CKL

Planning | Surveying | Engineering | Environmental

# Otaihanga Retirement Village, Paraparaumu

## Stormwater Management Plan

## Summerset Group Holdings Ltd

# Document Information

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| **Client** | Summerset Villages (Paraparaumu) Ltd |
| **Site Location** | Otaihanga Village - 65 & 73 Ratanui Road, Paraparaumu |
| **Legal Description** | Lot 4 DP 58017, Certificate of Title 29D/823 (65 Ratanui Road)<br>Lot 3 DP 497389, Certificate of Title 734643 (73 Ratanui Road) |
| **CKL Reference** | A25556 |
| **Office of Origin** | Auckland |

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|---|---|---|---|
| **Author** | Frances Deamer-Phillips | | |
| **Signed** | [redacted] | **Date** | 20/10/2026 |

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|---|---|---|---|
| **Reviewed & Authorised By** | Bronwyn Rhynd | | |
| **Signed** | [redacted] | **Date** | 20/10/2026 |

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# Executive Summary

This report outlines the stormwater management strategy and assesses flood effects for the proposed Otaihanga Retirement Village at 65 and 73 Ratanui Road, Paraparaumu.

The Site comprises undulating dune terrain and low-lying areas that currently provide flood storage and infiltration. An existing highly modified intermittent drainage channel (Drain) crosses the Site, conveying runoff from the upstream catchment to Mazengarb Stream. The strategy responds to these features and the stormwater management requirements of Kāpiti Coast District Council and Greater Wellington Regional Council.

An integrated stormwater management approach combines bioretention devices for water-quality treatment and retention of the first 15 mm of rainfall from connected impervious surfaces, and constructed wetlands providing extended detention, flood attenuation and ecological habitat, with controlled discharge to the Drain. A piped network will convey primary runoff, with roads accommodating secondary overland flows. The wetlands will also contribute to the Site's ecological enhancement.

A dedicated flood-storage basin along the western boundary will receive overland flows from neighbouring properties. Reshaping the existing depressions will maintain their flood-storage and infiltration functions while accommodating the development.

Two-dimensional (2D) flood modelling assessed the 100-year ARI rainfall event, including climate change, and informed the proposed earthworks and stormwater layout. Modelled flood-level changes on neighbouring properties are generally less than 10 mm. The attenuation wetlands will store Site runoff, with outlet controls preventing discharge during elevated Drain levels and allowing stored runoff to be released after the upstream catchment peak has passed. This arrangement achieves hydraulic neutrality at the downstream Site boundary. Modelling of smaller rainfall events also indicates that post-development peak flows can be maintained at or below pre-development levels.

Overall, the assessment demonstrates that the strategy is feasible and can accommodate the proposed development without materially increasing flood risk beyond the Site. Final device sizing, earthworks levels, outlet controls and stage-specific drainage arrangements will be verified during detailed design, with operation and maintenance provisions supporting long-term performance.

# 1 Introduction

CKL has been engaged by Summerset Villages (Paraparaumu) Ltd to prepare a Stormwater Management Plan (SMP) and Flood Risk Assessment (FRA) for Otaihanga Village at 65 & 73 Ratanui Road, Paraparaumu (the 'Site') to support a resource consent application. The Site was recently rezoned from Rural Lifestyle Zone to General Residential Zone through Private Plan Change 4 (PC4), which became operative on 1 September 2026, and introduced the Ratanui Development Area provisions. PC4 provides for residential development of the Site, including retirement village use.

It is proposed to develop the Site into a retirement village. Part of 65 Ratanui Road is proposed to be retained by the Vendor through a boundary adjustment subdivision to maintain the existing residence.

This report describes the existing stormwater and flood characteristics of the Site, identifies the key regulatory and physical constraints to development, and sets out the proposed stormwater management strategy. The assessment also considers the potential effects of the development on downstream receiving environments and flood risk within and beyond the Site.

The intention of this report is to provide guidance for development with respect to stormwater management to mitigate the effects on the downstream receiving environments.

# 2 Site, catchment and proposed development

## 2.1 Site and Catchment Description

The site of the proposed Otaihanga Retirement Village is shown in Figure 1. It is bounded by residential properties to the west, farmland to the north and east, and Ratanui Road to the south. A preschool is located opposite the site, on the southern side of Ratanui Road.

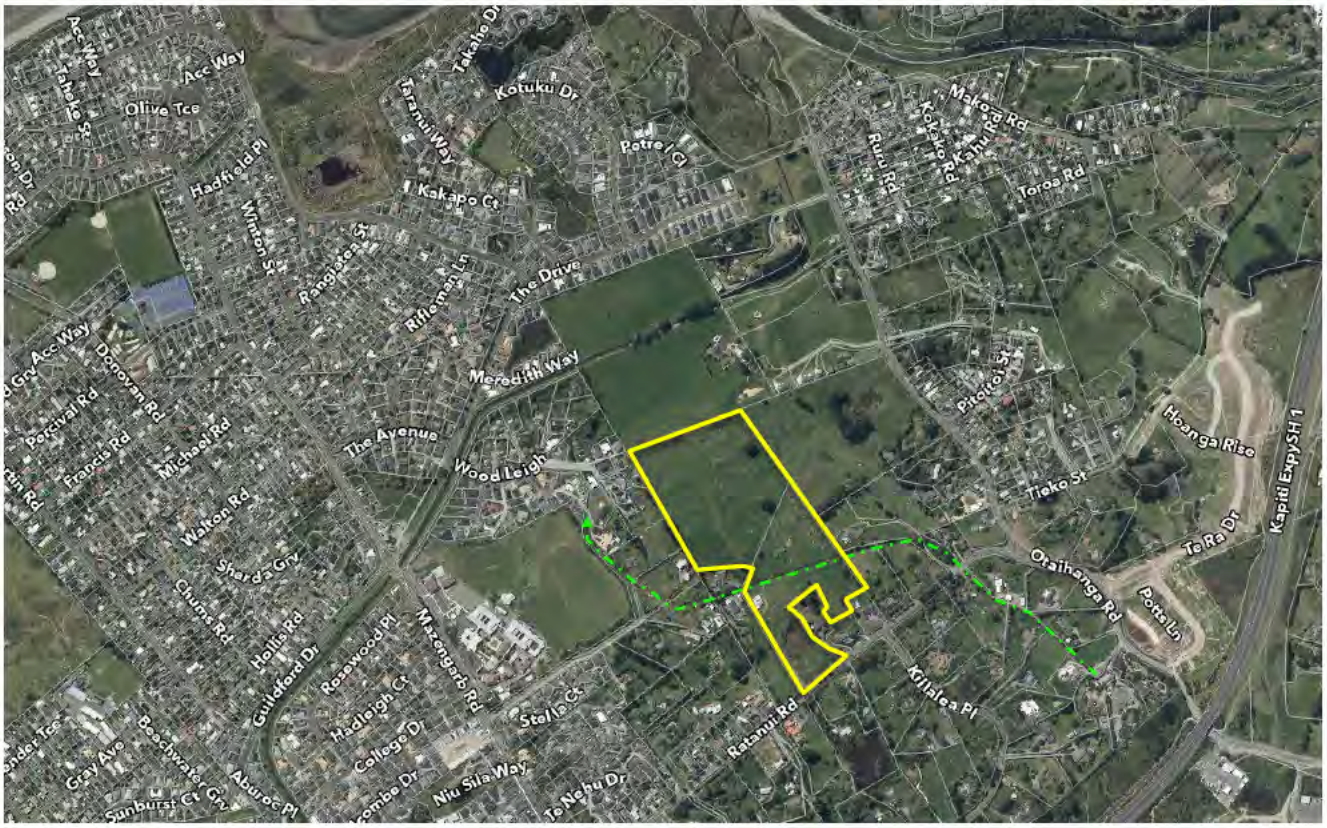



*Figure 1: Otaihanga Retirement Village Site*

The site occupies undulating dune terrain typical of the Kāpiti Coast, with small mounds and depressions. It is extensively fenced and currently used for small-scale pastoral farming. Two larger slopes along the northern boundary have areas of blackberry and boxthorn, while small clusters of trees occur elsewhere on the site.

An unnamed Drain bisects the site from east to west and conveys flows from the upstream catchment. Kāpiti Coast District Council (KCDC) maintains the channel as part of its stormwater network. It appears to be a constructed land drain, with a deeply incised bed and steep banks. Greater Wellington Regional Council (GWRC) classifies it as a highly modified stream (Figure 2).



*Figure 2: Photos of highly modified Drain that bisects Site*

A larger constructed pond is located south of the drainage channel. Several small, low-value wetlands with exotic vegetation have also been identified in depressions across the site. The ecological assessment prepared by Blue Green Ecology is provided in Appendix 8 of AEE Fast Track submission.

## 2.2 Soil conditions and groundwater monitoring

An initial geotechnical and contamination report, prepared by Riley Consultants, indicates that the Site is generally underlain by 0.15m to 0.50m (150mm – 500mm) of topsoil overlying a fine sand layer with varying quantities of silt. An updated geotechnical report was prepared for the proposed development by Engeo in September 2026. Both reports are included in Appendix 21 of AEE.

Original groundwater monitoring recorded at depths of 2.1m to 8m below ground level, with groundwater monitoring undertaken during Feb 2023 and May 2024. Additional testing was undertaken on 8th Sept 2026 by Engeo to capture levels in various times of the year. Groundwater was found to be higher during the winter conditions and after long periods of rain.

Groundwater was encountered at 0.6m to 2.2m at the specified testing location which was at key locations for proposed stormwater infrastructure in lower lying areas. These tests can be found in 21 of AEE.

Arsenic was detected above the high-density soil contaminant standard and the ecological soil guideline value in one soil sample near shed at 73 Ratanui Road from the ENGEO 2026 investigation[1], collected in an area where evidence of storage of treated timber was observed. Other samples indicated that the elevated arsenic was limited to shallow soils. ENGEO considers that due to the limited extent and depth of impacts detected, soil in the vicinity of the treated timber near the western site boundary is highly unlikely to present an unacceptable risk to human health or environmental receptors. Zinc concentrations were reported in exceedance of Eco-SGVs near historical vehicle storage area. The concentration detected does not represent a significant exceedance to ecological receptors.

[1] Detailed Site Investigation, 65 &73 Ratanui Road, Paraparaumu, Engeo, 29/09/2026

### 2.2.1 Infiltration Testing

Infiltration testing has been carried out by Leith Consulting across the northern section of the site in March and April 2023, and for the southern section in May 2024. Results from tests in 2023 present a significantly lower infiltration rate than the more recent testing in 2024, likely due to the groundwater levels being significantly higher during the 2023 testing. The results from the 2023 testing varies from 28-323mm/hr across the site.

Further testing at key locations of any proposed infiltration devices was taken on 8th September 2026. These tests showed high design infiltration rates of 185 mm/hr to 1168mm/hr (using both falling head and constant head test).

Results of all soakage testing is attached in Appendix 21 of AEE.

## 2.3 Proposed Development

The Proposed Development includes 257 Independent Living Units (ILU) and 90 care and assisted living units within a central main building and amenities located throughout the Site. Figure 3 below shows the proposed Master Plan for the development.




*Figure 3: Proposed Master Plan (Summerset Drawing #AR010, 15th Oct 2026)*

Given the undulating nature of the site, bulk earthworks are required to form a suitable finished surface. It is proposed to gently grade the site to fall towards the south for stormwater management prior to discharging to the Drain.

There is one proposed vehicular entry from Ratanui Road to the Site with internal roading throughout the Site. The proposed internal roading network will require two road crossings over the Drain.

# 3 Stormwater requirements and considerations

The comprehensive stormwater management strategy for the Site has assessed the regulatory inputs from both district and regional councils and nationwide policy whilst considering other key considerations.

## 3.1 Kāpiti Coast District Council

The following defines KCDC's Stormwater strategies for Low Impact Urban Design and Development (LIUDD).

- **Reduce Runoff Volume to the Maximum Extent Practicable** through maximising pervious area and through soakage or volume reduction devices
- **Provide Flood Control Detention** to control the increase in runoff rates from developed areas during frequent storm events that cause stream degradation and overflow that may cause flooding to properties
- **Water Quality Control**
- **Hydraulic Neutrality** where post-development peak discharges from a site to be restricted to the pre-development peak discharges, unless it can be shown that there is a minimal increase in discharges which have adverse effects that are no more than minor. Hydraulic neutrality has to be achieved by compensating for loss of flood storage associated with the development, and/or by managing the difference between the pre-development and post-development peak flows

KCDC's online GIS identifies significant areas of potential flooding across the Site, particularly adjacent to the existing Drain which bisects the Site. More recent catchment-wide flood modelling has been undertaken by Awa Environmental Ltd (Awa) on behalf of KCDC. CKL has obtained relevant outputs from the Awa model and used these to inform and calibrate the Site-specific flood model developed for the proposed development. The Awa model predicts less extensive flooding across the Site than the current publicly available KCDC GIS mapping; however, it identifies two overland flow paths entering the Site across the south-western boundary and discharging to existing low-lying depressions that currently provide flood storage and infiltration.

## 3.2 Greater Wellington Regional Council

In October 2023, GWRC notified Plan Change 1 (PC1) which proposes amendments to the Natural Resources Plan. In catchments defined as contributing to Te Whanganui-a-Tara and Te Awarua-o-Porirua, most new or redeveloped impervious areas are now subject to new stormwater rules under PC1, and some will include requirements for hydrological control where there is a discharge to a river. While this Site is not within the PC1 area, the PC1 guidelines are used as the standard approach to water sensitive design in the greater Wellington region and are in line with the KCDC guidance mentioned above. As such, they have been applied in the design.

For a development with over 3,000m$^2$ of new/redeveloped impervious area, such as this Site, the following rules apply:

- Stormwater Impact Assessment
- Water Quality Treatment
- Hydrological Control
    - Detention of 1/3$^{rd}$ of 2yr ARI runoff volume
    - Retention of 15mm rainfall on connected impervious surfaces
- Hydraulic neutrality/Flood Control

- WSUD (Water Sensitive Urban Design)

## 3.3 Stormwater Management Key Considerations

### 3.3.1 Water Quality treatment

Stormwater runoff from the development will discharge to the intermittent Drain, then to Mazengarb Stream and ultimately the Waikanae River before the ocean environment. Runoff must therefore be treated before discharge. Residential stormwater can contain significant contaminants, either dissolved or attached to fine clay or silt particles. These contaminants generally include:

- Sediments
- Metals
- Total petroleum hydrocarbons (TPH)
- Nutrients

Removal of these contaminants must be achieved to the highest practicable level prior to discharging into the sensitive downstream environments.

### 3.3.2 Hydrological Controls

As the Site ultimately discharges to a stream receiving environment, hydrological controls are required to manage the change in runoff volume, frequency and peak discharge resulting from development. The proposed stormwater system will therefore incorporate retention, extended detention and peak-flow attenuation measures.

In accordance with GWRC's PC1 hydrological control requirements adopted for the assessment, runoff associated with 1/3$^{rd}$ of the 2-year ARI event will be detained and released over 24hrs following the storm. For the Site, this corresponds to approximately 23.3 mm of rainfall, including the adopted climate change allowance (RCP 6.0).

Retention of the first 15mm of rainfall (inline with PC1 rules and KCDC rules) from connected impervious areas should be retained on site either through infiltration devices or non-potable water reuse.

Under KCDC's stormwater strategy, hydraulic neutrality is required (post-development peak discharges from a site to be restricted to the pre-development peak discharges) at the downstream boundary from site (the exit from the Drain along the south-western boundary) unless it can be shown that there is a minimal increase in discharges which have adverse effects that are no more than minor. The accompanying Flood Risk Assessment (Section 6.2 and Appendix 3 of FRA) assesses whether the proposed development results in any material increase in flood risk to neighbouring or downstream properties.

## 3.4 Ecology

An Ecological Assessment has been undertaken by BlueGreen Ecology. The assessment concludes that the Site contains no significant indigenous terrestrial ecological values and no threatened or "At Risk" species were recorded. The natural inland wetland features identified across the Site are small, highly modified, predominantly exotic-dominated dune hollows and are assessed as having negligible ecological value. The highly modified Drain that crosses the Site is also assessed as having negligible to very low aquatic ecological value, although occasional shortfin eel may use the channel during periods of winter flow.

Approximately 1,500 m² of natural wetland features would be affected by the proposed development. As part of the Private Plan Change (PC4), the small, low-value wetlands across the Site were proposed to be offset at a 1:1 ratio by creating larger wetlands adjacent to the Drain. The proposed wetlands would

provide higher-value wetland habitat, improve hydraulic stability, and enhance habitat and connectivity along the Drain. The Ecological Assessment considers that the offset wetlands on site will provide a positive ecological outcome and a net gain in wetland extent and quality, together with riparian enhancement along the Drain.

An important design consideration has been the separation of stormwater treatment and ecological wetland functions. In accordance with District Plan Policy DEV3-P1.5(c), the created ecological wetlands are not proposed to form part of the stormwater treatment train. Stormwater will instead be treated prior to entering the created wetlands, allowing the wetlands to function primarily as ecological features while also benefiting from treated inflows. Any maintenance is limited to entry and exit points and not included in the offsetting area values.

The Ecological Assessment is provided in Appendix 8 of AEE.

# 4 Existing drainage and flood conditions

## 4.1 Drainage system

Stormwater runoff under existing conditions is primarily managed through infiltration within the numerous low-lying depressions across the undulating Site, with some surface runoff discharging directly to the Drain that bisects the Site. Due to the Site topography, a significant proportion of runoff does not flow directly to the Drain but instead collects within local depressions and infiltrates to ground. This infiltrated water is likely to provide a slower subsurface hydraulic connection to the Drain via groundwater. As the Site is currently undeveloped farmland, there is no formal piped stormwater network.

The downstream receiving drainage network is the unnamed Drain, which ultimately discharges to Mazengarb Stream through several downstream culverts and eventually to the Kāpiti Coast.

## 4.2 Flood modelling and Existing site scenario outcomes

Two-dimensional HEC-RAS flood modelling has been used to establish existing flood conditions, utilising the Awa rain-on-grid model, which extends upstream to State Highway 1 (SH1).

