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | HN | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | LIDAR | 03/2022 |
| Design | HN | 05/2022 |
| Drawn | HN | 03/2025 |
| Checked | RW/KH | 03/2025 |


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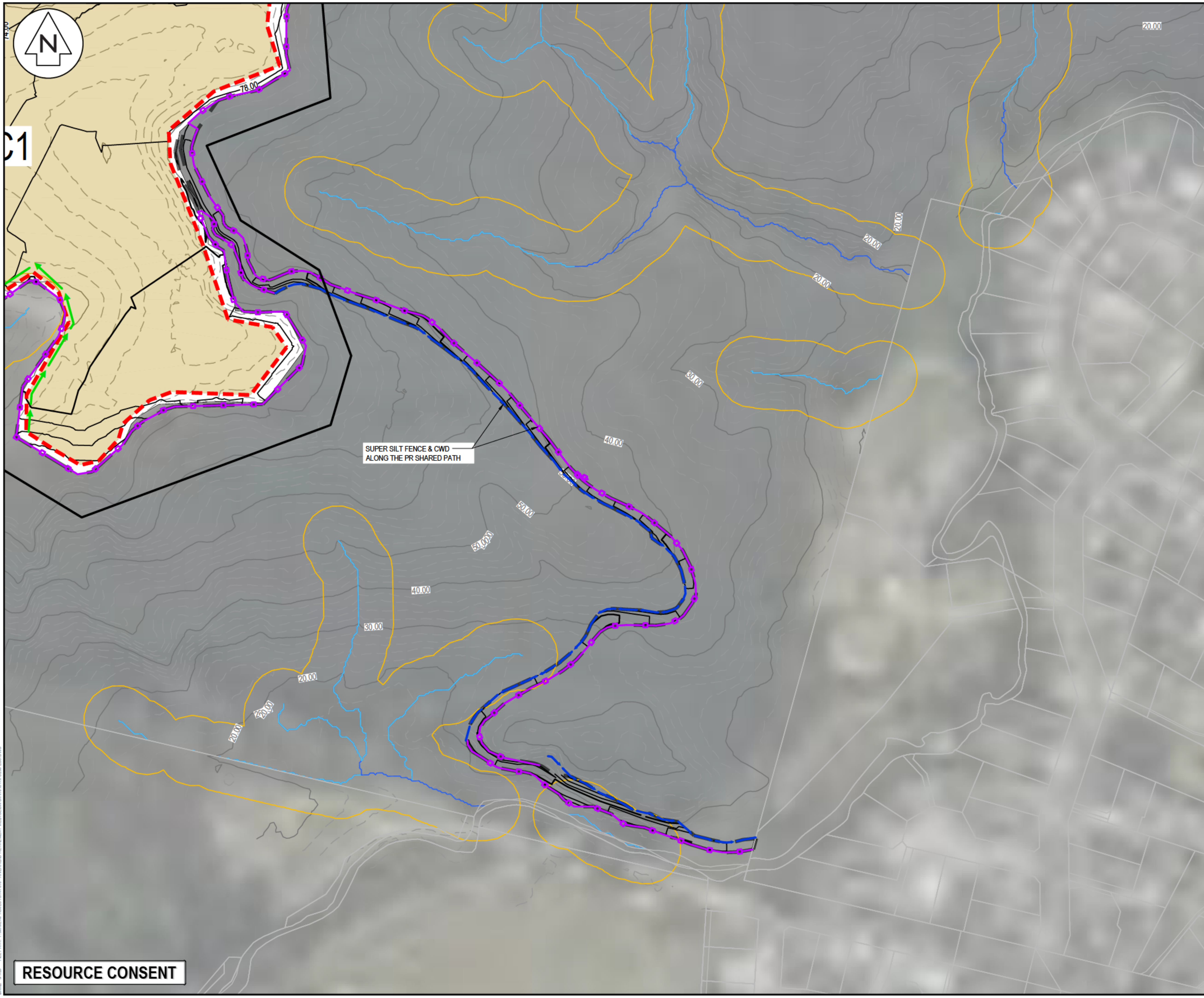
Project
# DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title
# RETIREMENT VILLAGE PROPOSED EROSION & SEDIMENT CONTROL PLAN

| | |
|---|---|
| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230-5 Rev A |




NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000.
3. LEVELS IN TERMS OF THE AUCKLAND VERTICAL DATUM 1946.
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LEGEND

- EX BDY
- PR BDY
- EX MAJOR CONTOUR
- EX MINOR CONTOUR
- PR MAJOR CONTOUR
- PR MINOR CONTOUR
- PR EXTENT WORK
- PR CLEANWATER
- PR DIRTYWATER
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- PR SUPER SILT FENCE
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- EX STREAM
- EX WETLAND
- RIPARIAN YARD 20m SETBACK
- WETLAND AREA 100m SETBACK
- WETLAND AREA 10m SETBACK
- EARTHWORKS CATCHMENT AREA

| Rev | Description | By | Date |
|---|---|---|---|
| | | | |
| | | | |
| | | | |
| A | RESOURCE CONSENT | HN | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | LIDAR | 03/2022 |
| Design | HN | 05/2022 |
| Drawn | HN | 03/2025 |
| Checked | RW/KH | 03/2025 |

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Project

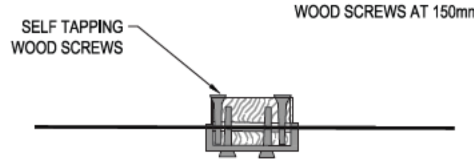
# DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title

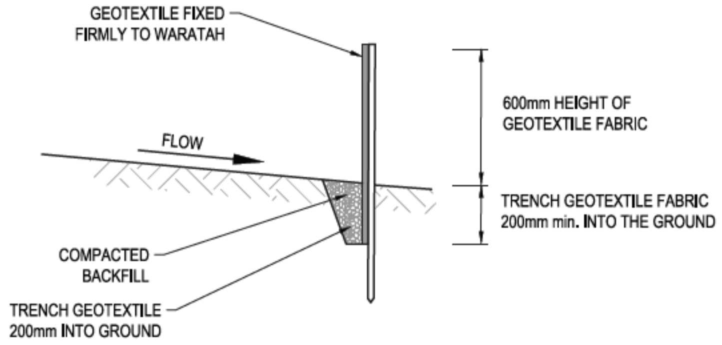
# RETIREMENT VILLAGE PROPOSED EROSION & SEDIMENT CONTROL PLAN

| | |
|---|---|
| Project no. | 147016 |
| Scale | 1:4000 @ A3 |
| Cad file | 147016-RV-C230 ESC.DWG |
| Drawing no. | C230-6 |
| Rev | A |

DATE: 6/1/25 FILE PATH: F:\MAVEN\PROJECTS\147016 - RIVERHEAD RETIREMENT VILLAGE\DWG\147016-RV-C230 ESC.DWG

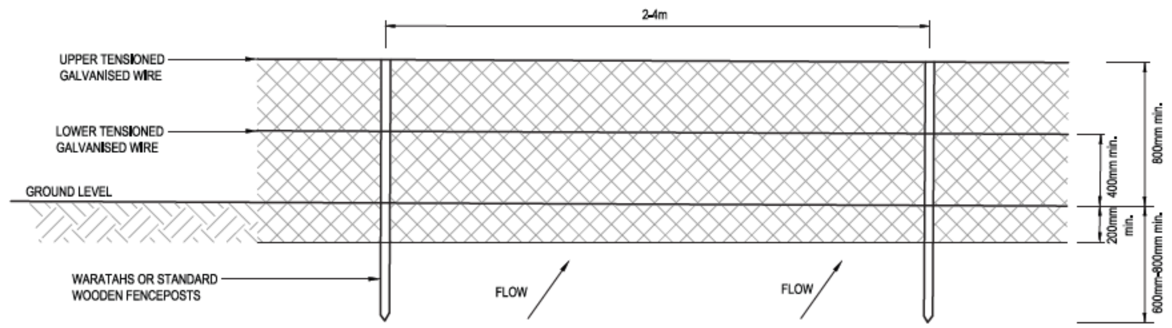



STANDARD DETAIL FOR FABRIC JOIN



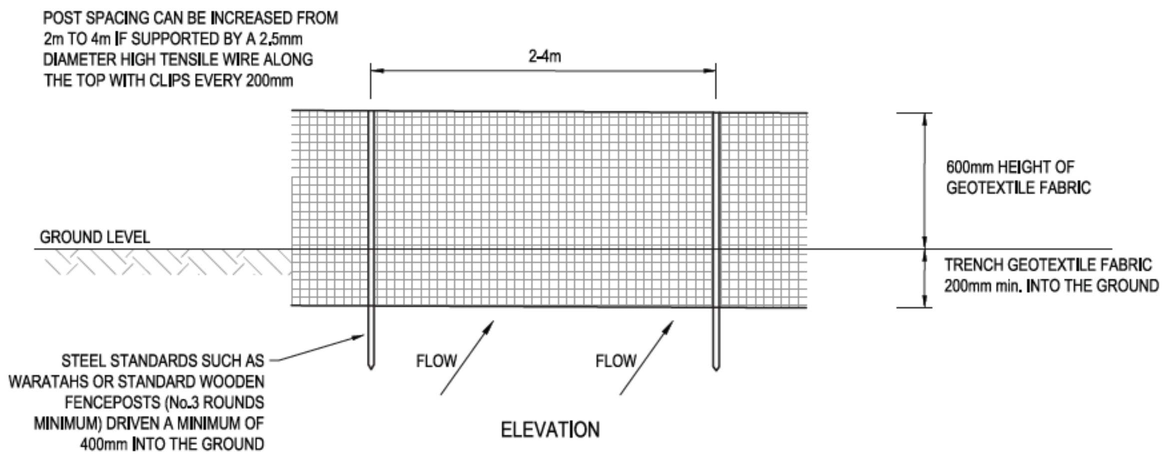

CROSS SECTION

SUPER SILT FENCE DETAIL

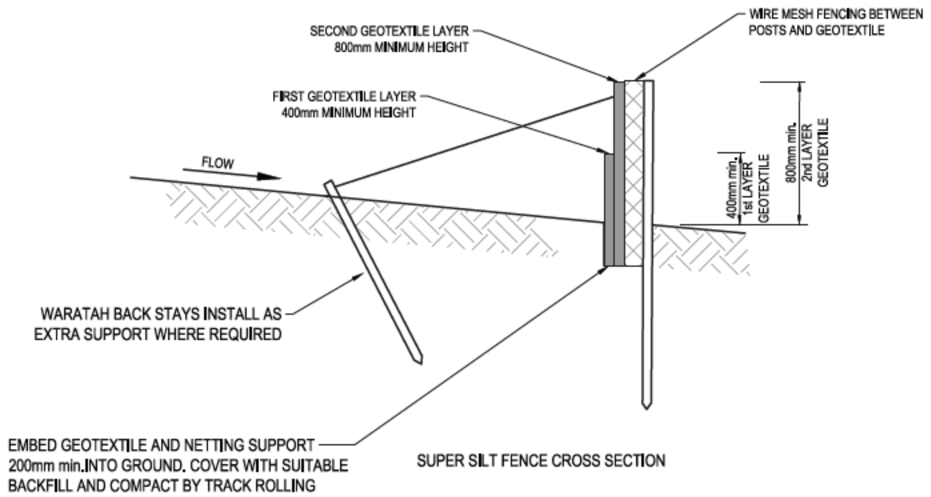



ELEVATION

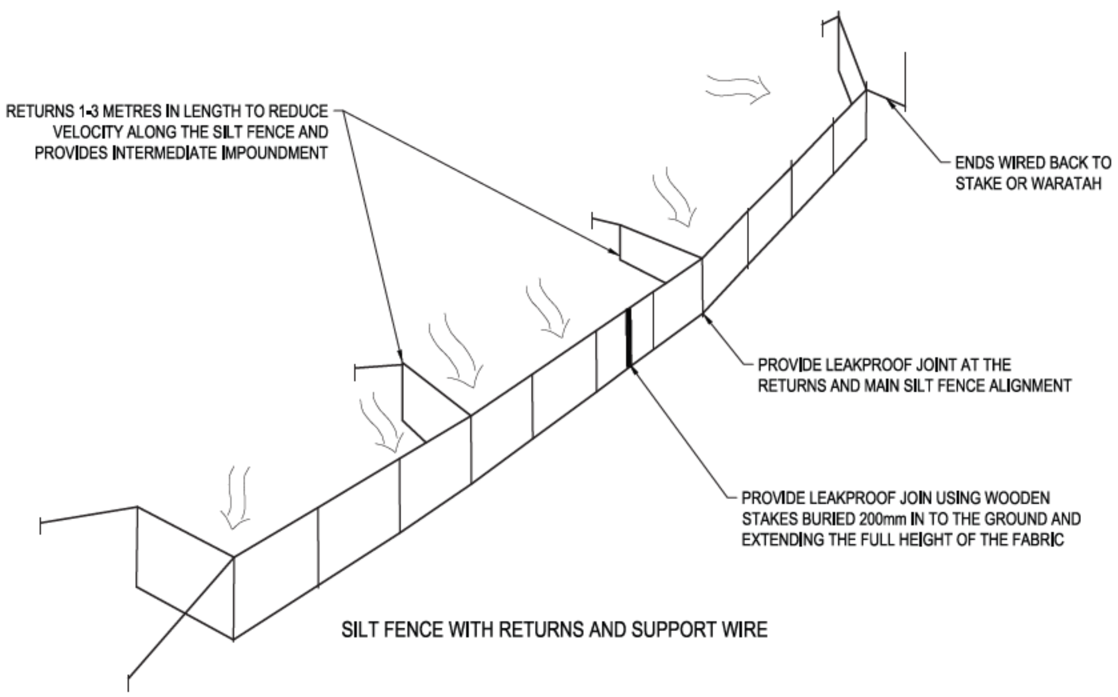
STANDARD DETAIL FOR SILT FENCE




ELEVATION




SUPER SILT FENCE CROSS SECTION




SILT FENCE WITH RETURNS AND SUPPORT WIRE

**RESOURCE CONSENT**

NOTES