CKL developed a separate, site-specific model using boundary conditions and hydrographs derived from the Awa model. The upstream catchment flows are represented by inflow hydrographs, while a stage hydrograph at Mazengarb Stream represents the downstream boundary and tidal influence. The CKL model was refined until flood behaviour and depths were broadly comparable within the area of interest. It was then used to assess the proposed development for the 100-year ARI event, including climate change, and to inform the stormwater management strategy.

The existing scenario flood model identifies flooding within the Drain that bisects the site and within the scattered depression areas across the site, with flood depths ranging from 0.5m to 1.0m. Much of the site discharges to these depression areas and to ground, with smaller catchments that discharge overflow to the Drain.

Two larger depression areas were identified along the western boundary that accepts flow from neighbouring sites to the west and act as flood storage and provide infiltration to ground. Figure 4 below shows existing flooding on site in 100yr ARI event.

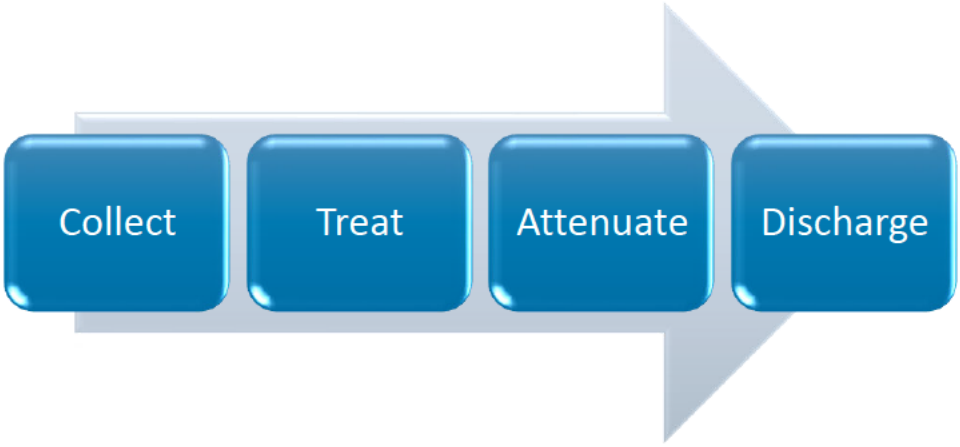
This section outlines the flood model results only; the full Flood Risk Assessment, including the flood model summary, is included in Appendix 4.




*Figure 4: Flood Extents Under Existing Conditions_100yrCC*

# 5 Stormwater Management Strategy

Based on the stormwater management key considerations and the flood assessment outlined above, the proposed stormwater management strategy adopts a hierarchy of stormwater management comprising the following:




Each of the stormwater management stages is described further below:

*Collect and Convey*

Collection of runoff includes downpipes from roof areas, surface flow of paved areas such as driveways and roads and associated pervious areas.

Conveyance of runoff within the road corridor will be via a piped network for primary flows or overland within the road corridor for secondary flows.

Due to the variability in measured infiltration rates and groundwater conditions across the Site, infiltration is not proposed as the primary means of disposing of flows. Infiltration will instead be incorporated as part of the required retention strategy.

*Treat and Retain*

Runoff is treated to reduce contaminants entrained the receiving environment. Treatment can be applied either at source or in communal devices at the base of the road, outside the floodplain.

The volume of runoff generated by a 15 mm rainfall event should be retained through either infiltration to ground or reuse.

Individual rainwater reuse tanks are not considered practical for the proposed retirement village due to the significant infrastructure and ongoing maintenance requirements. The villages are designed to be low maintenance for residents, with landscaping and irrigation managed through a site-wide irrigation system. Individual rainwater tanks therefore provide limited practical reuse opportunities. In summer, when irrigation demand is highest, the tanks are likely to be depleted with little rainfall, while in winter, when rainfall is greatest, there is little or no irrigation demand.

Connecting individual tanks into the wider irrigation network also requires a relatively complex system of linked tanks, pumps and pressure controls. A secondary town-supply connection would be required to maintain irrigation supply when tank levels are low, otherwise the system would require ongoing monitoring and manual intervention. This adds significant capital and operational complexity for relatively limited beneficial reuse of the captured water, increasing the complexity and lifecycle costs of the plumbing system.

While rainwater reuse may provide benefits in reducing potable water demand, the relatively limited reuse opportunities within individual units in retirement villages need to be balanced against the large capital cost and ongoing infrastructure and maintenance requirements for individual tanks. From a stormwater management perspective, the required hydrological controls can be achieved more reliably through centralised systems such as bioretention and wetlands, without reliance on individual household water consumption and space for tanks on each lot. Rainwater reuse may be more practical for larger communal buildings, such as the main building, where sufficient and consistent non-potable demand can be established; this can be considered at detailed design stage. An on-site bore for irrigation demand and outdoor taps, or a non-potable supply to the main building, could also be investigated further and considered to reduce demand on the Council reticulated system.

*Hydrological controls*

After treatment and retention are achieved, runoff from one-third of the 2-year ARI event will be detained, and runoff from larger rainfall events attenuated, prior to discharging to the Drain to achieve hydraulic neutrality, mitigate flood risk downstream and protect the downstream environment from erosion, as assessed in the Flood Risk Assessment.

*Discharge*

Discharge will be to the Drain. An assessment of effects at the downstream boundary (along the western edge of the Site) has been undertaken to confirm there is no increase in flooding or erosion due to the proposed Site works.

## 5.1 Stormwater Management Best Practicable Options Assessment

The implementation of the Stormwater strategy for the Site requires consideration as to the options for management of the runoff in the final build form. A Best Practicable Option (BPO) assessment has been undertaken to determine the most appropriate stormwater management approach for the proposed development. The assessment considers the Site characteristics, proposed retirement village land use, receiving environment, groundwater conditions, flood constraints, and the whole-of-life cost.

The following table summarises the principal stormwater management options considered.

*Table 1: Best Practicable Options Assessment*

| Device | Opportunities | Limitation | Recommendation |
|---|---|---|---|
| Living Roof | -Achieves detention and retention for residential buildings<br>-Amenity<br>-Noise insulation and enhanced air quality and dust interception.<br>-Reduce the impervious area | -It is not possible to direct the runoff from roads and car parks to this device<br>-Higher construction and maintenance costs<br>-Generally, not suitable for volume control<br>- Not suitable for water quality<br>-Where water quality is required, it needs to be | **NOT** recommended for this site due to practicality and intensive operation and maintenance requirements |

| Device | Opportunities | Limitation | Recommendation |
|---|---|---|---|
| | | incorporated with another SW device | |
| Pervious Paving | - Active system could provide detention and retention.<br>- Reduce effective impervious area<br>-Providing amenity | - Noisy to drive on<br>- Higher maintenance cost and if pervious paving fails, the surface will be considered out of compliance<br>-Generally, not suitable for volume control<br>- Less suitable for higher-use traffic areas and turning movements.<br>-Doesn't provide water quality so another device may be required<br>-Seasonal groundwater may limit infiltration performance | **NOT recommended** for roading, as it is not considered a good device for whole-of-life costs. |
| Rainwater Tank/Reuse | - Able to provide detention and retention<br>- Reduce the use of potable water from the public water supply system for non-potable uses (such as garden irrigation)<br>- Particularly suitable for larger roof areas | - Does not provide water-quality treatment for trafficked surfaces<br>- Limited reuse demand: Individual retirement units are unlikely to have significant outdoor irrigation demand, particularly where landscaping and irrigation are managed communally. Rainwater tanks only provide stormwater retention when sufficient storage capacity is available before a rainfall event. Effective reuse may therefore require additional internal plumbing for toilet flushing.<br>- Individual tanks and plumbing for greywater for every dwelling would result | **NOT recommended** as the primary Site-wide retention solution due to the low reuse in retirement villages, on-site non-resident garden/landscape management and the high maintenance requirements associated with individual tanks across the retirement village. |

| Device | Opportunities | Limitation | Recommendation |
|---|---|---|---|
| | | in a large number of assets requiring operation and maintenance and are hard on pipes and appliances (whole of life cost for infrastructure is high)<br>- Large tanks would be required on each lot, with limited space available, to also provide attenuation of roofs; centralised wetlands are already required for offsetting and can provide hydrological control | |
| Bioretention Devices/Raingardens | -Provides a full suite of stormwater management with detention, retention and water quality treatment<br>- Removes sediment, metals and other contaminants from road and paved-area runoff<br>-provide enhanced amenity, safety and aesthetic value through planting and educational opportunities | -Requires specific construction and ongoing maintenance<br>- Performance is affected by seasonal groundwater levels | **IS recommended** as the primary water-quality treatment and retention device.<br>Centralised devices are preferred over numerous smaller devices to reduce whole of life cost by reducing maintenance requirements, create larger habitat for vegetation health and biodiversity, create amenity.<br>Underdrains are proposed where seasonal groundwater may affect device performance. |
| Swales | -Can provide conveyance and some water-quality treatment<br>-Simple, naturalised stormwater solution | -Requires significant corridor width<br>-Need to be incorporated with another SW device to achieve detention and attenuation<br>-Culverts required for driveways into each lot | **NOT recommended** as the primary stormwater treatment or conveyance system within the developed areas due to safety concerns and road layout constraints. |

| Device | Opportunities | Limitation | Recommendation |
|---|---|---|---|
| | | (crossing swale) creating more infrastructure and reducing the length of treatment<br>- Open channels can create accessibility and safety issues within a retirement village | |
| Infiltration/soakage Devices | -Able to provide the retention<br>- Helps mimic the existing Site hydrology and recharge groundwater | - Infiltration rates vary considerably across the Site, with some levels below reasonable rates for primary disposal (less than 150mm/hr)<br>- Seasonal high groundwater occurs in lower-lying areas and these devices need considerable separation to high groundwater<br>- Does not provide sufficient water-quality treatment or attenuation on its own, so additional attenuation device may still be required<br>- Performance can reduce over time due to clogging<br>- High maintenance cost to manage individual lot soakage devices | **NOT recommended** as the Site-wide primary stormwater disposal method due to variable infiltration and groundwater conditions.<br><br>**IS recommended** as part of the bioretention system where ground and groundwater conditions permit. |
| Detention Ponds | - Provides detention and attenuation of larger storm events<br>- Relatively simple hydraulic control<br>- Can provide substantial storage volume (if hydraulic connectivity is available) | - Requires a large land area<br>- No water quality treatment<br>- Permanent or deep open water can create safety and amenity concerns<br>-Provides no ecological benefit compared to a constructed wetland<br>Lining maybe required to avoid interaction with Ground water table | **NOT preferred** over constructed wetlands. Wetlands can provide the required attenuation while also providing ecological habitat and contributing to the proposed wetland offsetting. |

| Device | Opportunities | Limitation | Recommendation |
|---|---|---|---|
| Wetland and Ponds | -Provides water quality, extended detention and attenuation of larger events<br>-Added amenity value for local communities<br>-Provides ecological habitat and biodiversity benefits and contribute to the proposed ecological offsetting required | - Requires a relatively large area<br>-Does not provide retention<br>-Safety, e.g. water hazard<br>- Outlet structures and forebays require ongoing maintenance<br>Lining maybe required to avoid interaction with Ground water table | **IS recommended** as the primary extended-detention and attenuation device, as well as offsetting habitat.<br>Wetlands provide multiple functions including attenuation, ecological enhancement and habitat creation, and are considered preferable to conventional detention ponds or underground storage. |
| Proprietary Treatment Devices | -Small footprint or can be under roads<br>-Can provide targeted removal of sediment and contaminants<br>-Useful where conventional/naturalized devices cannot be accommodated above ground<br>Often high cost option | -No retention and attenuation benefit<br>-Does not provide ecological or amenity benefits<br>-Requires proprietary maintenance and replacement of components<br>-Additional devices would still be required to meet hydrological-control requirements<br>-Can provide targeted removal of sediment and contaminants | **NOT recommended** as the primary Site-wide treatment solution.<br>May be considered locally at detailed design where space constraints prevent use of bioretention. |

### 5.1.1 Conveyance options

The conveyance options for the development area are essentially either (or a combination) of the following:

- Piped Networks
- Swales
- Overland flow/conveyance channels

Based on the nature of the proposed development being a retirement village, swales were **not** considered the best practicable option for safety/accessibility and due to the number of vehicle crossings that would

be required over the swales. Therefore, the best practicable option for conveyance of primary flows is a piped network to centralised devices and the Drain.

Secondary flow will be contained within the carriage way discharging to the centralised devices for attenuation prior to discharge to the Drian.

There are several overland conveyance channels/swales proposed along the boundary to convey overland flows from neighbouring sites and discharges to the Drain or the proposed flood storage areas along the western boundary.

### 5.1.2 Treatment

Bioretention devices are considered the most effective option for treating road runoff and providing retention as part of the Site's stormwater management system. Centralised devices are proposed upstream of the flood-attenuation devices. Compared with smaller, localised devices, they are less prone to drying out, supporting healthier, longer-lived plants and providing greater opportunities for biodiversity and habitat enhancement. Consolidated maintenance is also expected to reduce whole-of-life costs.

Inert roofing and cladding materials are also proposed to minimise contaminants at source. Together, the following measures are considered the Best Practicable Option (BPO) for managing stormwater quality at the Site:

- Raingardens/bioretention
- Inert roof and cladding materials

### 5.1.3 Hydrological Control

The following hydrological controls are required:

- Retention of 15mm rainfall
- Detention of 1/3$^{rd}$ of 2yr ARI event (23.3 mm)
- Flood mitigation and/or hydraulic neutrality at downstream boundary

As shown above in Section 5.1.2, the proposed bioretention devices will provide water quality treatment and contribute to the required 15 mm runoff retention through infiltration to ground. Surface ponding within the bioretention devices will also provide some temporary detention; however, the devices are not relied upon to provide the full extended-detention for the development (excluding that the retention volume calculated would cover the required detention volume as well) nor would they provide attenuation for larger devices.

Wetlands are therefore proposed to provide additional storage. These wetlands will provide the required extended detention and attenuation of larger storm events while also contributing to the ecological mitigation proposed for the Site. This combined approach enables the bioretention and wetland systems to perform complementary functions within the overall stormwater strategy.

### 5.1.4 Attenuation/Flood Risk Management

A localised rain-on-grid flood model incorporating an upstream catchment hydrograph was developed for the proposed development. The modelling assessed whether the development would increase flood risk in surrounding areas, achieve hydraulic neutrality during the 100-year ARI event including climate change allowances, and provide safe access and appropriate freeboard throughout the Site. The results indicated that large attenuation devices were required. Wetlands were therefore selected as the preferred management device because they can provide naturalised attenuation while also creating habitat.

The assessment of the Best Practicable Option (BPO) for flood management was informed by a Site-specific rain-on-grid flood model incorporating an upstream catchment inflow hydrograph. The modelling identified the need for substantial attenuation storage to achieve hydraulic neutrality during the 100-year ARI event, manage flood effects on surrounding properties, and provide safe access and appropriate freeboard throughout the Site. Constructed wetlands were selected as the preferred option because they provide the required storage in a naturalised form while also creating habitat.

# 6 Proposed Stormwater Management System

The proposed stormwater management system includes components which are described in the following subsections. These components operate as an integrated system rather than as individual management devices.

## 6.1 Conveyance

The primary stormwater network has been sized in accordance with regulatory requirements for the 10yr ARI event including climate change. The pipe is generally laid at 0.5% given the low grade of the site and outlet to the Drain. Therefore, relatively large pipes are required.

The pipe capacity calculation can be found in Appendix 3.

## 6.2 Treatment and Attenuation

### 6.2.1 Bioretention

For preliminary design, the centralised bioretention devices have been sized to provide treatment and retention (15mm rainfall) for the entire contributing catchment. In the absence of GWRC or KCDC guidelines for bioretention/raingarden sizing, the bioretention devices have been sized in accordance with Wellington Water's Water Sensitive Design (WSD) for Stormwater Guideline[2] to be 2% of untreated contributing catchment or larger to ensure the water quality flow rate (10mm/hr) from untreated runoff can drain through the bioretention media. Retention via infiltration to ground is provided for all impervious areas (designed in accordance with Wellington Water's WSD and Auckland GD01[3]). Table 2 below demonstrates the contributing catchment and bioretention device sizing. Appendix 3 includes calculations for bioretention devices.

*Table 2: Bioretention device Catchment Areas and Sizing*

| Catchment | South Bioretention device | North Bioretention device |
|---|---|---|
| **Contributing Catchment Impervious Area ($m^2$)** | 25,013 | 36,984 |
| **2% of Catchment Impervious Area ($m^2$)** | 500 | 740 |
| **Bioretention device Area ($m^2$)** | 510 | 1150 |

As seen above, in Table 2, the bioretention devices designed are larger than the required 2% of contributing impervious areas and have been designed to infiltrate the full retention volume to ground.

[2] Water Sensitive Design for Stormwater: Treatment Device Design Guideline, Wellington City Council, December 2019
[3] Stormwater Management Devices in Auckland Region, GD01, Auckland Council, Dec 2017

The native soil infiltration rate is relatively high, even assuming a 50% reduction rate in infiltration due clogging over time, the proposed bioretention devices are calculated to provide well over the required retention volume over a 72-hour period for the entire contributing catchment's impervious areas. However, this will be limited by the available volume within the device itself. Refer to Appendix 3 for the associated calculations.

Further groundwater monitoring indicated that groundwater was encountered above the proposed media in the southern bioretention device. The groundwater level found on 8th September 2026 was at RL 4.6m and the proposed base of media is RL3.7m. Data loggers have been installed in several bores across the site and these have indicated that the levels on 8th September are much higher after extended period of winter rain, nearby datalogger recorded groundwater levels fluctuating between RL 3.0 and 3.25m. Additionally, previous groundwater levels in the area in 2023 test were found to be around RL 2.1m.