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7. SEDIMENT CONTROL TO COMPLY WITH GD05 STANDARDS.

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | SP | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | -- | --/---- |
| Design | -- | - |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |


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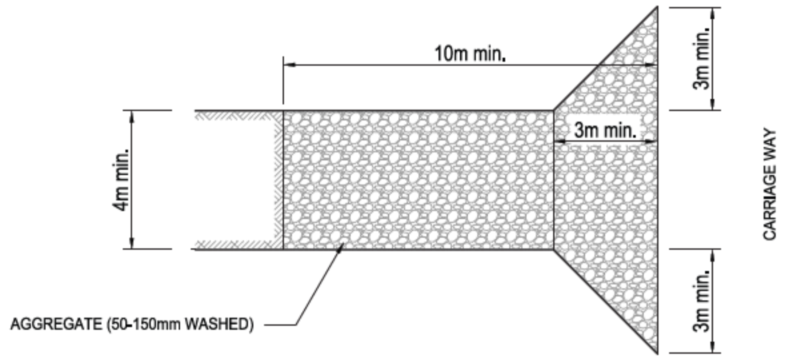
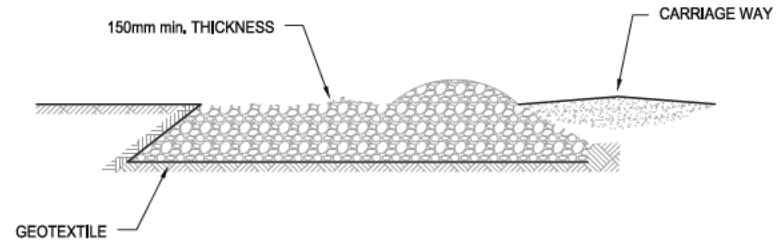
Project
## DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title
## RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS

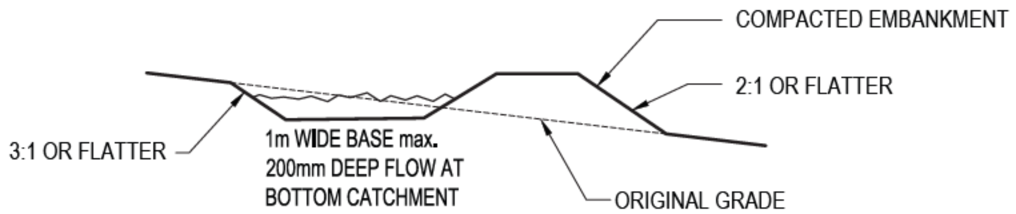
| | |
|---|---|
| Project no. | 147016 |
| Scale | N.T.S |
| Cad file | 147016-RV-C240 ESC STD DETAILS.DWG |
| Drawing no. | C240 |
| Rev | A |