Given the height constraints of the site, raising the bioretention device and maintaining the 600mm of media depth is not possible. It is therefore considered that underdrains be installed within the drainage media below the bioretention media of the southern bioretention device. For the majority of the year it is assumed that groundwater will be below the underdrain level, and runoff from bioretention can freely drain to ground; however, when there is seasonably high groundwater, the groundwater can locally discharge through underdrains to the adjacent Drain to protect the bioretention media from being saturated for extended periods of time. It is expected this will only happen rarely given the groundwater monitoring done to date. This will be further detailed at detailed design stage and with further groundwater monitoring.

### 6.2.2 Wetlands

The wetlands are designed downstream of the bioretention devices to attenuate treated runoff. There is a forebay at the entry of each wetland to dissipate energy from flow entering the wetlands. The wetlands are designed to have a permanent water level at RL4.3m with undulating base providing deep and shallow marsh zones to naturalize it and establish healthy vegetation and habitat.

The wetlands will provide extended detention and flood storage/attenuation of larger events, as wetlands naturally do. An outlet structure is designed to attenuate flows to pre-development levels. This will be in a deeper pool along the wetland edge so regular maintenance can be provided to ensure it is clear of debris. The outlet scruffy domes details are provided below in Table 3 and in DWG#4501.

*Table 3: Wetland Outlet Details*

| Parameter | Northern Wetland Outlet | Southern Wetland Outlet |
|---|---|---|
| **EDV Orifice** | 225mmØ orifice at RL 4.3m | 167mmØ orifice at RL 4.3m |
| **Slot Weir** | 700mm wide slot at RL 4.5m | 135mm wide slot at RL 4.5m |
| **1200mm Ø Scruffy Dome Lid Level** | Lid level at RL 5.0m | Lid level at RL 5.0m |

The wetland outlet pipes will be fitted with stop valves to prevent backflow from the Drain. In the 100-year ARI HEC-RAS model, elevated water levels in the Drain close the stop valves and temporarily prevent the wetlands from discharging while the upstream peak passes. This interaction is not represented in the HEC-HMS model because, during smaller events, water levels in the Drain are expected to remain below the outlet-pipe and stop-valve levels. If elevated Drain levels were to coincide with a smaller event, runoff would be temporarily stored in the wetlands and released gradually after the upstream peak had passed. This would flatten the post-development peak rather than increase it.

There will be no maintenance (beyond the establishment period) within the main wetland body as it is an offset wetland and the habitat should not be damaged through maintenance practices. Maintenance will be localized to the forebays to clear any built-up sediment and the outlet scruffy dome structures to keep them clear of debris and blockages so they can function appropriately.

## 6.3 Culverts

There are two proposed road crossings over the Drain. It is proposed to have a 2mx2m box culvert to convey flows under the proposed road for each crossing. These culverts are designed to comply with the as Permitted Activity standards under Regulation 70 of National Environmental Standard for Freshwater (NES-F). This includes:

- Laid parallel to slope of bed of Drain
- 2m wide box culverts are 1.3 times the width of Drain base which is 1.5m wide (1.3x 1.5m is 1.95m)
- Culverts are embedded 25% and are native bed substrate backfilled
- The mean cross-sectional area is larger than the Drain cross-sectional area, therefore velocities will be no greater than upstream of downstream of culvert. Additionally, the low grade of the Drain and culverts, the velocities are low and not expected to transport sediments during majority of events
- Given the above, the culverts are designed to mimic the natural drain bed providing continuity of geomorphic processes and providing the same fish passage as upstream and downstream Drain sections

The culvert designs can be found in drawing #A25556-4502 in Appendix 2.

## 6.4 Flood Storage Areas

A flood-storage basin is proposed along the western boundary of the Site to receive overland flows from neighbouring properties and allow them to infiltrate to ground. The two principal overland flow paths will be directed to this single, larger basin. The basin slopes will be planted to reduce visual effects and protect against erosion, while fencing will restrict access.

Two culverts, located to the north and south of the basin, will collect flows from other depressions along the boundary and convey them to the basin. The culverts have been oversized and modelled with 50% blockage as a sensitivity check (discussed further in Flood Risk Assessment). They will be regularly inspected and cleared of debris to maintain their capacity.

Accumulated debris will also be removed from the basin floor to prevent its infiltration capacity from being reduced. Draft operation and maintenance plans for the culverts and flood-storage basin are provided in Appendix 5.

# 7 Flood Modelling and Development Effects

## 7.1 Proposed site development and flood modelling outcomes

The proposed earthworks will alter the existing runoff response of the Site by grading much of the developed area towards the Drain. Under existing conditions, a substantial proportion of runoff is captured within local depressions and infiltrates to ground rather than discharging more rapidly overland to the Drain. The proposed development, without mitigations therefore has the potential to increase and accelerate runoff reaching the Drain.

Attenuation is required to manage this change and achieve hydraulic neutrality at the downstream Site boundary. However, the effectiveness of attenuation also depends on the timing of Site runoff relative to the peak flow within the wider Drain catchment. The stormwater strategy has therefore been developed to avoid, as far as practicable, the attenuated Site discharge coinciding with the peak upstream catchment flow, as this will likely exacerbate flooding risks within the surrounding flood plains.

Initially, the proposed development was run in the HEC-RAS model using preliminary attenuation ponds and a flood storage area which was identified within the Private Plan Change PC4 application. This included two large attenuation ponds for each sub-catchments either side of the Drain and a floodplain area connected to the Drain for attenuating flows from upstream of the Site. The flood model results showed a significant decrease in peak flow in the post-development scenario and delay in the time to peak compared to the existing scenario, given attenuation of both the Site catchment and upstream flows. Therefore, a scenario was run to maintain the attenuation ponds for attenuating flow from the Site and remove the flood storage to allow the upstream flow to discharge though the Site prior to release of the attenuated Site flow.

Based on the initial modelling, the preferred strategy is to allow upstream catchment flows to pass through the Site while temporarily attenuating runoff generated by the development. Site runoff is directed to the northern and southern attenuation devices, where it is stored and released at a controlled rate to the Drain. The outlet from the devices is fitted with a stop valve so when the peak flow from upstream catchment arrives in the Drain, no flow from site discharges to the Drain until the flow in the Drain lowers and the peak has passed.

The flood modelling was run for the 100yr ARI rainfall events including the effects of climate change ($100yr_{CC}$ARI) to assess the proposed stormwater management system and the development's effects on the site and wider catchment. The extent of the flood plain and flow depth in the proposed scenario for $100yr_{CC}$ARI rainfall event is presented in Figure 5. Sensitivity has been run for 500yr ARI event to ensure the road crossing over the Drain can be safely passed in emergency vehicles, the 500yr flood levels don't overtop the road and have sufficient freeboard.

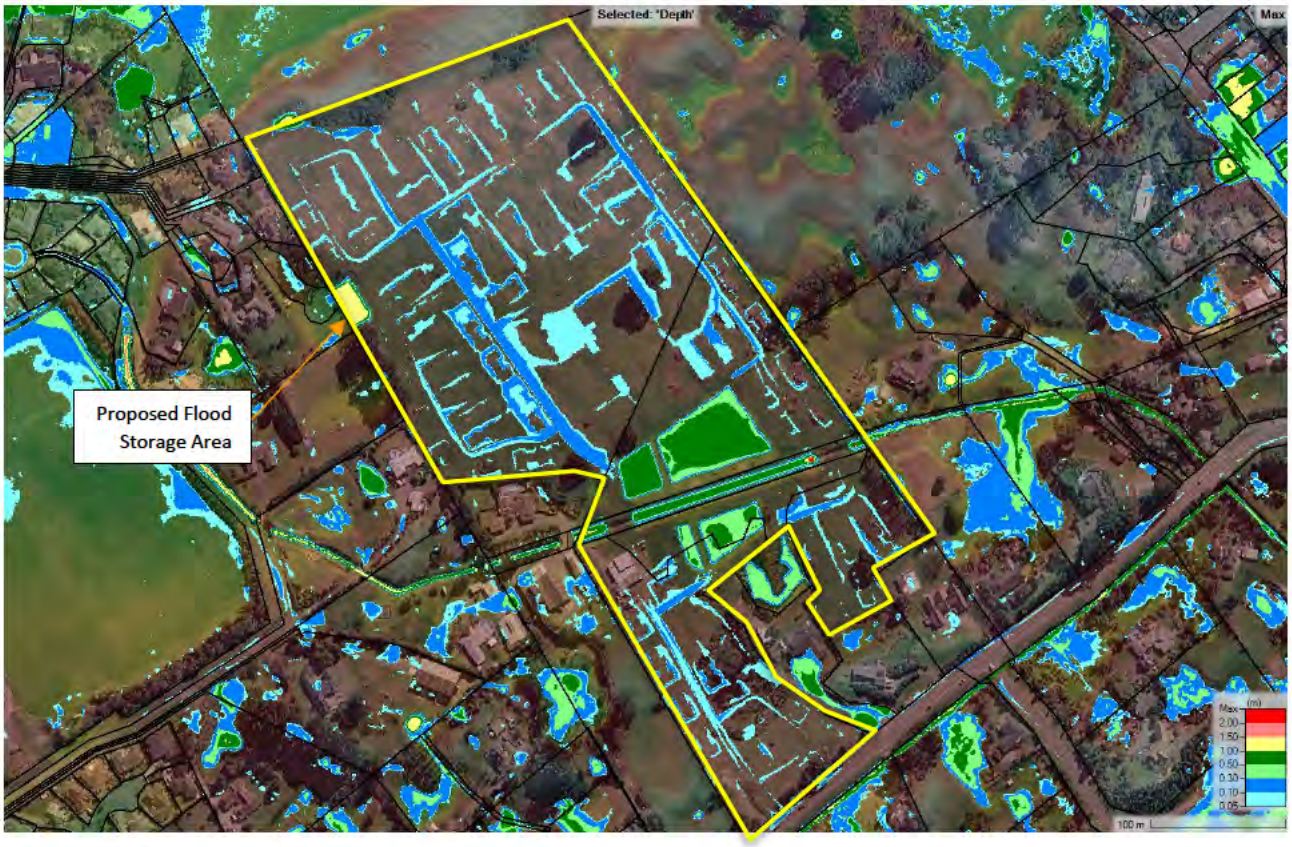



*Figure 5: Flood Extents Under Proposed Conditions_100yrCC*

Flood-level difference mapping was used to assess the effects of the proposed development on the existing floodplain. Initial modelling identified localised increases in the flood level near parts of the Site boundary. These increases were primarily associated with filling existing low-lying ponding areas to form the development platform, rather than increased stormwater discharge to neighbouring properties or obstruction of existing overland flow paths.

The proposed earthworks surface was subsequently refined to reduce these effects where practicable. Most notably, the two existing depressions along the western boundary were reshaped and consolidated into one larger flood storage area by the northern existing depression area to accommodate proposed development while maintaining flood-storage function. A smaller depression area is provided to accept the southern overland flow path and conveyed through a culvert to the large flood storage. A culvert is also provided to the north of the proposed flood storage to pick up smaller overland flows along the boundary. The resulting model predicts a reduction in flood levels within these areas and generally limits the flood-level increases elsewhere to less than 10 mm, except at the specific area mentioned below and further detailed in the Flood Risk Assessment in Appendix 4.

The flood level difference map is presented in Figure 6. Flood level increase or decrease of 10 mm or less are shown in white. Blue areas indicate a reduction in flood level, while yellow areas indicate an increase of greater than 10 mm, the legend in below figure demonstrates level of increase flood depth. The increases within the site will be in areas of depression areas in existing scenario and due to proposed fill.

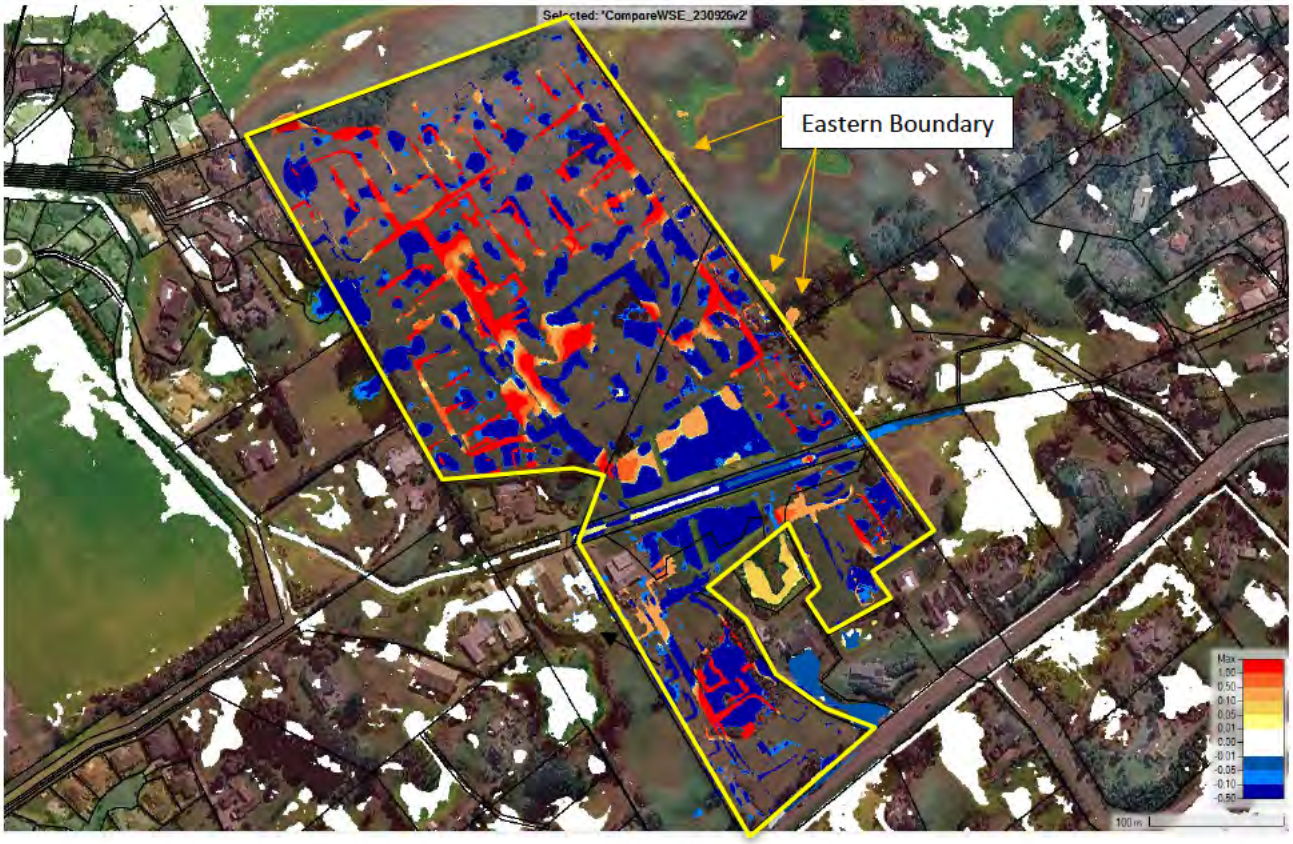



*Figure 6: Flood level difference map between proposed and existing scenario_100yrCC*

At several locations, the undulating terrain means that mitigation of small flood storage areas along the boundary would require impracticable engineering works, including extensive retaining structures, particularly along the eastern boundary. A separate memorandum has been prepared to document these constraints and assess the pre- and post-development flood risks at the affected locations. These locations are outlined on the flood-level difference map (Figure 6) and within the flood modelling memorandum, Appendix 4.

## 7.2 Hydraulic Neutrality

The purpose of hydraulic neutrality is to avoid increasing flood risk downstream of the Site. The flood model extends beyond the Site boundary and indicates that the proposed development will have negligible effect on flood levels outside the Site. Hydraulic neutrality is achieved through storage within the proposed northern and southern attenuation devices and controlled discharge to the Drain.

Figure 7, below, demonstrates the pre-and post-development peak flow downstream of the Site boundary within the Drain.

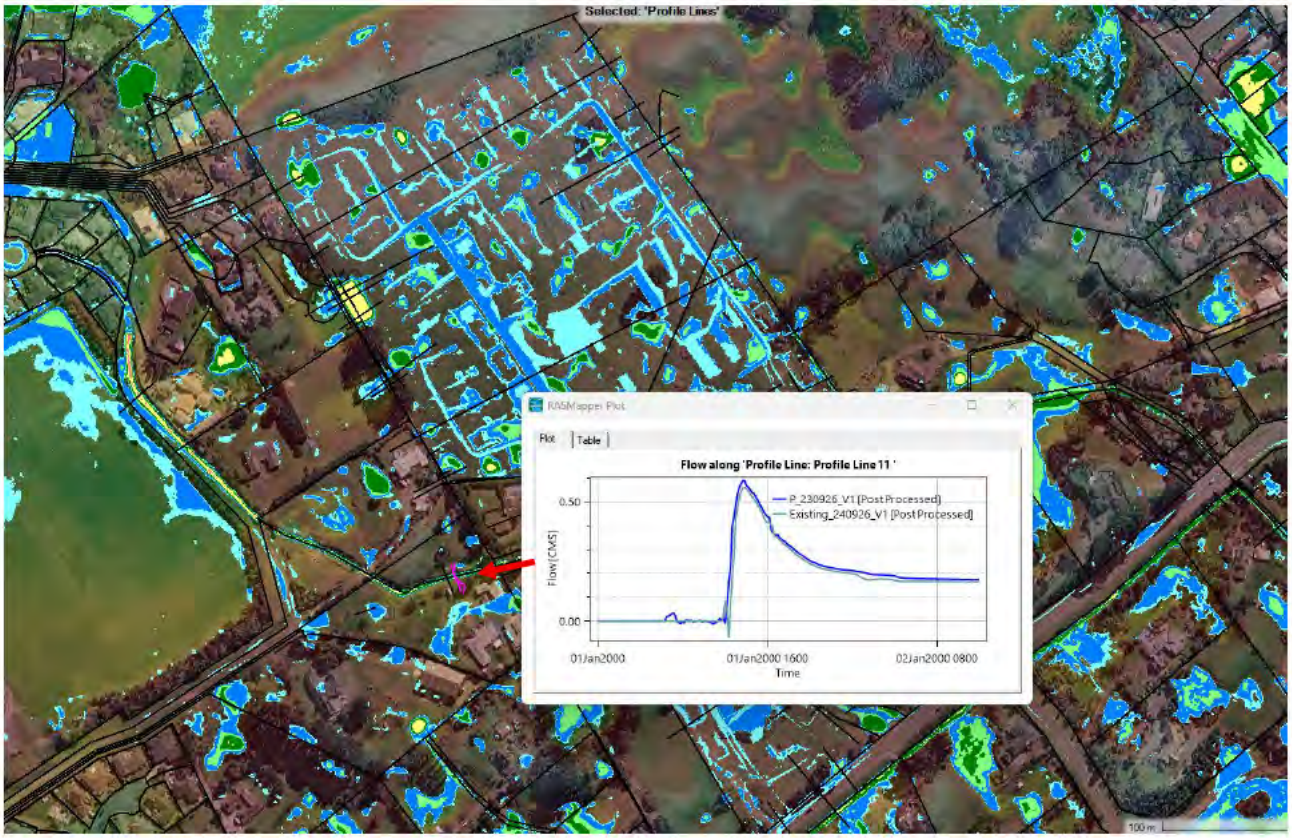



*Figure 7: Existing and proposed peak flows at downstream boundary_100yrCC*

The HEC-RAS model was used primarily to assess interaction between the proposed development, attenuation storage and flood flows within the Drain during the major design event (100yr$_{CC}$ ARI). This model includes outlet pipes from attenuation devices with stop valve but not the network upstream of wetlands as assumed to be blocked in 100yr$_{CC}$ event.