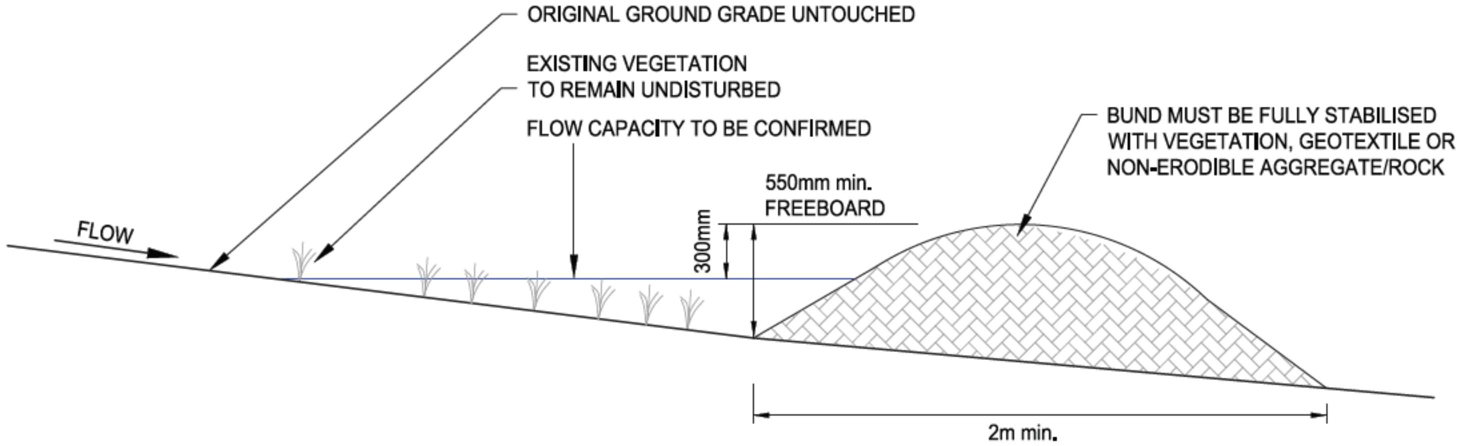

STABALISED VEHICLE ENTRANCE
PLAN VIEW




STABALISED VEHICLE ENTRANCE DETAIL
SIDE ELEVATION




TYPICAL CROSS SECTION OF A RUNOFF DIVERSION
TYPICAL DIMENSIONS UNLESS OTHERWISE NOTED




CLEAN WATER DIVERSION BUND DETAIL

RESOURCE CONSENT

NOTES
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6. CONTRACTOR SHALL PROVIDE ASBUILT OF WORKING SEDIMENT CONTROL DEVICES AND CONFIRMATION OF POND/DECENT VOLUMES TO ENGINEER.
7. SEDIMENT CONTROL TO COMPLY WITH GD05 STANDARDS.

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | SP | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | -- | --/--- |
| Design | -- | - |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |


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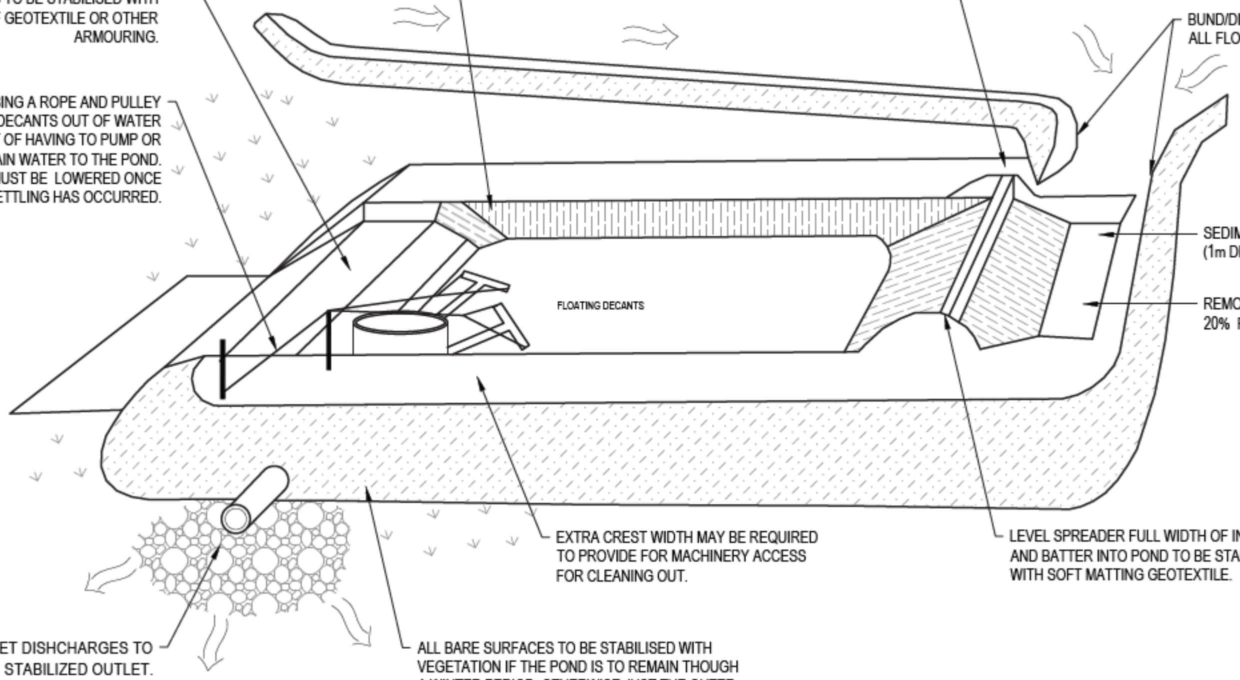
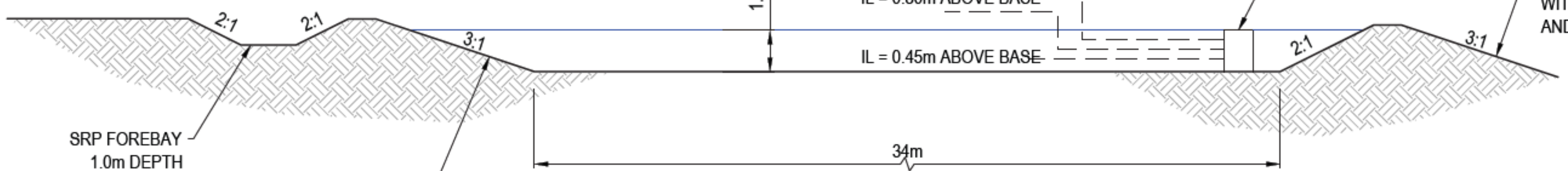
Project
DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title
RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS

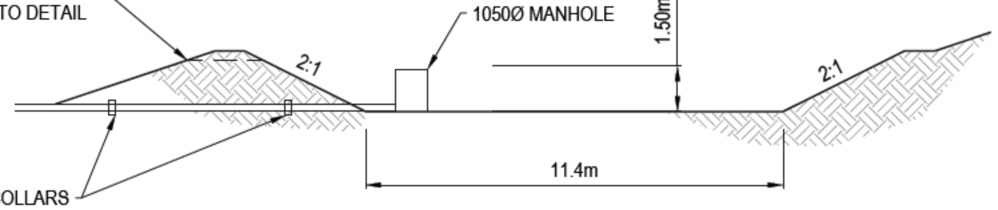
| | |
|---|---|
| Project no. | 147016 |
| Scale | N.T.S |
| Cad file | 147016-RV-C240 ESC STD DETAILS.DWG |
| Drawing no. | C241 — Rev A |



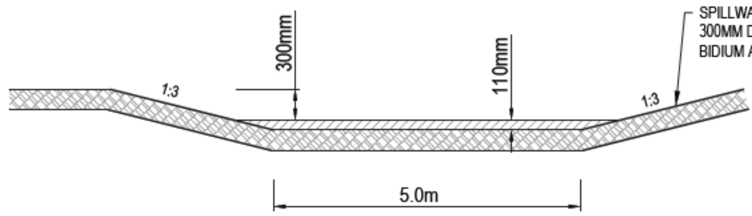

## SEDIMENT POND DETAILS
(NTS)




A / -- SECTION - SEDIMENT RETENTION POND
SCALE: N.T.S




B / -- SECTION - SEDIMENT RETENTION POND
SCALE: N.T.S




DETAIL I
EMERGENCY SPILLWAY

**RESOURCE CONSENT**

DATE: 4/9/025 FILE PATH: F:\MAVEN\PROJECTS\147016 - RIVERHEAD RETIREMENT VILLAGE\DWG\147016-RV-C240 ESC STD DETAILS.DWG

NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
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7. SEDIMENT CONTROL TO COMPLY WITH GD05 STANDARDS.

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | SP | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | -- | --/---- |
| Design | -- | - |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |

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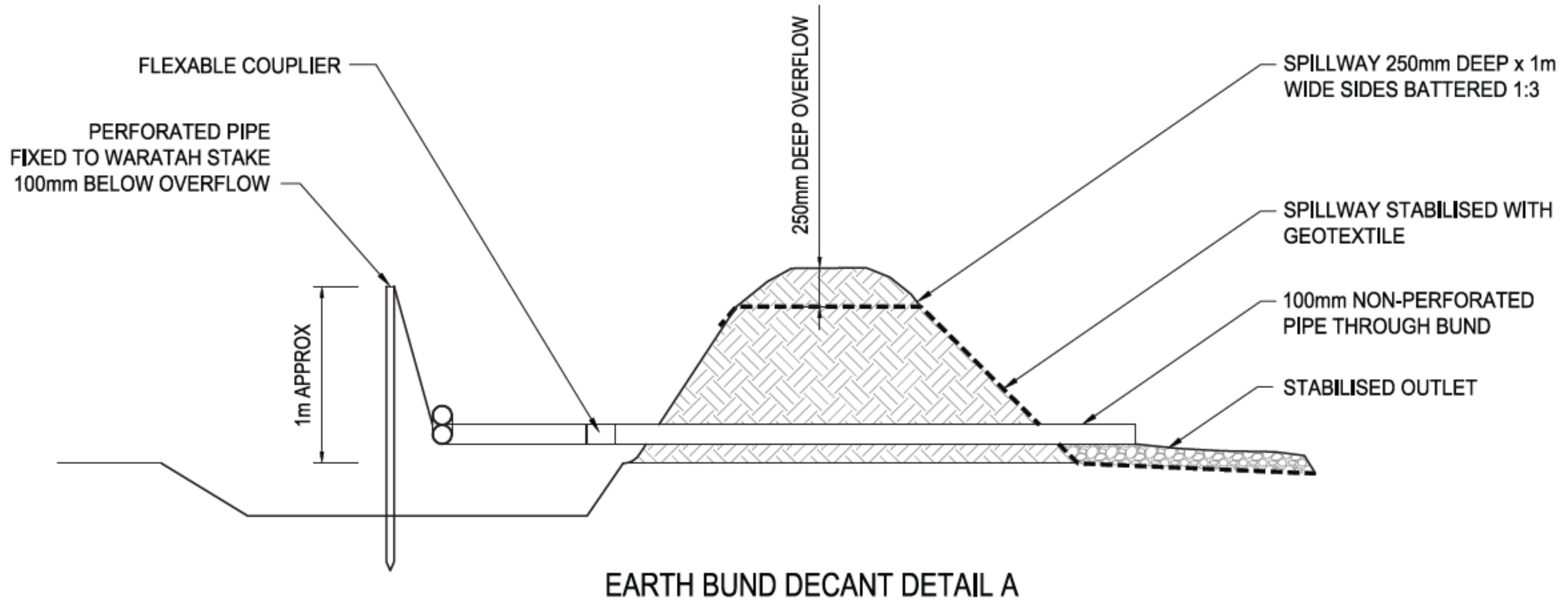
Auckland 1023

Project
**DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP**

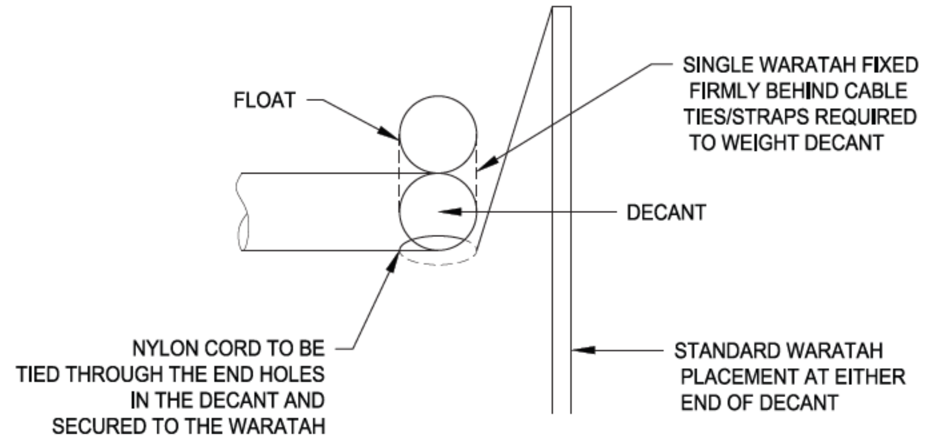
Title
**RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS**