A complementary HEC-HMS model was developed to assess the performance of the attenuation devices across a wider range of rainfall events, including events where a proposed scruffy dome structure controls discharge due to Drain flow being lower than the outlet pipe. Together, the models are used to inform attenuation storage and outlet sizing and to demonstrate the required peak-flow control.

### 7.2.1 Attenuation device performance

The HEC-RAS model was done for 100yr ARI event, including climate change allowances, to assess flooding risk as well as hydraulic neutrality in this event considering the larger catchment dynamics, as outlined in the previous section. A HEC-HMS model was also utilised to assess the hydraulic neutrality in smaller rainfall events to protect the downstream environment from erosion and flooding in primary events.

Under existing conditions, only a relatively small proportion of the Site drains overland directly to the Drain. Most runoff collects in depressions, infiltrates to ground and reaches the Drain more slowly through groundwater. By contrast, most of the developed Site impervious areas will discharge direct to the Drain. To represent the slower existing runoff response, the HEC-HMS model treats the entire Site as pervious land and applies the SCS loss method with the Delmarva unit hydrograph as the transform method. This produces a lower, flatter peak and a longer hydrograph tail, reflecting the Site's undulating terrain and delayed hydraulic connection to the Drain. Table 3 presents the modelled pre- and post-development peak flows for each sub catchment. The HEC-HMS model is provided in Appendix 3.

*Table 4: HEC-HMS Peak flow analysis in smaller rainfall events*

| Rainfall Event (ARI) | Pre-Development Peak Flows ($m^3/s$) | Post-Development Peak Flows($m^3/s$) |
|---|---|---|
| **Northern Wetland** | | |
| **2yr** | 0.104 | 0.047 |
| **10yr** | 0.243 | 0.141 |
| **Southern Wetland** | | |
| **2yr** | 0.035 | 0.034 |
| **10yr** | 0.082 | 0.082 |

# 8 Staging, Detailed Design and Maintenance

## 8.1 Staging of development

The development is proposed to proceed in stages, beginning at the southern end of the Site near the proposed main access road and Site entrance. Stormwater management facilities serving each stage will be constructed before each stage is completed

## 8.2 Detailed Design Verification

The flood modelling undertaken for this assessment establishes the overall feasibility and performance of the proposed stormwater and flood-management strategy. Detailed-design verification will be required as final earthworks, road and stormwater designs are developed. The detailed-design model updates will principally comprise:

- final earthworks and finished-ground levels;
- final flood-storage geometry;
- final pipe and culvert sizes and invert levels;
- final wetland and infiltration-device storage;
- calibrated outlet-control dimensions;
- final road and secondary-flow-path geometry;
- stage-specific temporary and permanent drainage arrangements.

These updates are intended to refine the design demonstrated by this SMP rather than defer resolution of fundamental flood-management issues. The current modelling therefore demonstrates the required development-wide drainage strategy, principal flood corridors, downstream-effect outcomes and critical hydraulic constraints.

## 8.3 Operation and Maintenance

A draft operation and maintenance plan for the stormwater management system is provided in Appendix 5.

# 9 Summary

CKL has prepared this Stormwater Management Plan and Flood Risk Assessment to support the fast-track consent application for the proposed Otaihanga Retirement Village at 65 and 73 Ratanui Road, Paraparaumu. The assessment responds to the Site's key stormwater and flooding constraints, including its undulating dune topography, localised depressions that currently provide infiltration and flood storage, and the intermittent Drain that crosses the Site and ultimately discharges to Mazengarb Stream.

The proposed stormwater strategy comprises piped primary conveyance, centralised bioretention for water-quality treatment and runoff retention, and downstream wetlands for extended detention, peak-flow attenuation and ecological offsetting of the low-value wetlands affected by the development. The integrated system is designed to retain the first 15 mm of runoff from connected impervious surfaces, detain one-third of the two-year ARI runoff volume and achieve hydraulic neutrality at the downstream Site boundary.

Flood modelling was used to refine the proposed earthworks and stormwater strategy so that existing overland flow paths and flood-storage functions are maintained where practicable and effects on neighbouring and downstream properties are minimised. The modelling indicates that flood-level increases will generally be limited to less than 10 mm, except at the specific locations identified and assessed separately in the Flood Risk Assessment. Runoff from the developed Site will be stored within the northern and southern attenuation devices and discharged to the Drain at controlled rates. The modelling demonstrates that this arrangement achieves hydraulic neutrality at the downstream Site boundary and avoids any material increase in off-site flood risk.

# 10 Limitations

The report provides a technical basis for stormwater and flood risk management for resource consenting purposes. Detailed design, final device sizing, final earthworks and drain geometry, construction methodology, operation and maintenance requirements, detailed erosion protection design, consent conditions and any additional assessment required through further engagement with Kapiti Coast District Council, Greater Wellington Regional Council and mana whenua will be addressed during subsequent design and consenting stages.

This report has been prepared solely for the benefit of our client with respect to the particular brief and it may not be relied upon in other contexts for any other purpose without the express approval by CKL. Neither CKL nor any employee or sub-consultant accepts any responsibility with respect to its use, either in full or in part, by any other person or entity. This disclaimer shall apply notwithstanding that the report may be made available to other persons including to the Environmental Protection Authority for a fast-track consent application.

# Appendix 1 Statement of Experience

Statement of Experience for:

**Frances Deamer-Phillips**

**Qualifications and professional experience:**

My full name is Frances Deamer-Phillips. I am a Senior Environmental Engineer with CKL NZ Limited (CKL). I provide technical leadership in stormwater management, catchment planning, hydrology, flood-risk assessment and water-sensitive urban design.

I hold a Bachelor of Art (Hons) with focus in Environmental Architecture and Engineering from the Connecticut College in the United States and Masters of Engineering Studies (Hons) with speciality in Environmental Engineering from University of Auckland.

I am a Member of Engineering New Zealand (MEngNZ) and have completed an approved Knowledge Assessment for my overseas qualifications as part of my application for Chartered Professional Engineer (CPEng) registration. My recognised practice area is stormwater engineering.

I have more than 10 years' experience in civil and environmental engineering, including approximately 7 years specialising in the planning, design, consenting and implementation of stormwater management systems for public infrastructure and land development projects in New Zealand.

**Relevant technical experience**

My professional experience is primarily focused on stormwater management, including catchment hydrology, stormwater network design, hydraulic assessment and the integration of stormwater infrastructure into land development. My work involves assessing pre- and post-development runoff, stormwater conveyance and attenuation requirements, culvert and channel capacity, overland flow paths, downstream receiving environments and the effects of urbanisation and climate change on stormwater systems.

I have experience in hydrological and hydraulic modelling, particularly using HEC-HMS and EPA SWMM to assess catchment runoff, design stormwater networks, size detention and treatment systems, and evaluate the performance of stormwater infrastructure. I also undertake flood risk assessments and review hydraulic modelling results to inform stormwater design and assess potential development effects, working alongside specialist flood modellers where more detailed floodplain modelling is required.

A significant component of my work involves the design and integration of water-sensitive urban design and green infrastructure. This includes constructed wetlands, raingardens, biofiltration systems, swales, detention and retention systems, and open-channel conveyance. My approach is to develop practical stormwater management solutions that address water quality, hydrological mitigation, conveyance and flood management requirements while considering ecological, landscape, operational and maintenance outcomes.

In my role at CKL, I am responsible for developing stormwater management strategies and design solutions, undertaking and reviewing technical assessments, preparing stormwater management plans and supporting resource consent applications. I work closely with other engineering disciplines, planners, ecologists and landscape architects to ensure that stormwater management is integrated into the wider development design.

Recent comparable New Zealand assignments include the Southern Gateway development near Auckland Aiport, where I am the lead for the overarching stormwater strategy and design work for large scale light industrial development and integration of treatment train approach including raingardens and multi-stage wetlands; Ayrburn Hospitality, Retirement and Film Hub in Arrowtown, where I was lead stormwater management designer providing expertise for a catchment wide treatment train approach and sediment removal to improve water quality to Lake Hayes and integrate flood management along Mill Creek and the wider landscape design, to which CKL were recognised as leaders in their field of stormwater and environmental engineering by Engineering NZ.

**Role in the Otaihanga Retirement Fast-track referral**

I am the lead author for the Stormwater Management Plan prepared by CKL in support of the referral application for the Otaihanga Retirement Village, comprising the retirement village proposed in Paraparaumu. The working referral document identifies me as the author and Senior Environmental Engineer.

My role has included assessment of the existing catchment characteristics and drainage infrastructure, identifying stormwater, flooding, and receiving-environment constraints, developing the overarching stormwater management strategy, and assessing the feasibility and performance of the proposed stormwater management systems.

The stormwater strategy adopts an integrated catchment-based approach, combining conventional drainage infrastructure with green infrastructure to manage stormwater quantity, quality and conveyance. Key components include constructed wetlands, bioretention devices, stormwater attenuation, primary and secondary drainage networks and the integration of existing drainage corridors into the development layout.

My involvement has included consideration of pre- and post-development runoff, hydraulic neutrality, stormwater treatment and retention requirements, downstream discharge effects, and the integration of stormwater infrastructure with ecological and landscape objectives. I have also considered the practical operation and maintenance of these systems, including the suitability of different stormwater management options for a retirement village development.

The assessment has been undertaken at a level appropriate to the design and consenting stage, with the stormwater management approach supported by hydrological and hydraulic analysis and coordination with the wider project team. Where specialist flood modelling or ecological assessments have been required, I have reviewed and incorporated the relevant findings into the overall stormwater management strategy.

In preparing my assessment, I have reviewed and/or relied upon the following principal documents and information:

- Kāpiti Coast District Council, *Operative Kāpiti Coast District Plan 2021*, including the provisions introduced through Private Plan Change 4 (Welhom Developments Ltd) and the Otaihanga Development Area provisions.
- Kāpiti Coast District Council, *Land Development Minimum Requirements* (April 2022), particularly the requirements for stormwater design, hydraulic neutrality, drainage systems and climate change.
- Kāpiti Coast District Council, *Low Impact Urban Design and Development (LIUDD) Stormwater Guideline* (2012).
- Kāpiti Coast District Council, *Subdivision and Development Principles and Requirements – Isohyet Based Calculation of Design Peakflows* (2011).

- NZS 4404:2010, *Land Development and Subdivision Infrastructure*, including Section 4 – Stormwater, as amended by the Council's Land Development Minimum Requirements.
- Greater Wellington Regional Council, *Natural Resources Plan for the Wellington Region*, including relevant provisions for stormwater discharges and freshwater management.
- Relevant site-specific information, including topographical survey, existing drainage information, hydrological and hydraulic modelling (KCDC flood model for Paraparaumu prepared by AWA consultants), ecological assessments, geotechnical information and proposed development plans prepared by the wider project team.

The principal source documents currently identified in the referral strategy are recorded in its references and source-document schedule.

Where I have relied on information or specialist advice prepared by other members of the project team, including flood modelling, planning, ecology, geotechnical, transport, landscape or architectural information, that reliance is identified or implicit from the relevant source material. My opinions are confined to matters within my expertise in stormwater management.

**Independence and expert evidence convention**

Although this statement and the associated stormwater assessment have been prepared to support a referral application under the Fast-track Approvals Act 2024, rather than as evidence in an Environment Court proceeding, I have prepared my technical assessment and opinions on an independent professional basis.

I have read the Code of Conduct for Expert Witnesses contained in section 9 of the Environment Court Practice Note 2023 and agree to comply with that Code to the extent that it is applicable to my involvement in this project. I understand that an expert's primary obligation when providing expert opinion is to provide impartial assistance on matters within their expertise and not to act as an advocate for the party engaging them.

Except where I expressly identify reliance on another specialist, the matters addressed by me are within my area of expertise. I have identified the information, assumptions and limitations material to the opinions I have expressed, and I have not knowingly omitted to consider any material fact that might alter or detract from those opinions. Where information remains incomplete at referral stage, I have identified the further investigation or assessment required rather than assuming that issue to have been resolved.

I will also comply with any project-specific expert evidence convention or protocol agreed by the project legal team for the purposes of the Fast-track process.

**Frances Deamer-Phillips**
**BA (Hons), MengSt (Hons)**
Senior Environmental Engineer
CKL NZ Limited

# Appendix 2 Drawings

Refer to Appendix 17 of AEE for Civil Drawings

# Appendix 3 Calculations

| | |
|---|---|
| Job Name | Summerset Otaihanga Village, Paraparaumu |
| Job No. | A25556 |
| Date | 25/08/2026 |
| By | FDP Checked JC |

## Site Coverage Breakdown Post- Development Conditions

| | Pre-Development | |
|---|---|---|
| **Total** | **127025** | **100%** |
| Roof Area | 0 | 0% |
| Landscaping | 127025 | 100% |

| | Post development | | | |
|---|---|---|---|---|
| | **Area** | % | Net Change ($m^2$) | % change |
| **Total** | **127025** | **100%** | | |
| Total pervious | **65028** | 51% | | |
| Total Impervious | **61997** | 49% | | |
| North Total Area | 93738 | 74% | 93738 | 74% |
| North Pervious | 56754 | 61% | 56754 | 61% |
| North Impervious | 36984 | 39% | 36984 | 39% |
| North Roof/other imp | 22358 | 24% | 22358 | 24% |
| North Roading | 14626 | 16% | 14626 | 16% |
| South Total Area | 33287 | 26% | 33287 | 26% |
| South Pervious | 8274 | 25% | 8274 | 25% |
| South Impervious | 25013 | 75% | 25013 | 75% |
| South Roof/other imp | 18538 | 56% | 18538 | 56% |
| South Roading | 6475 | 19% | -120550 | -81% |

| | | | |
|---|---|---|---|
| Job Name | Summerset Otaihanga Village, Paraparaumu | | |
| Job No. | A25556 | | |
| Date | 25/08/2026 | | |
| By | FDP | Checked | JC |

# Northern Bioretention Sizing

**Catchment Areas**

| | Area (m2) | 25mm Detention Volume (m3) | 15mm Retention Volume (m3) |
|---|---|---|---|
| Northern Total Catchment Area | 93738 | | |
| Northern Impervious Area | 36984 | 924.60 | 554.76 |
| Northern Roading Area (Contaminant Generating) | 14626 | 365.65 | 219.39 |
| 2% of Contributing Impervious | 739.68 | | |

**Bioretention Volume Sizing**

**Bioretention Parameters**

| | | |
|---|---|---|
| Area of Bioretention Device | 1160 | m2 |
| Ponding Depth | 200 | mm |
| Media Depth | 600 | mm |
| Media Void Ratio | 30% | |
| Storage layer Depth | 200 | mm |
| Storage void ratio | 35% | |

**Detention Volume**

| | | |
|---|---|---|
| $V_{pond}$ | 232 | m3 |
| $V_{media}$ | 208.8 | m3 |
| **$V_{detention}$** | **440.8** | **m3** |

**Treatment flow**

| | | |
|---|---|---|
| Water Quality Rainfall Intensity | 10 | mm/hr |
| WQF of Roading | 0.03860 | m3/s |
| WQF of Roading | 138.95 | m3/hr |
| Infiltration rate of media | 250 | mm/hr |
| Area required | 1111.6 | m2 |

**Retention Volume**

| | | |
|---|---|---|
| Native Soil Infiltration Rate | 48 | mm/hr |
| Assumed Infiltration rate- 50% reduction for clogging | 24 | mm/hr |
| $K_{subsoil\ rate}$ | 0.024 | m/hr |
| $V_{storage}$ | 81.2 | m3 |
| $V_{infiltration\text{-}72hrs}$* | 2004.48 | m3 |
| $V_{infiltration\text{-}24hrs}$ | 668.16 | m3 |

| | | | |
|---|---|---|---|
| **$V_{retention}$ (volume in device)** | 522 | m3 | (calucated volume in ponding, media and storage volume) |
| **$V_{retention}$ (includng 24hr infiltrat** | 1190.16 | m3 | |

*Note that this is calcuated over 72 hr period, in reality it is limited by the volume above. Total retetnion provided can be caculated by the volume within bioretention device. Infiotlration over 24hr can also be assumed

| | | | |
|---|---|---|---|
| Job Name | Summerset Otaihanga Village, Paraparaumu | | |
| Job No. | A25556 | | |
| Date | 25/08/2026 | | |
| By | FDP | Checked | JC |

# Southern Bioretention Sizing

**Catchment Areas**

| | Area (m2) | 25mm Detention Volume (m3) | 15mm Retention Volume (m3) |
|---|---|---|---|
| Southern Total Catchment Area | 33287 | | |
| Southern Impervious Area | 25013 | 625.33 | 375.20 |
| Southern Roading Area (Contaminant Generating) | 6475 | 161.88 | 97.13 |
| 2% of Contributing Impervious Area | 500.26 | | |

**Bioretention Volume Sizing**

**Bioretention Parameters**

| | | |
|---|---|---|
| Area of Bioretention Device | 505 | m2 |
| Ponding Depth | 200 | mm |
| Media Depth | 600 | mm |
| Media Void Ratio | 30% | |
| Storage layer Depth | 200 | mm |
| Storage void ratio | 35% | |

**Detention Volume**

| | | |
|---|---|---|
| $V_{pond}$ | 101 | m3 |
| $V_{media}$ | 90.9 | m3 |
| **$V_{detention}$** | **191.9** | **m3** |

**Treatment flow**

| | | |
|---|---|---|
| Water Quality Rainfall Intensity | 10 | mm/hr |
| WQF of Roading | 0.0171 | m3/s |
| WQF of Roading | 61.56 | m3/hr |
| Infiltration rate of media | 250 | mm/hr |
| Area required | 492.5 | m2 |

**Retention Volume**

| | | |
|---|---|---|
| Native Soil Infiltration Rate | 48 | mm/hr |
| Assumed Infiltration rate- 50% reduction for clogging | 24 | mm/hr |
| $K_{subsoil\ rate}$ | 0.024 | m/hr |
| $V_{storage}$ | 35.35 | m3 |
| $V_{infiltration\text{-}72hrs}$* | 872.64 | m3 |
| $V_{infiltration\text{-}24hrs}$ | 290.88 | m3 |

| | | | |
|---|---|---|---|
| **$V_{retention}$ (volume in device)** | 227.25 | m3 | (calucated detention volume in ponding, media and storage volume) |
| **$V_{retention}$ (includng 24hr infiltration)** | 518.13 | m3 | |

*Note that this is calcuated over 72 hr period, in reality it is limited by the volume above. Total retetnion provided can be caculated by the volume within bioretention device. Infiotlration over 24hr can also be assumed

#### 4.3.2.1 Key design requirements for bioretention sizing treatment footprint

The sizing of bioretention devices for water quality purposes is to be based on the footprint of the filter media, relative to the contributing impervious catchment. This simplified method reflects the function of these systems in terms of surface detention, through flow and discharge over a range of operating conditions.