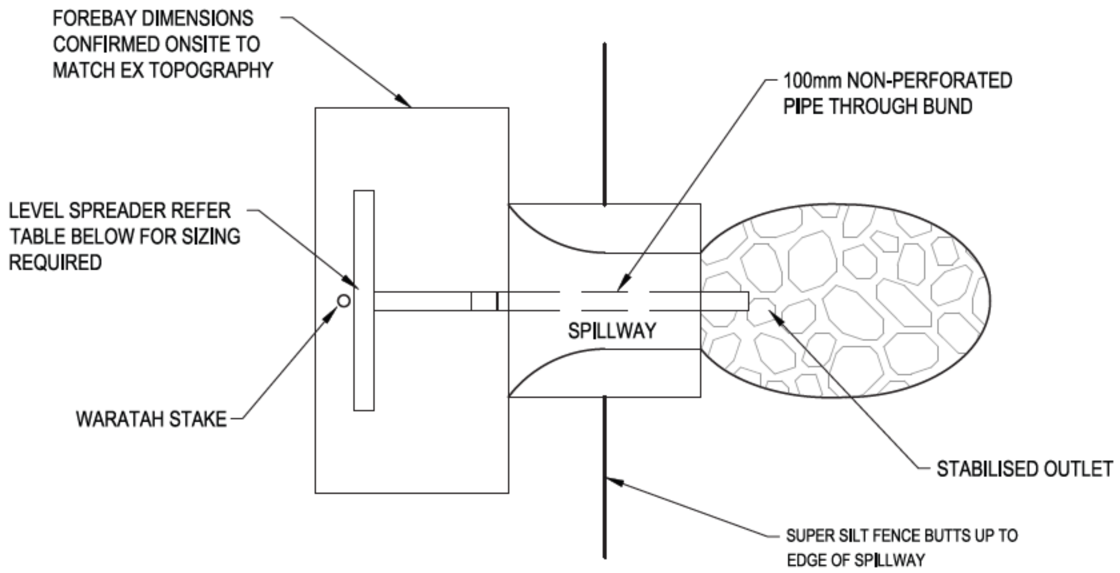
| | |
|---|---|
| Project no. | 147016 |
| Scale | N.T.S |
| Cad file | 147016-RV-C240 ESC STD DETAILS.DWG |
| Drawing no. | C242 |
| Rev | A |




## EARTH BUND DECANT DETAIL A

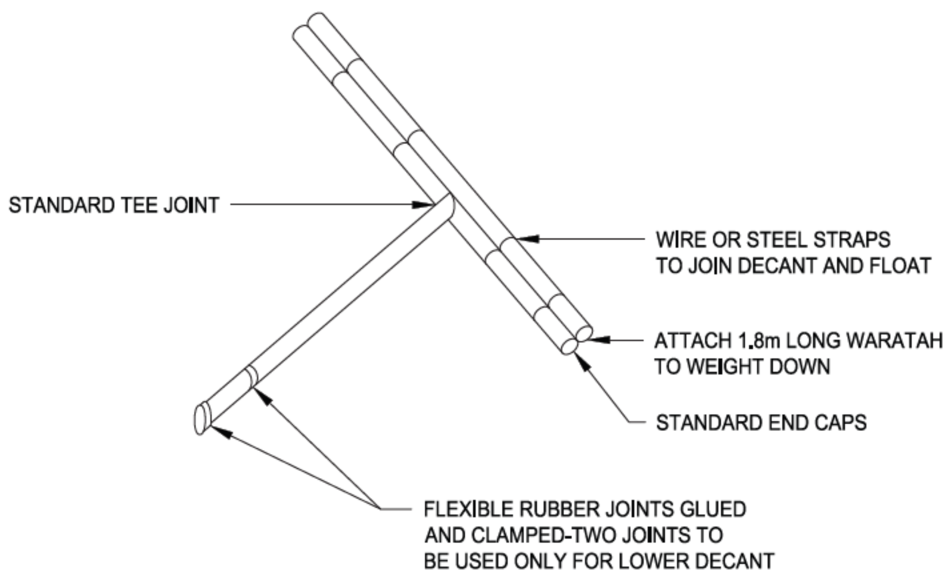
N.T.S




## DECANT PLAN VIEW

N.T.S




| LEVEL SPREADER DESIGN CRITERIA (20 YEAR STORM EVENT) | | | | |
|---|---|---|---|---|
| DESIGN FLOW ($m^3$/sec) | INLET WIDTH (m) | DEPTH (m) | END WIDTH (m) | LENGTH (mm) |
| 0-0.3 | 3 | 150 | 1 | 3 |
| 0.3-0.6 | 5 | 180 | 1 | 7 |
| 0.6-0.9 | 7 | 220 | 1 | 10 |

**RESOURCE CONSENT**

NOTES

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7. SEDIMENT CONTROL TO COMPLY WITH GD05 STANDARDS.

| Rev | Description | By | Date |
|---|---|---|---|
| A | RESOURCE CONSENT | SP | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | -- | --/---- |
| Design | -- | - |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |

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Project

**DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP**

Title

**RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS**

| | |
|---|---|
| Project no. | 147016 |
| Scale | N.T.S |
| Cad file | 147016-RV-C240 ESC STD DETAILS.DWG |
| Drawing no. | C243 |
| Rev | A |

NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH AUCKLAND COUNCIL STANDARDS.
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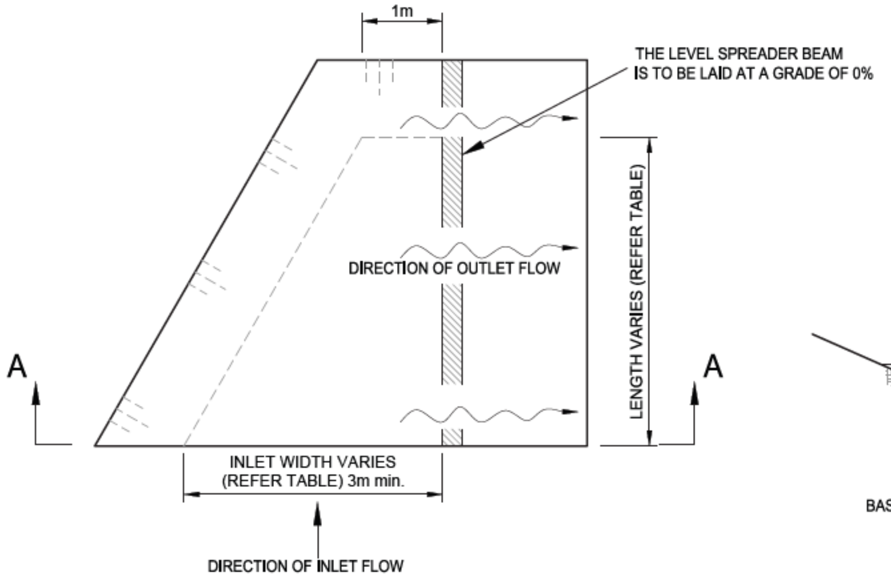
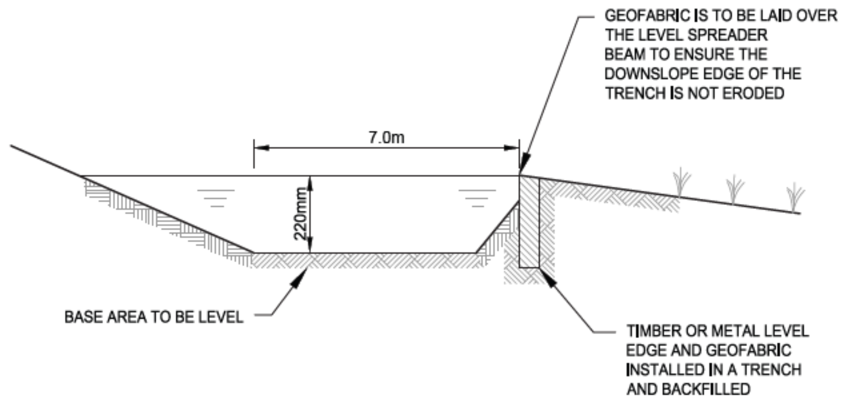
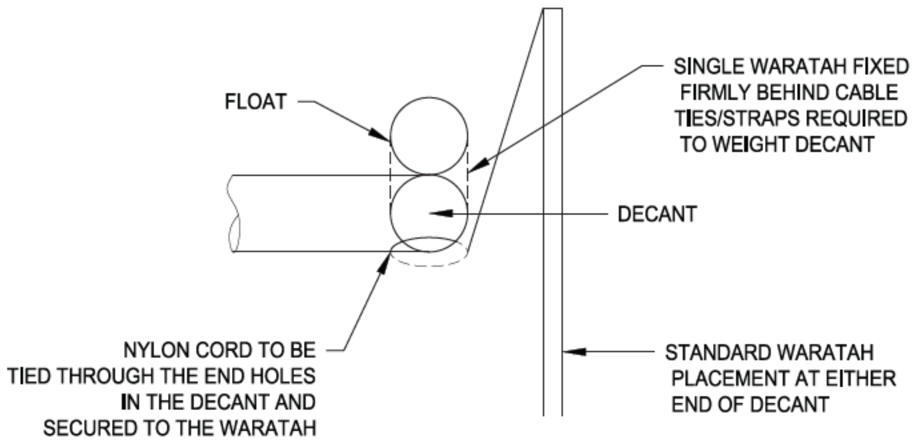


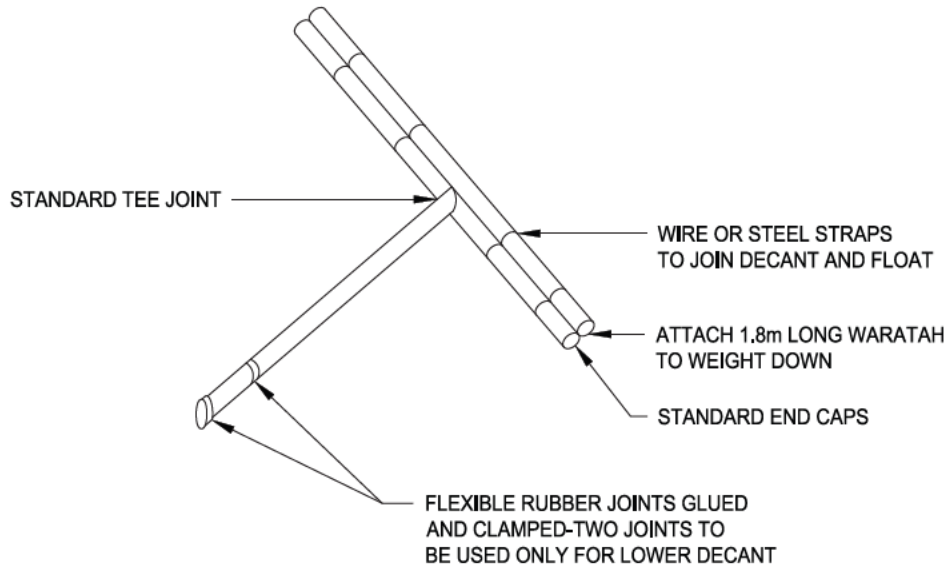

PLAN VIEW




SECTION A-A

| LEVEL SPREADER DESIGN CRITERIA (20 YEAR STORM EVENT) | | | | |
|---|---|---|---|---|
| DESIGN FLOW ($m^3$/sec) | INLET WIDTH (m) | DEPTH (m) | END WIDTH (m) | LENGTH (mm) |
| 0-0.3 | 3 | 150 | 1 | 3 |
| 0.3-0.6 | 5 | 180 | 1 | 7 |
| 0.6-0.9 | 7 | 220 | 1 | 10 |