A maximum footprint area of 1,000 to 1,200m$^2$ is recommended for the bioretention systems covered by this guideline.

The maximum area is dictated by constructability to achieve even infiltration rates, even flow distribution and maintenance practicability.

Bioretention devices must be sized based on a relationship of the filter media area being 2% of the contributing untreated impervious area.

This relationship is based on the material and functional properties detailed in Section 4.3.3.2, a ponding depth of 200mm and the design of inlet/outlet hydraulics covered in Section 4.3.3.

This 2% area has been shown to enable bioretention to treat approximately 90% of the mean annual stormwater volume with the remainder bypassing.

This size will also retain adequate moisture in the inter event periods and support robust vegetation, which will maintain the infiltration rates over the long term.

Where bioretention devices are constructed with sloping batters below the filter media surface due to ground instability concerns, the filter media area must be calculated as the footprint of the bioretention measured at the midpoint of the filter media depth (ie 300mm below the finished filter media level). This is particularly important for smaller scale systems where the actual footprint of the device (including ponding depth) can be significantly greater than the defined filter area that is the basis for sizing.

#### 4.3.2.4 Key design requirements for retention

Where retention of flows is required, internal water storage (IWS) can be provided in bioretention. IWS can be provided through the use of a zone at the base (below the underdrains) that does not drain out to the stormwater network, but infiltrates over a period once it has stopped raining. Depending on the site-specific exfiltration rate, IWS can also provide a water source for plants between rainfall events.

The main functions of the IWS are to increase retention and support plant growth over dry periods. There may also be some enhanced treatment due to the anoxic conditions that develop.

To meet specific retention requirements, the required retention volume is calculated. To account for losses to exfiltration during the design event use Equation 5 (below).

**Equation 5: Calculation of the required internal water storage volume.**

$$V_{IWSreq} = REV - i \times A_{bio}$$

Where:

$V_{IWSreq}$ = required volume of the IWS zone ($m^3$)

$REV$ = retention event volume ($m^3$)

$A_{bio}$ = bioretention surface area ($m^2$)

$i$ = exfiltration rate of base soils ($m\ day^{-1}$)

The final volume of the IWS is then determined by calculating the volume below the level of the underdrains, based on the footprint and depth and the corresponding void ratio in Equation 6 below. The depth is then iteratively adjusted until the required volume of the IWS is achieved.

**Equation 6: Calculation of the final internal water storage volume.**

$$V_{iws} = A_{UD} \times D_{IWS} \times e_{IWS}$$

Where:

$V_{iws}$ = volume of IWS zone ($m^3$)

$A_{UD}$ = average area of the underdrainage layer ($m^2$)

$D_{IWS}$ = depth of the internal storage below underdrain outlet (m)

$e_{IWS}$ = internal water storage void ratio (-)

| | |
|---|---|
| VIWSreq | -200.81 |
| i | 576 |

**Retention volumes**

The total available retention volume in the bioretention device can be determined by calculating how much water will infiltrate from the storage layer into the underlying soil over a period of three days (72 hours). The total volume that will infiltrate over three days cannot exceed the total volume that is available in the storage layer below the underdrain.

Any losses due to evapotranspiration from vegetation in the bioretention device can be added to the total retention volume.

Calculate the volume in the retention storage layer below the underdrain (Equation 9).

$$V_{(storage)} = l_{(storage)} \times w_{(storage)} \times d_{(storage)} \times \emptyset \quad \text{Equation 9}$$

Where:

- $V_{(storage)}$ - Volume of water in storage layer (below the drainage layer) ($m^3$)
- $l_{(storage)}$ - Length of storage layer (m)
- $w_{(storage)}$ - Width of storage layer (m)
- $d_{(storage)}$ - Depth of storage layer (m)
- $\emptyset$ - Void space

Calculate the infiltration volume; being the volume infiltrated in 3 days (72 hours) and ensure that the storage volume $V_{(storage)}$ can be infiltrated (Equation 10).

$$V_{(infiltration)} = l_{(storage)} \times w_{(storage)} \times 72 \times K_{(subsoil)} \quad \text{Equation 10}$$
$$V_{(infiltration)} \leq V_{(storage)}$$

Where:

- $V_{(infiltration)}$ - Volume of water that will infiltrate over three days (72 hours) from the storage layer ($m^3$)
- $K_{(subsoil)}$ - Infiltration rate of underlying subsoil (minimum 0.002) (m/hr)

Calculate the evapotranspiration volume; being the volume evapotranspired in 3 days (Equation 11).

$$V_{(evaporation)} = l_{(pond)} \times w_{(pond)} \times 3 \times ET \quad \text{Equation 11}$$

Where:

- $V_{(evapotranspiration)}$ - Volume of water that will evaporate over three days ($m^3$)
- ET - Evapotranspiration rate (use 0.003 for plants in bioretention devices) (m/day)

Calculate the total retention volume $V_{(retention)}$; being the sum of infiltration volume and evapotranspiration volume (Equation 12).

### Detention volumes

The following equations should be used to calculate the available water storage volumes in the ponding, media and drainage layers. Together, these three volumes determine your total available detention volume.

$$V_{(pond)} = l_{(pond)} \times w_{(pond)} \times d_{(pond)}$$ **Equation 5**

Where:

- $V_{(ponding)}$ - Volume of water in the ponding area ($m^3$)
- $l_{(pond)}$ - Length of ponding area (m)
- $w_{(pond)}$ - Width of ponding area (m)
- $d_{(pond)}$ - Ponding depth (m)

$$V_{(media)} = l_{(media)} \times w_{(media)} \times d_{(media)} \times \emptyset$$ **Equation 6**

Where:

- $V_{(media}$ - Volume of water in the media layer ($m^3$)
- $l_{(media)}$ - Average length of media layer (m)
- $w_{(media)}$ - Average width of media layer (m)
- $d_{(media)}$ - Depth of media layer (including depth of transition layer) (m)
- $\emptyset$ - Void space of media

$$V_{(drainage)} = l_{(drainage)} \times w_{(drainage)} \times d_{(drainage)} \times \emptyset$$ **Equation 7**

Where:

- $V_{(drainage)}$ - Volume of water in the drainage layer (around the underdrain) ($m^3$)
- $l_{(drainage)}$ - Length of drainage layer (m)
- $w_{(drainage)}$ - Width of drainage layer (m)
- $d_{(drainage)}$ - Depth of drainage layer (m)
- $\emptyset$ - Void space of drainage aggregate

Calculate the total available detention volume $V_{(detention)}$ in the bioretention device (Equation 8):

$$V_{(detention)} = V_{(ponding)} + V_{(media)} + V_{(drainage)}$$ **Equation 8**

$$A = \frac{WQF}{(0.5 \times K_{(media)})}$$ **Equation 13**

| | | | |
|---|---|---|---|
| Where | A | - | Area of bioretention media bed at its narrowest point ($m^2$) |
| | WQF | - | Water quality flow ($m^3/hr$) |
| | $K_{(media)}$ | - | Infiltration rate of bioretention media (m/hr) |
| | Safety factor for clogging | - | 0.5 |

$$WQF = i_{const}\left(\frac{CA}{360}\right)$$

Where

$WQF$ = Water Quality Flowrate ($m^3/s$)

$i_{const}$ = Constant rainfall intensity 10mm/hr

$C$ = Runoff coefficient as per Regional Standard for Water Services

$A$ = Area of catchment (hectares) as per Regional Standard for Water Services

#### 4.3.1.2 Runoff Coefficients (C)

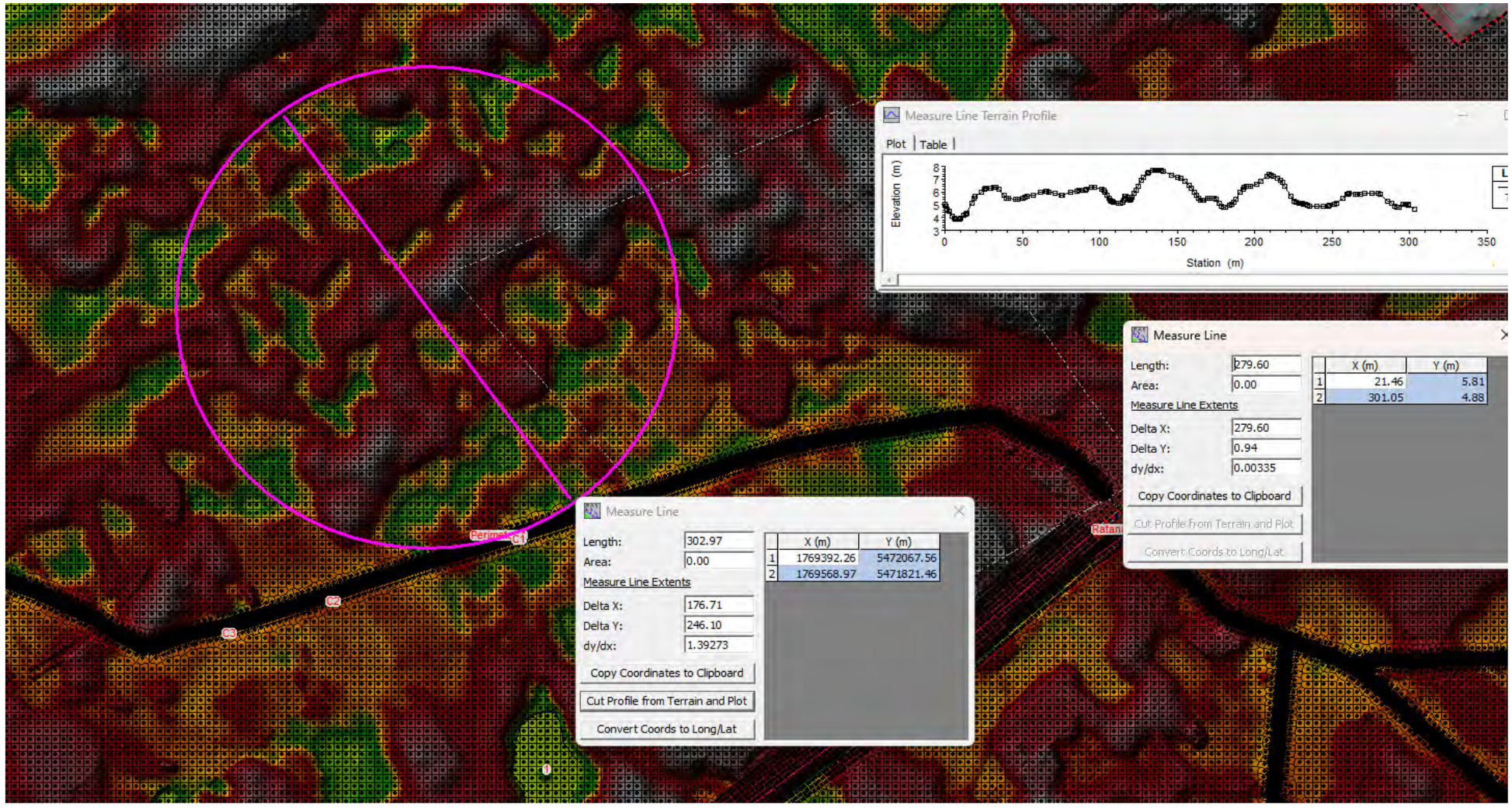
**Table 4.4 – Runoff Coefficients to be used with Hydrological Design**

| Area Type | Coefficient C |
|---|---|
| Fully paved or roofed areas, CBD areas or urban, industrial or commercial areas with greater than 65% coverage permitted by the district plan. | 0.95 |
| Industrial/commercial areas with paved plus roof area up to 65% coverage permitted by the district plan. | 0.70 |
| Urban areas allowing between 36% and 65% impervious site coverage (inner residential, infill housing, intensive residential development). | 0.65 |
| Urban areas allowing coverage up to 35% (residential or outer areas). | 0.50 |
| Parks, reserves, green spaces, rural areas. | 0.35 |

| Elevation (m) | Surface Area ($m^2$) | Incremental Volume ($m^3$) | Cumulative Volume ($m^3$) | 1000m3 |
|---|---|---|---|---|
| 4 | 2,803.50 | 0 | 0 | 0 |
| 4.1 | 2,867.72 | 283.56 | 283.56 | 0.28356 |
| 4.2 | 2,932.67 | 290.02 | 573.58 | 0.57358 |
| 4.3 | 2,998.36 | 296.55 | 870.13 | 0.87013 |
| 4.4 | 3,064.78 | 303.16 | 1,173.29 | 1.17329 |
| 4.5 | 3,131.93 | 309.84 | 1,483.13 | 1.48313 |
| 4.6 | 3,199.81 | 316.59 | 1,799.72 | 1.79972 |
| 4.7 | 3,268.42 | 323.41 | 2,123.13 | 2.12313 |
| 4.8 | 3,337.77 | 330.31 | 2,453.44 | 2.45344 |
| 4.9 | 3,407.85 | 337.28 | 2,790.72 | 2.79072 |
| 5 | 3,478.66 | 344.33 | 3,135.05 | 3.13505 |
| 5.1 | 3,550.20 | 351.44 | 3,486.49 | 3.48649 |
| 5.2 | 3,622.48 | 358.63 | 3,845.12 | 3.84512 |
| 5.3 | 3,695.49 | 365.9 | 4,211.02 | 4.21102 |
| 5.4 | 3,769.23 | 373.24 | 4,584.26 | 4.58426 |

| Elevation (m) | Surface Area ($m^2$) | Incremental Volume ($m^3$) | Cumulative Volume ($m^3$) | 1000m3 |
|---|---|---|---|---|
| 4 | 1,022.00 | 0 | 0 | 0 |
| 4.1 | 1,061.12 | 104.16 | 104.16 | 0.10416 |
| 4.2 | 1,100.97 | 108.1 | 212.26 | 0.21226 |
| 4.3 | 1,141.56 | 112.13 | 324.39 | 0.32439 |
| 4.4 | 1,182.88 | 116.22 | 440.61 | 0.44061 |
| 4.5 | 1,224.93 | 120.39 | 561 | 0.561 |
| 4.6 | 1,267.72 | 124.63 | 685.63 | 0.68563 |
| 4.7 | 1,311.25 | 128.95 | 814.58 | 0.81458 |
| 4.8 | 1,355.51 | 133.34 | 947.92 | 0.94792 |
| 4.9 | 1,400.50 | 137.8 | 1,085.72 | 1.08572 |
| 5 | 1,446.23 | 142.34 | 1,228.05 | 1.22805 |
| 5.1 | 1,492.69 | 146.95 | 1,375.00 | 1.375 |
| 5.2 | 1,539.89 | 151.63 | 1,526.63 | 1.52663 |
| 5.3 | 1,587.83 | 156.39 | 1,683.02 | 1.68302 |
| 5.4 | 1,636.49 | 161.23 | 1,844.23 | 1.84423 |

| Subcathment | Curve No | St | Ia | Longest flow path length (m) | Slope % | n | Overland flow Tc (min) | Lag (min) |
|---|---|---|---|---|---|---|---|---|
| North | 66 | 130.85 | 13.08 | 280 | 0.03 | 0.045 | 65.69 | 39.41 |
| South | 66 | 130.85 | 13.08 | 280 | 0.02 | 0.045 | 71.89 | 43.13 |




| | |
|---|---|
| HSG | 80% A |
| | 10% B and D |

Refer sheet Smap

| Subcathment | Curve No | Weighted curve No | Ia | Weighted Ia | Longest flow path length (m) | Slope % | n | Composite n | Overland flow Tc (min) | Lag (min) |
|---|---|---|---|---|---|---|---|---|---|---|
| Proposed Nothern | | | | | | | | | | |
| Impervious Area | 98 | | 0 | | | | 0.015 | | | |
| Pervious Area | 37 | 61.7 | 5 | 2.98 | 489.7 | 2.02 | 0.045 | 0.033 | 24.03 | 14.42 |
| Proposed Southern | | | | | | | | | | |
| Impervious Area | 98 | | 0 | | | | 0.015 | | | |
| Pervious Area | 37 | 78.4 | 5 | 1.61 | 291 | 1.03 | 0.045 | 0.025 | 17.34 | 10.41 |

| Areas | Nothern | Southern | | |
|---|---|---|---|---|
| | Area (m2) | | | |
| Proposed Impevious Area | 37907 | 22574 | | 60481 |
| Proposed Pervious Area | 55831 | 10713 | | 66544 |
| Total Area | 93738 | 33287 | 127025 | 127025 |
| | 0.093738 | 0.033287 | | 0.066544 |
| | | 0.355107 | | 0.127025 |

**South Pond Detention Outlet Sizing**

| | | | | |
|---|---|---|---|---|
| Average 24 Hour Release Rate (L/s) | | | | 6.53 |
| Peak 24 Hour Release Rate (L/s) | | | | 13.06 |
| Tank Outlet Head (m) | | | | 0.30 |
| Peak Outflow Velocity Formula | | | | $0.62(2gH)^{1/2}$ |
| Peak Outflow Velocity (m/s) | | | | 1.50 |
| Orifice Area ($mm^2$) | | | | 8684.86 |
| Orifice Diameter (mm) | | | | 105.16 |

0.008684863

Table B.1 Curve number values used to formulate the SCS curve number map

| LAND COVER | SOIL GROUP A Sand, loamy sand, or sandy loam (low runoff potential) | B Silt loam or loam | C Sandy clay loam | D Clay loam, silty clay loam, sandy clay, silty clay, or clay (high runoff potential) |
|---|---|---|---|---|
| Alpine tussock/grass | 66 | 77 | 84 | 87 |
| Bare | 66 | 77 | 84 | 87 |
| Forest | 28 | 46 | 63 | 71 |
| Impervious | 98 | 98 | 98 | 98 |
| Pasture-Crop | 37 | 59 | 72 | 78 |
| Scrub/Flax | 33 | 54 | 68 | 75 |
| Urban Open Space | 37 | 59 | 72 | 78 |

Table 2-1 Initial abstraction values

| Landuse | Initial Abstraction |
|---|---|
| Undeveloped (i.e. rural, pasture, forestry) | 0.1 Si |
| Developed pervious | 5 mm |
| Developed impervious | 0 mm |

In mixed landuse catchments where a portion of the landuse is undeveloped, the $I_a$ for the undeveloped area should be calculated as a function of S to determine the initial abstraction depth in millimetres [Equation 2]. This value can then be weighted based on area with values from the urban portion of the catchment. Where the catchment is entirely rural, InfoWorks software allows the user to directly enter the factor rather than the abstraction depth.