| | | | |
|---|---|---|---|
| A | RESOURCE CONSENT | SP | 03/2025 |
| Rev | Description | By | Date |

| | By | Date |
|---|---|---|
| Survey | -- | --/---- |
| Design | -- | - |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |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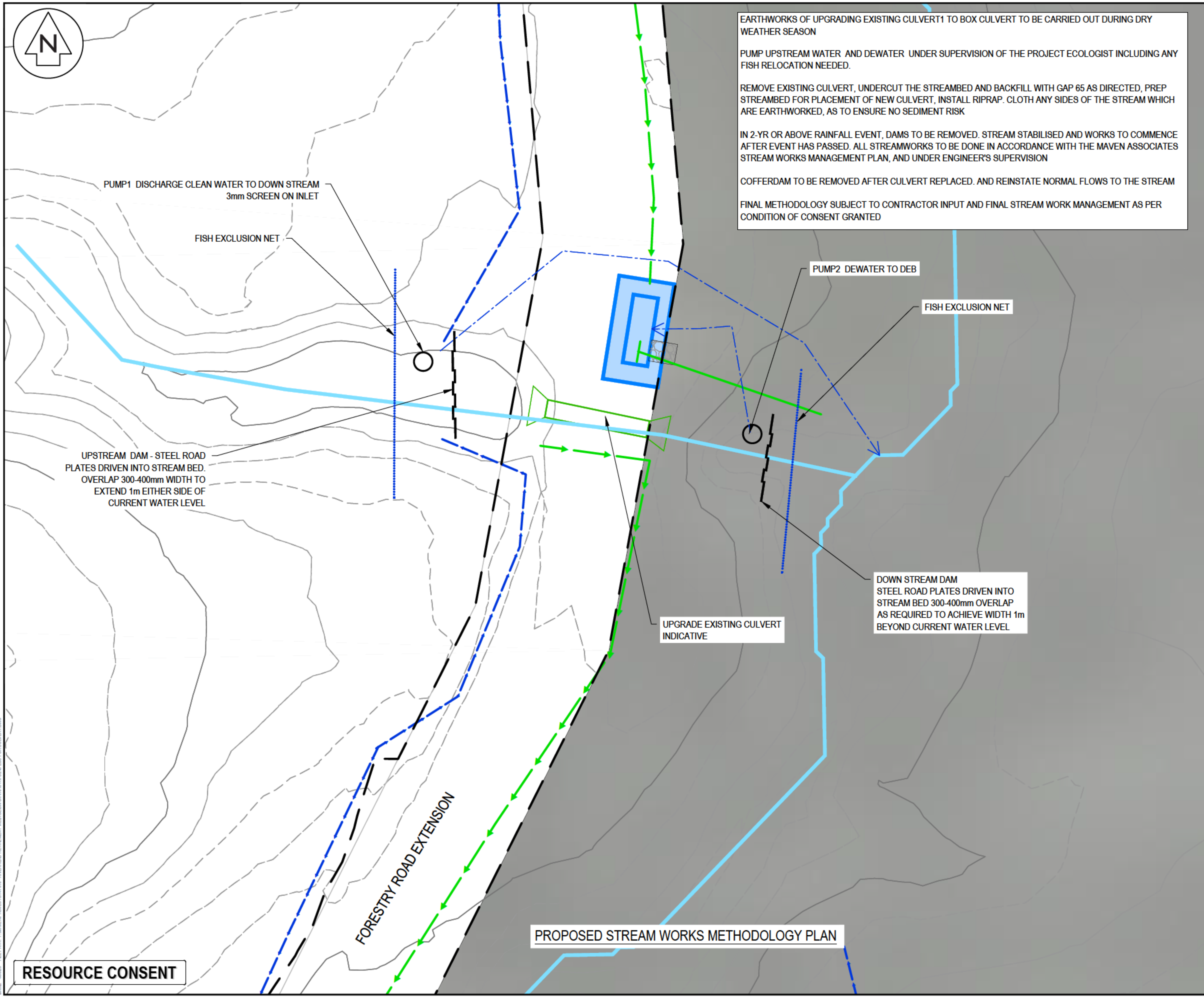
Project
DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP

Title
RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS

| | |
|---|---|
| Project no. | 147016 |
| Scale | N.T.S |
| Cad file | 147016-RV-C240 ESC STD DETAILS.DWG |
| Drawing no. | C244 |
| Rev | A |

RESOURCE CONSENT

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LEGEND

- EX BDY
- PR BDY
- EX MAJOR CONTOUR
- EX MINOR CONTOUR
- PR MAJOR CONTOUR
- PR MINOR CONTOUR
- PR EXTENT WORK
- PR CLEANWATER
- PR DIRTYWATER
- PR SILT FENCE
- PR SUPER SILT FENCE
- PR STOCKPILE
- PR DECANT
- PR DECANT BAR
- STAGE BOUNDARY
- EX STREAM
- EX WETLAND
- RIPARIAN YARD 20m SETBACK
- WETLAND AREA 100m SETBACK
- WETLAND AREA 10m SETBACK
- EARTHWORKS CATCHMENT AREA

| Rev | Description | By | Date |
|---|---|---|---|
| | | | |
| | | | |
| | | | |
| A | RESOURCE CONSENT | HN | 03/2025 |

| | By | Date |
|---|---|---|
| Survey | -- | --/--- |
| Design | HN | 03/2025 |
| Drawn | SP | 03/2025 |
| Checked | RW/KH | 03/2025 |


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Project

**DEVELOPMENT OF RIVERHEAD FOREST FOR RANGITOOPUNI DEVELOPMENTS LIMITED PARTNERSHIP**

Title

**RETIREMENT VILLAGE EROSION AND SEDIMENT CONTROL DETAILS**

| | |
|---|---|
| Project no. | 147016 |
| Scale | 1:500 @ A3 |
| Cad file | 147016-RV-C245 ESC FOR CULVERT.DWG |
| Drawing no. | C245 |
| Rev | A |