The weighted $I_a$ can be calculated using Equation 4.

$$Weighted\ I_a = \frac{0.1S_pA_r + 0A_i + 5A_p}{Total\ area}$$

[Equation 4]

Where $I_a$ is in mm;
r is the rural portion of the catchment;





i is the impervious portion of the developed catchment; and
p is the pervious portion of the developed catchment

The weighted curve number can be calculated using [Equation 1].

$$Weighted\ CN = \frac{\sum CN_x A_x}{A_{tot}}$$

[Equation 1]

**North Pond Detention Outlet Sizing**

| | | | | |
|---|---|---|---|---|
| Average 24 Hour Release Rate (L/s) | | | | 10.70 |
| Peak 24 Hour Release Rate (L/s) | | | | 21.40 |
| Tank Outlet Head (m) | | | | 0.30 |
| Peak Outflow Velocity Formula | | | | $0.62(2gH)^{1/2}$ |
| Peak Outflow Velocity (m/s) | | | | 1.50 |
| Orifice Area ($mm^2$) | | | | 14228.80 |
| Orifice Diameter (mm) | | | | 134.60 |

0.014228802




| subbasin | Length (m) | Basin Slope (m/m) |
|---|---|---|
| North Development | 489.69 | 0.02021 |
| South Development | 291.045 | 0.04092 |

3

0.010308

**Table 2-3 Horton's roughness values**

| Surface Type | Horton's roughness values |
|---|---|
| Paved | 0.015 |
| Bare Soil | 0.0275 |
| Poorly Grassed | 0.035 |
| Average Grass | 0.045 |
| Dense Grass | 0.06 |

To estimate the time of concentration of the component parts, time of overland flow (also often referred to as sheet flow) can be determined using Friend's equation [Equation 8], time of shallow concentrated flow for pervious areas can be estimated from Manning's derived equation for unpaved areas [Equation 9], and for impervious areas from Manning's derived equation for gutter flow [Equation 10]. Time of pipe flow can be estimated as a function of pipe velocity [Equation 11] and time of open channel flow can be estimated from Manning's equation [Equation 11, 12 and 13].

$$Time\ of\ overland\ flow = \frac{107nL^{0.333}}{s^{0.2}}$$

[Equation 8]

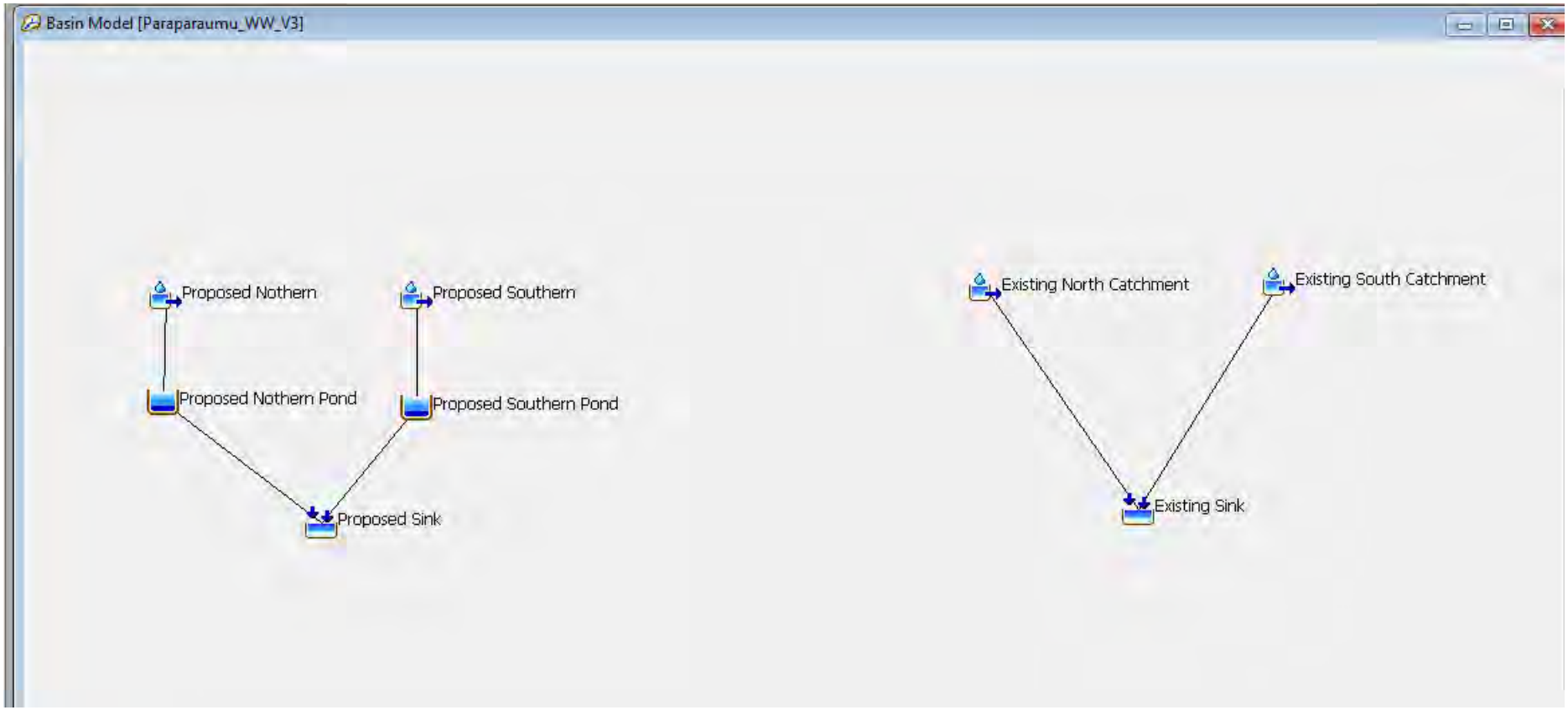
Where overland flow is in minutes;

n is Horton's roughness value for the surface;

L is length in metres (m); and

s is slope in percentage (integer i.e. 3.0 for 3%)

Horton's roughness values are similar, though not identical, to Manning's *n*. Horton's roughness values are detailed in Table 2-3.


Basin Model [Paraparaumu_WW_V3]
Proposed Nothern
Proposed Southern
Proposed Nothern Pond
Proposed Southern Pond
Proposed Sink
Existing North Catchment
Existing South Catchment
Existing Sink

Global Summary Results for Run "V3 2yr"

Project: T1 Simulation Run: V3 2yr

Start of Run: 01Jan2000, 00:00
End of Run: 02Jan2000, 12:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: Paraparaumu_WW_V3
Meteorologic Model: Met 2
Control Specifications: Control 1

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Watershed Explorer

| Hydrologic Element | Drainage Area (KM2) | Peak Discharge (M3/S) | Time of Peak | Volume (MM) |
|---|---|---|---|---|
| Proposed Southern | 0.03329 | 0.219 | 1 January 2000, ... | 42.36 |
| Proposed Nothern | 0.09374 | 0.312 | 1 January 2000, ... | 25.92 |
| Proposed Nothern Pond | 0.09374 | 0.047 | 1 January 2000, ... | 25.64 |
| Proposed Southern Pond | 0.03329 | 0.034 | 1 January 2000, ... | 42.10 |
| Proposed Sink | 0.12702 | 0.081 | 1 January 2000, ... | 29.96 |
| Existing Sink | 0.12703 | 0.139 | 1 January 2000, ... | 22.90 |
| Existing North Catchment | 0.09374 | 0.104 | 1 January 2000, ... | 22.91 |
| Existing South Catchment | 0.03329 | 0.035 | 1 January 2000, ... | 22.87 |

Global Summary Results for Run "V3 10yr"

Project: T1 Simulation Run: V3 10yr

Start of Run: 01Jan2000, 00:00
End of Run: 02Jan2000, 12:00
Compute Time: 24Sep2026, 21:05:47

Basin Model: Paraparaumu_WW_V3
Meteorologic Model: Met 10
Control Specifications: Control 1

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Watershed Explorer

| Hydrologic Element | Drainage Area (KM2) | Peak Discharge (M3/S) | Time of Peak | Volume (MM) |
|---|---|---|---|---|
| Proposed Southern | 0.03329 | 0.415 | 1 January 2000,... | 74.92 |
| Proposed Nothern | 0.09374 | 0.653 | 1 January 2000,... | 50.21 |
| Proposed Nothern Pond | 0.09374 | 0.141 | 1 January 2000,... | 49.14 |
| Proposed Southern Pond | 0.03329 | 0.082 | 1 January 2000,... | 74.29 |
| Proposed Sink | 0.12702 | 0.222 | 1 January 2000,... | 55.73 |
| Existing Sink | 0.12703 | 0.324 | 1 January 2000,... | 47.75 |
| Existing North Catchment | 0.09374 | 0.243 | 1 January 2000,... | 47.77 |
| Existing South Catchment | 0.03329 | 0.082 | 1 January 2000,... | 47.69 |

# Appendix 4 Flood Risk Assessment

# Flood Risk Assessment

## Summerset Group Holdings Ltd

CKL

Planning | Surveying | Engineering | Environmental

# Document Information

| | |
|---|---|
| **Client** | **Summerset Group Holdings Ltd** |
| **Site Location** | 65 & 73 Ratanui Road, Paraparaumu, Wellington, WELL, NZ |
| **Legal Description** | LOT 4 DP 58017 CT 29D/823 & LOT 3 DP 497389 CT 734643 |
| **CKL Reference** | A25556 |
| **Office of Origin** | Auckland |

| | | | |
|---|---|---|---|
| **Author** | Sarala Karunarathna | | |
| **Signed** | [redacted] | **Date** | 9/10/2026 |

| | | | |
|---|---|---|---|
| **Reviewed By** | Frances Deamer-Phillips | | |
| **Signed** | [redacted] | **Date** | 9/10/2026 |

| | | | |
|---|---|---|---|
| **Authorised By** | Zeb Worth | | |
| **Signed** | [redacted] | **Date** | 19/10/2026 |

| Revision | Status | Date | Author | Reviewed By | Authorised By |
|---|---|---|---|---|---|
| 0 | Issue for peer review | 25.08.26 | SK | FDP | ZW |
| 1 | Issue for RC | 20.10.26 | SK | FDP | ZW |
| | | | | | |

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# Executive Summary

This Flood Risk Assessment has been prepared to support the Fast-track resource consent application for the proposed Summerset retirement village at 65 and 73 Ratanui Road, Paraparaumu. The development involves extensive earthworks and filling on undulating dune landscape to establish raised building platforms above existing flood levels, together with associated infrastructure and stormwater management facilities. The site is traversed by an existing drain and contains low-lying areas subject to flooding. The proposed stormwater management strategy incorporates ecological and attenuation wetlands and drainage infrastructure to manage runoff and maintain downstream flood conveyance. Hydraulic modelling has been undertaken to assess the effects of the proposed development on flood levels, flow paths and neighbouring properties, with iterative design refinements incorporated to minimise adverse effects.

- The assessment has considered the 1% [1]Annual Exceedance Probability (AEP) rainfall event, including climate change, with additional sensitivity and resilience assessments undertaken. The hydraulic modelling demonstrates that: **Flood attenuation systems operate effectively**, with sufficient capacity to accommodate the 1% AEP event without surcharging and controlled outlets limiting downstream discharges.
- **The practical intent of hydraulic neutrality has been achieved for the proposal**, despite a minor increase in peak discharge of approximately 0.03 $m^3/s$, with negligible downstream flood-level changes, generally less than 10 mm.
- **Downstream flood effects are negligible**, with no material increase in flood risk within the downstream receiving environment.
- **Residual boundary effects are localised and acceptable**, with mitigation incorporated where practicable and remaining effects assessed as low relative to the existing flood risk.
- **Safe egress is maintained** during the 100-yr ARI event on road crossing over the drain, with resilience assessment confirming that no flood waters cross the road during the 500-year event.

Overall, the assessment demonstrates that the proposed filling, stormwater attenuation wetlands and associated drainage infrastructure can appropriately manage flood risk while enabling development of the site. The proposal is not expected to result in material adverse flood effects on neighbouring properties or the downstream receiving environment and is considered appropriate from a flood risk perspective.

[1] 100 yr ARI event

# 1 Introduction

CKL has been engaged by Summerset Villages (Paraparaumu) Ltd to undertake a flood risk assessment for the proposed retirement village development located at 65 and 73 Ratanui Road, Paraparaumu (The site). The purpose of this assessment is to support informed decision-making during both the conceptual and detailed design stages of the project.

The site is intersected by a 1% Annual Exceedance Probability (AEP) conveyance channel. As such, the assessment is required to evaluate potential flood-related impacts and to ensure that the proposed development represents hydraulic neutrality, with no adverse effects on flood behaviour within or beyond the site.

Essential information in completing the assessment has been derived from the following sources.

- Waikanae Flood Plain Management Plan, 1997 (Greater Wellington Regional Council – (GWRC))
- Waikanae Floodplain Management Plan-10-year Review, 2010 (GWRC)
- Kapiti Flood Hazard Assessment Mazengarb Model Build Report, 2025 (AWA Environmental on behalf of Kapiti Coast District Council- KCDC)
- Review of Kapiti Coast Flood Hazard Management Reports, 2025 (Dr. Willem de Lange)
- Flood Hazard Modelling Standard, Greater Wellington Regional Council, 2025
- Reference Guide for Design Storm Hydrology by Cardno NZ Ltd. on behalf of Wellington Water, 2019
- NIWA HIRDS V4
- S Map Soil Reports
- Green- Ampt Infiltration Parameters from Soils Data (Rawls et al.), 1983
- HEC RAS Hydraulic Reference Manual
- AWA flood model information received by AWA Environmental
- Land Development Minimum Requirements, April 2022 by Kapiti Coast District Council

## 1.1 Site Location

The proposed retirement village site is located south of the lower Waikanae River, upstream of the Waikanae River estuary where the river discharges to the sea. The development comprises two land parcels located at 65 and 73 Ratanui Road, Paraparaumu.

As demonstrated below in Figure 1, the site is bounded by Ratanui Road to the south and Otaihanga Road to the east, with State Highway 1 (SH1) situated further to the south.




*Figure 1: Site Location*

## 1.2 Site Topography

The site is characterised by gently undulating terrain, with several localised depressions typically ranging from approximately 1 m to 2 m in depth. Ground elevations across the site are generally in the order of 5–6 m RL (NZVD 2016), with localised variations of approximately ±3 m.

A defined 1% Annual Exceedance Probability (AEP) overland flow conveyance channel traverses the site, draining the upstream catchment. This flow path connects to the Mazengarb Stream system and is subject to hydraulic influence from backwater effects associated with elevated water levels in the Waikanae River during 1% AEP flood events (Figure 2).




*Figure 2: Site Topography*

## 1.3 Existing and Proposed Conditions

The Site was recently rezoned from Rural Lifestyle Zone to General Residential Zone through Private Plan Change 4 (PC4), which became operative on 1 September, and introduced the Ratanui Development Area provisions. PC4 provides for either residential or retirement village development of the Site. The site contains one existing building. The majority of the site comprises undeveloped pastureland with limited built infrastructure.

The proposed development involves the construction of a comprehensive care retirement village, comprising residential units, main building and associated infrastructure.

# 2 Assessment Methodology

This section presents the methodology and stages of the flood risk assessment undertaken for the proposed development, including a review of existing flood behaviour and an assessment of potential effects of the proposed development. The assessment evaluates whether hydraulic neutrality and/or a similar outcome can be achieved and identifies any changes in flood levels, flow paths, and flood extents within and beyond the site under proposed development conditions.

The assessment comprises the following:

1. Verification of the CKL flood model results (existing conditions) against the existing AWA flood model developed for Kapiti Coast District Council.
2. Master planning assessment.
3. Concept design assessment.
4. Sensitivity and resilience assessments.

The following subsections describe each component of the assessment in detail, commencing with the modelling approach in Section 2.1.

## 2.1 Modelling Approach

A two-dimensional (2D) HEC-RAS rain-on-grid model has been developed to simulate flood behaviour within and surrounding the site. The model domain extends to State Highway 1 (SH1), covering the majority of the contributing catchment. Runoff from catchments upstream of SH1 has been incorporated as inflow hydrographs derived from the existing AWA flood model, while a stage hydrograph at Mazengarb Stream has been adopted as the downstream boundary condition to represent downstream tidal influences.

Simulations have been undertaken for both existing (pre-development) and proposed (post-development) conditions, with the primary assessment based on the 1% Annual Exceedance Probability (AEP) event, including a +3.8°C climate change allowance (RCP 2081–2100), consistent with the AWA model.

Hydraulic performance has been assessed by comparing model outputs between the existing and proposed scenarios. Hydraulic neutrality has been evaluated downstream of the development, to confirm that the proposal does not result in adverse off-site flood impacts. Please refer model build summary in Appendix 1.

## 2.2 Verification of the CKL Model

The existing Council model, developed by AWA Environmental, was used to verify the CKL model. Although inherent differences exist between the models due to the modelling software, adopted methodologies, and other contributing factors, the CKL model was refined to achieve comparable flood depths across the model domain. Please see the following figures (Figure 3 and Figure 4) for a comparison of flood depths between the AWA and CKL models. Flood depths have been plotted using the same colour bands as the AWA model to facilitate comparison.

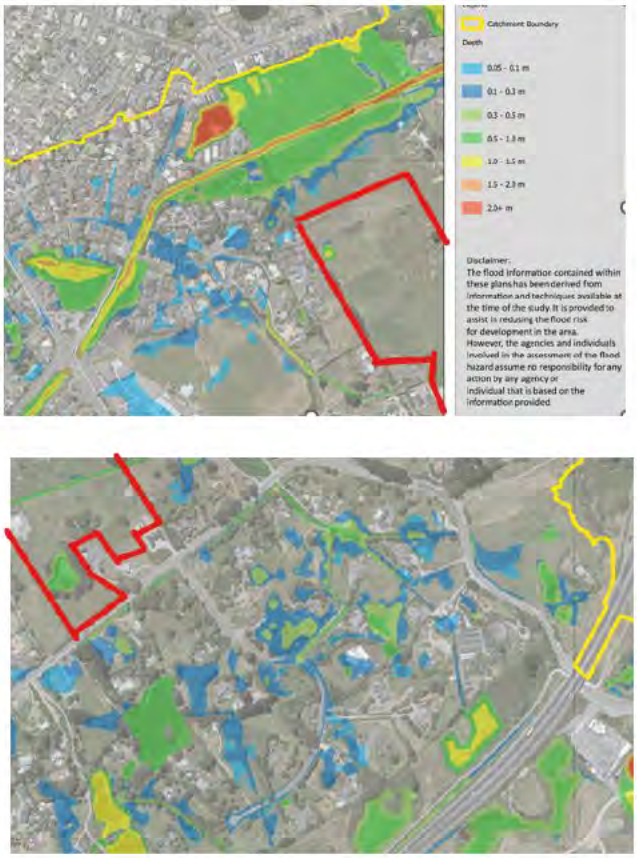



*Figure 3: Flood depths from AWA flood model 100CC (Site extents outlined in red solid line)*

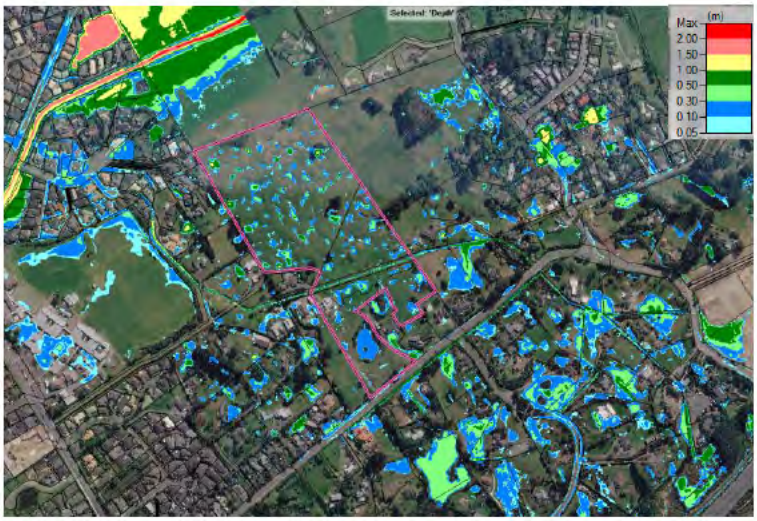



*Figure 4: Flood depths from CKL flood model 100CC (Site extents outlined in pink solid line)*

## 2.3 Master Planning

A concept design was previously prepared by Woods and has been adopted for this assessment with targeted refinements to maximise developable area while minimising potential flooding effects. The original layout included northern and southern attenuation basins and an additional northern flood storage area (Figure 5).

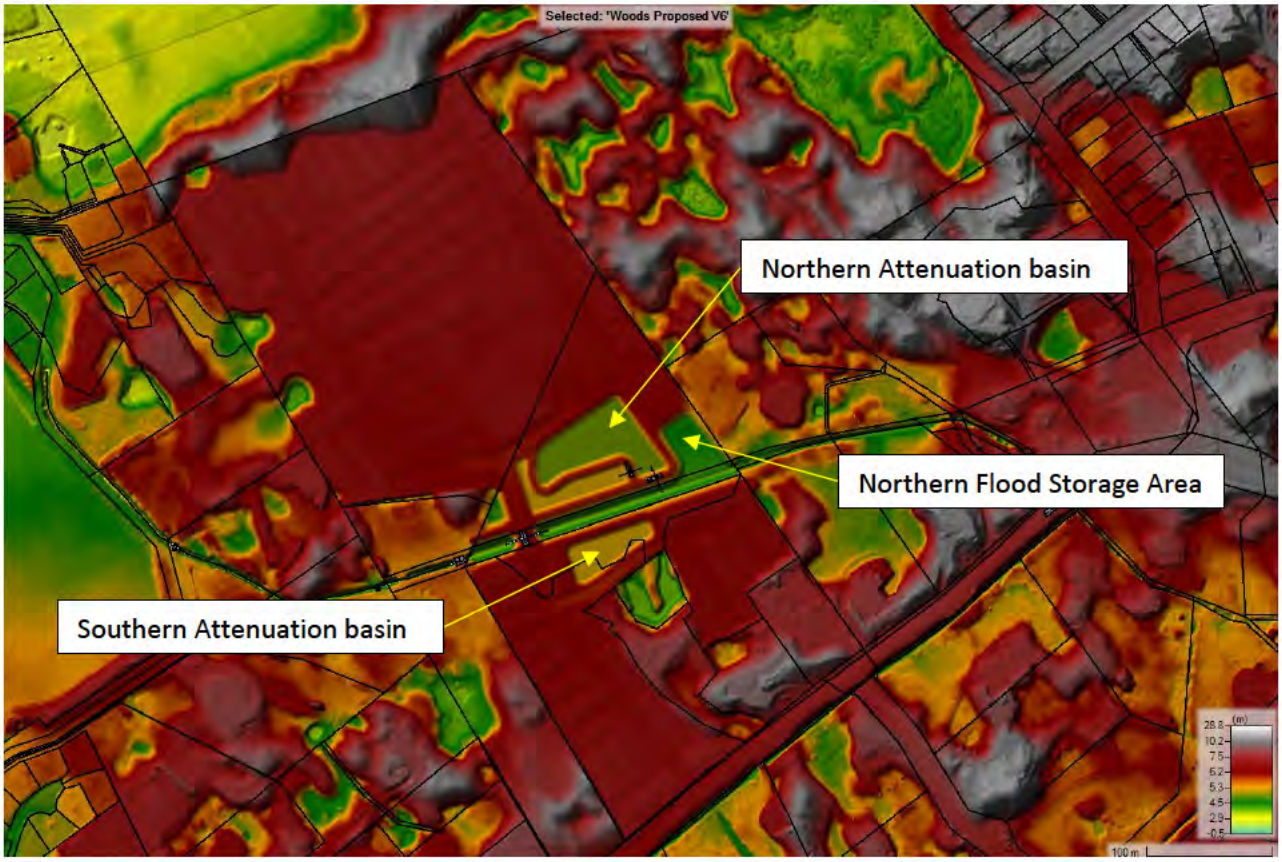



*Figure 5: Woods Proposed Concept Design*

CKL assessed two alternative scenarios:

- removal of the northern flood storage area while retaining the northern attenuation basin; and
- retention of the northern flood storage area while removing the northern attenuation basin.

The southern attenuation basin remained unchanged in both scenarios. The assessment indicated that removing the northern flood storage area would not compromise the hydraulic neutrality and would still be maintained. This option was therefore selected, providing additional usable area for the development. Please see Figure 6.

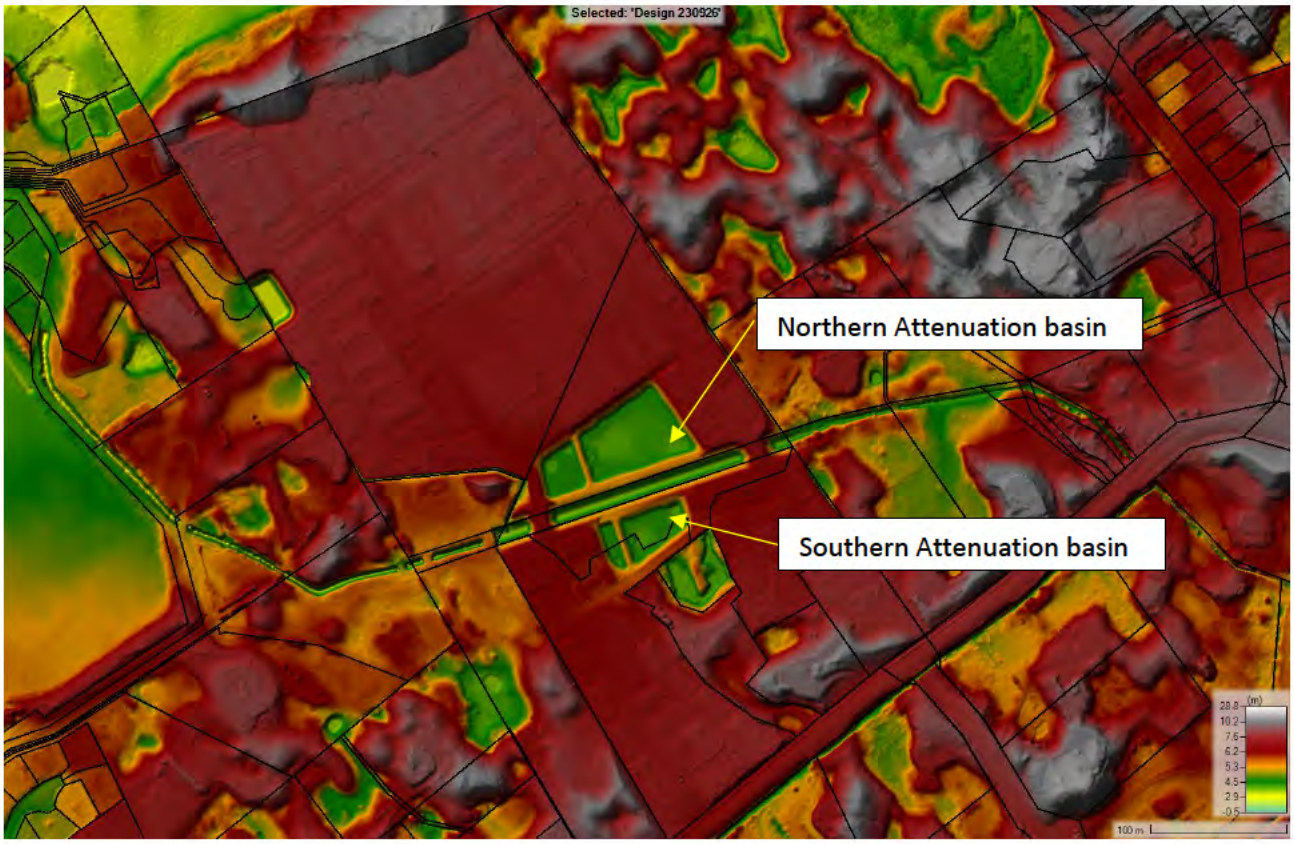



*Figure 6: CKL Modified Design*

## 2.4 Concept Design Assessment

Following completion of the master-planning assessment described above, the concept design was developed and assessed. From a flood-modelling perspective, this involved incorporating the proposed design surface into the model and comparing the results with the existing-condition scenario.

Each design iteration was assessed against the following criteria:

1. whether the proposed design would result in adverse upstream or downstream flood effects in terms of increased flood levels; and
2. whether hydraulic neutrality and/or a similar outcome could be achieved.

Whenever the design surface was refined, it was incorporated into the model and reassessed against these criteria.

Localised increases in flooding were identified at several neighbouring properties. These increases were primarily associated with proposed filling that obstructed external overland flow paths entering the site. Under existing conditions, these flows enter the site, pond within low-lying areas of the site, and infiltrate into the ground.

Where practicable, mitigation measures were incorporated to convey these external flows through the site and manage their discharge through the proposed drainage system.

Where mitigation was not practicable due to the highly undulating terrain and the extensive retaining structures required, the affected areas were assessed individually. The significance of the residual flood hazards was documented in the memorandum dated 09.10.26, considering the magnitude of the predicted changes and the associated risk relative to existing conditions.

According to the risk assessment the existing risk level is H1 as per the Greater Wellington Regional Council risk categorization. It will remain the same under the proposed conditions despite the slight increase in flood levels. Risk category H1 is defined as generally safe for people, buildings and vehicles.

## 2.5 Resilience Assessment

The assessment confirms that safe egress is maintained along the proposed internal road network and across the drainage channel during the 100-year rainfall event. Given that Ratanui Road provides the sole access to the site, and the proposed culvert crossings are the only access routes to and from the northern development area, a further resilience assessment was undertaken for the 500-year rainfall event. This assessment evaluates whether the crossings remain accessible during more extreme flood events, when safe evacuation is particularly important. The results are presented in the following sections.

# 3 Method of Assessment

CKL used HEC-RAS Version 6.5 to assess the flood effects of the proposed development. A rain-on-grid approach was adopted, with the model domain covering the site and neighbouring properties. The domain extends upstream to the State Highway and downstream to Mazengarb Stream.

Flows from the catchment upstream of the State Highway were incorporated using a flow hydrograph derived from the existing AWA model. A stage hydrograph for Mazengarb Stream, also derived from the AWA model, was applied as the downstream boundary condition.

Rainfall depths were obtained from the NIWA High Intensity Rainfall Design System (HIRDS) Version 4 dataset and temporally distributed using the nested rainfall pattern presented in the *Reference Guide for Design Storm Hydrology*, prepared by Cardno on behalf of Wellington Water.

Two model scenarios were assessed: existing and proposed conditions. The differences between the scenarios relate primarily to terrain, land use, and hydraulic controls.

The primary assessment was undertaken for the 100-year design event, incorporating the RCP8.5 climate-change projection for the 2081–2100 horizon. The 500-year event has also been assessed as part of the resilience assessment. Further details of the model inputs are provided in the Hydraulic Modelling Summary in Appendix 1.

# 4 Results of Assessment

Section 4.1 presents the modelled flood depths, water levels, and velocities under existing conditions, while Section 4.2 presents the corresponding results under proposed conditions. Section 4.3 will discuss the flood level difference map and hydraulic neutrality between the two scenarios.

## 4.1 Flood results under existing scenario

### 4.1.1 Flood Extents and Depths

Consistent with the floodplain characteristics, scattered areas of ponding are observed across the model domain, with flood depths ranging from 0.05 m to 0.5 m. In Figure 7, flood depths are mostly depicted in light blue, dark blue and light green colours which as per the shown legend represents flood depths ranging from 0.05m to 0.5m. The extent of flooding under existing conditions is presented below (Figure 7), followed by the corresponding flood-depth map (Figure 8).

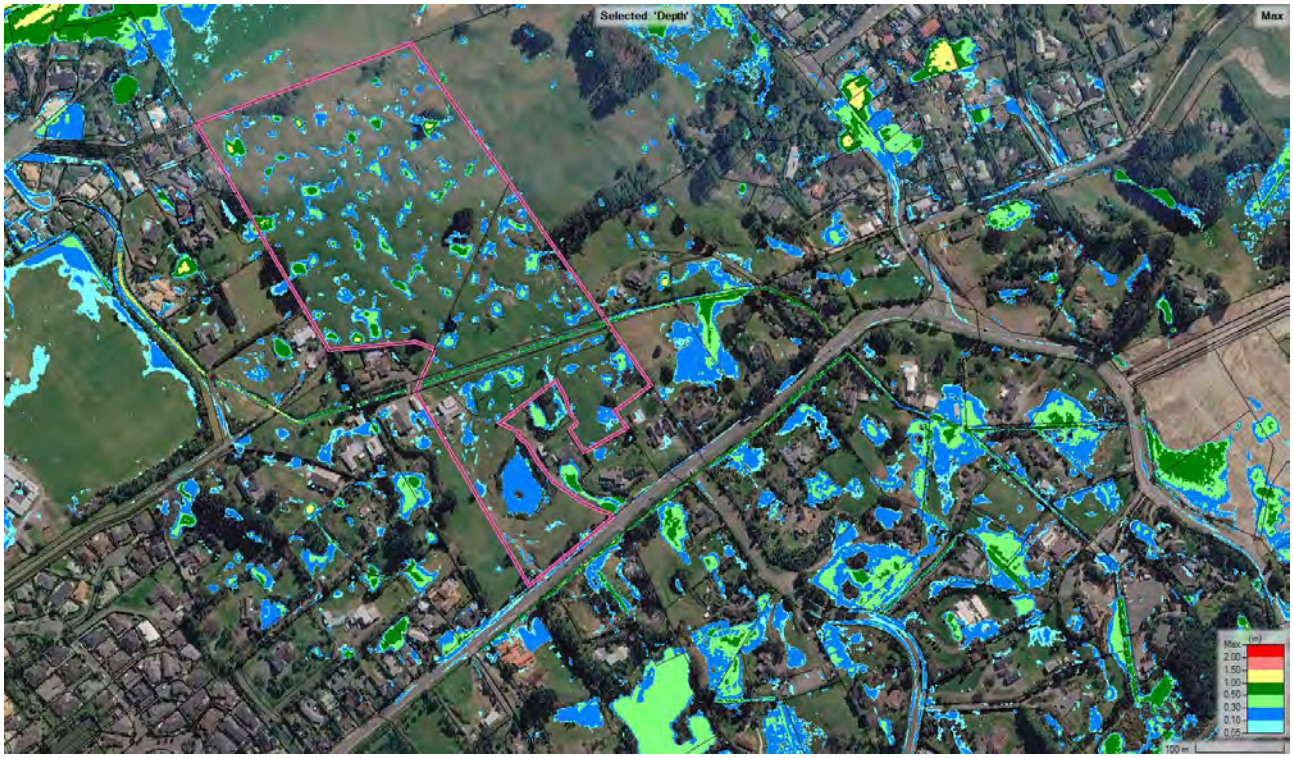



*Figure 7: Flood Extents Under Existing Conditions - 100yrCC*




*Figure 8: Flood Depths Under Existing Conditions - 100yrCC*

### 4.1.2 Flood Levels

Corresponding flood levels in NZVD2016 are shown below in Figure 9.




*Figure 9: Flood Levels Under Existing Conditions - 100yrCC*

### 4.1.3 Flood Velocities

Under existing conditions, flood velocities range from approximately 0.01 m/s in ponding areas to 0.5 m/s within the stream channel, as shown in Figure 10.

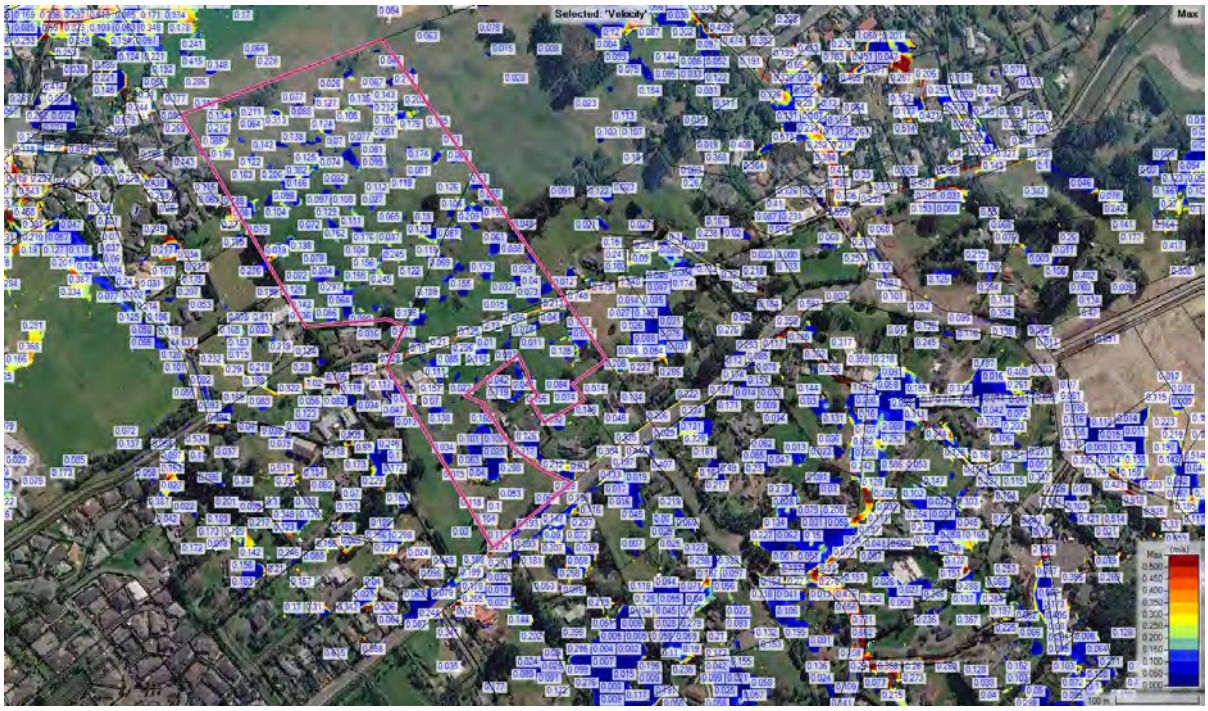



*Figure 10: Flood Velocities Under Existing Conditions - 100yrCC*

## 4.2 Flood results under proposed scenario

### 4.2.1 Flood Extents and Depths

The modelled flood extent and depths under proposed conditions, following incorporation of the proposed development surface, are shown in Figure 11 and Figure 12, respectively.

Similar to the existing scenario, flood depths under proposed condition are generally ranging from 0.05m to 0.5m within and outside the site. Please refer Figure 11 and attached legend. An exception is on the proposed western flood storage area where flood depths exceeding 1m are reached.

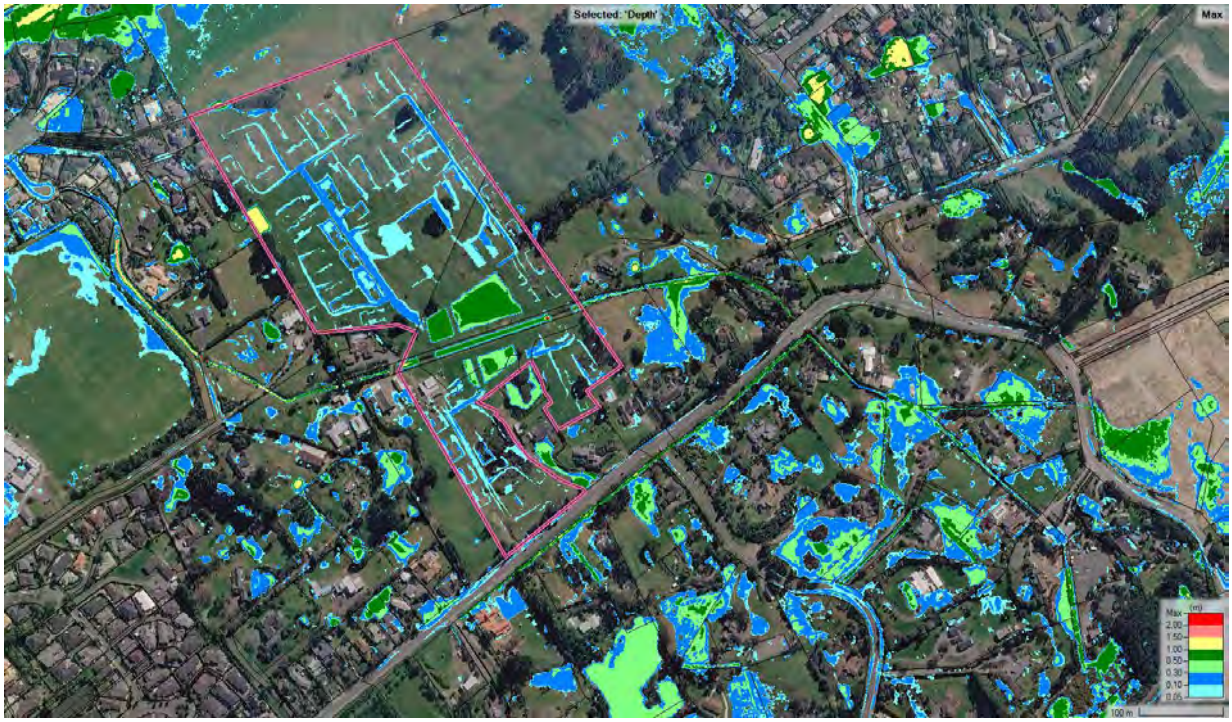



*Figure 11: Flood Extents Under Proposed Conditions - 100yrCC*




*Figure 12: Flood Depths Under Proposed Conditions - 100yrCC*

### 4.2.2 Flood Levels

Under proposed conditions, flood levels across the surrounding floodplain remain generally consistent with existing conditions, except within the proposed development area as shown in Figure 13. This is further discussed in Section 4.3 impact assessment, which elaborates more into proposed flood extents and levels and how it deviates from existing.




*Figure 13: Flood Levels Under Proposed Conditions - 100yrCC*

### 4.2.3 Flood Velocities

Distribution of flood velocities under the proposed conditions is shown in Figure 14.

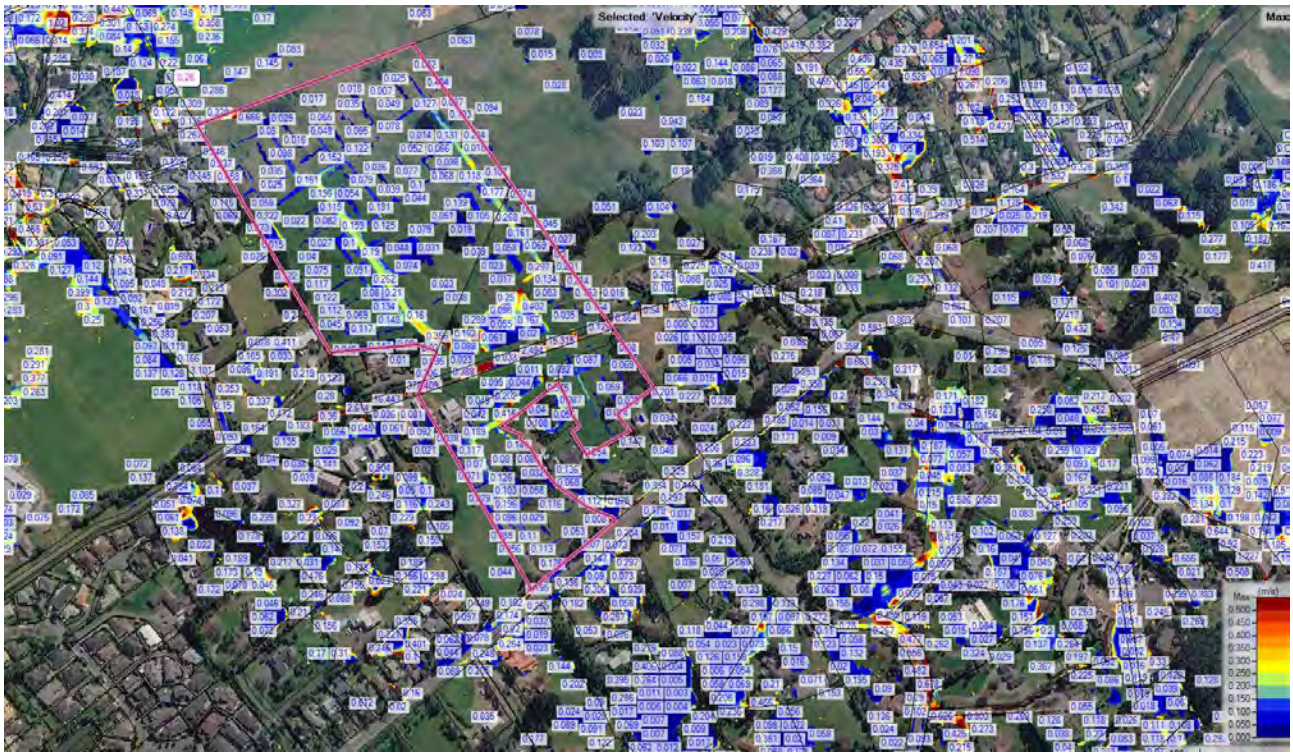



*Figure 14: Flood Velocities Under Proposed Conditions - 100yr*

## 4.3 Flood Impact Assessment

Flood impact has been assessed in terms of:

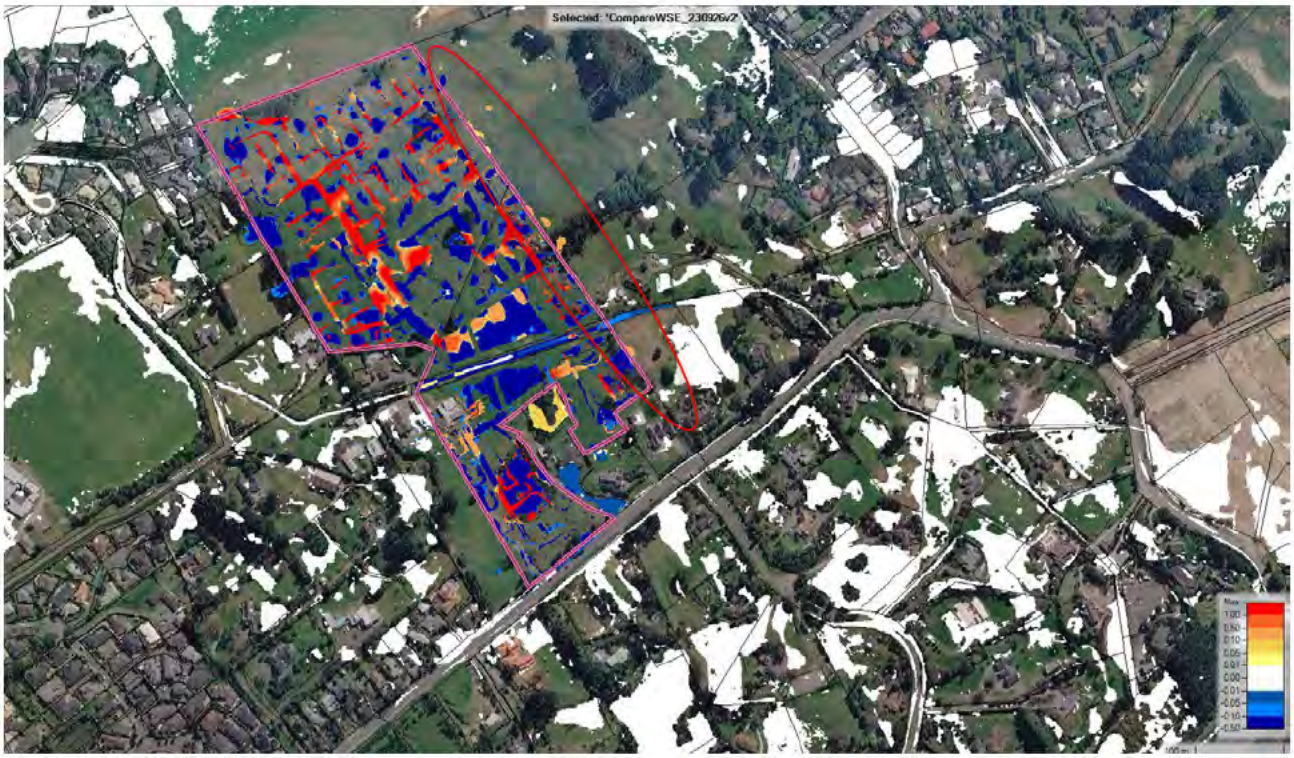
1) the change in flood levels under the proposed conditions compared with existing conditions; and
2) hydraulic neutrality.

Consistent with the AWA model, this assessment assumes all culverts are operational. The primary stormwater network within the site has not been included in the model.

### 4.3.1 Flood Level Difference map

A flood-level difference map was prepared by comparing the proposed-condition flood levels with those under existing conditions. The initial model results indicated localised increases in flood levels along the development boundary. These increases were primarily caused by the raised development platform obstructing external overland flows that currently enter the site, pond within low-lying areas of the site, and infiltrate into the ground. The increases were not caused by additional discharge from the proposed development onto neighbouring properties.

Where practicable, the proposed design surface was iteratively refined to mitigate the effects of obstructing these external flow paths. Figure 15 presents the results of the mitigated scenario, incorporating the proposed mitigation measures where feasible. These mitigation measures are further discussed in the memorandum dated 09.10.26. Please refer Appendix 2.




Note: Increase in flood levels marked using red circle, these areas are further discussed in the memo attached in Appendix 2

*Figure 15: Flood level difference map between proposed and existing scenario - 100yrCC*

At several locations, the undulating terrain means that mitigation would require impracticable engineering works, including extensive retaining structures. A separate memorandum[2] has been prepared to document these constraints and assess the pre- and post-development flood risks at the affected locations. These locations are outlined in red circles on the flood-level difference map (Figure 15).

Except at the identified locations, increases in flood level are generally less than 10 mm and are shown in white in accordance with the figure legend. Blue areas indicate a reduction in flood level, while yellow areas indicate an increase greater than 10 mm.

### 4.3.2 Hydraulic Neutrality[3]

The modelling demonstrates negligible downstream flood level changes (generally less than 10 mm) despite a minor increase in peak discharge of approximately 0.03 m³/s. Hydraulic neutrality has narrowly not been achieved when assessed solely by comparing the peak discharge hydrographs, as the proposed peak flow is approximately 0.03 m³/s higher than the existing peak. However, the hydraulic model extends downstream to the Mazengarb Stream and represents the wider receiving environment. The modelling indicates that the

[2] Memo dated 09.10.26 referenced "Summerset Paraparaumu: Flood Hazard along Development Boundary" attached in Appendix 2 of this report

[3] This assessment has been conducted considering all culverts operational excluding the internal primary network

proposed development results in no increase or only negligible increases in downstream flood levels, less than 10 mm compared with the existing scenario. Please refer Section 4.3.

The wetlands provide sufficient storage to contain the 100-year design event without surcharging. The outlet structures restrict the discharge rate from the site and limit the potential for increased flows to the downstream receiving environment.

The minor increase in peak discharge is likely associated with external overland flow intercepted at the site boundary and conveyed along the boundary before discharging to the stream, rather than the direct discharge from the proposed wetlands. The timing and concentration of this diverted external flow may result in the small increase observed in the downstream hydrograph. Figure 16 shows the relevant external flow paths as yellow dashed lines for reference.

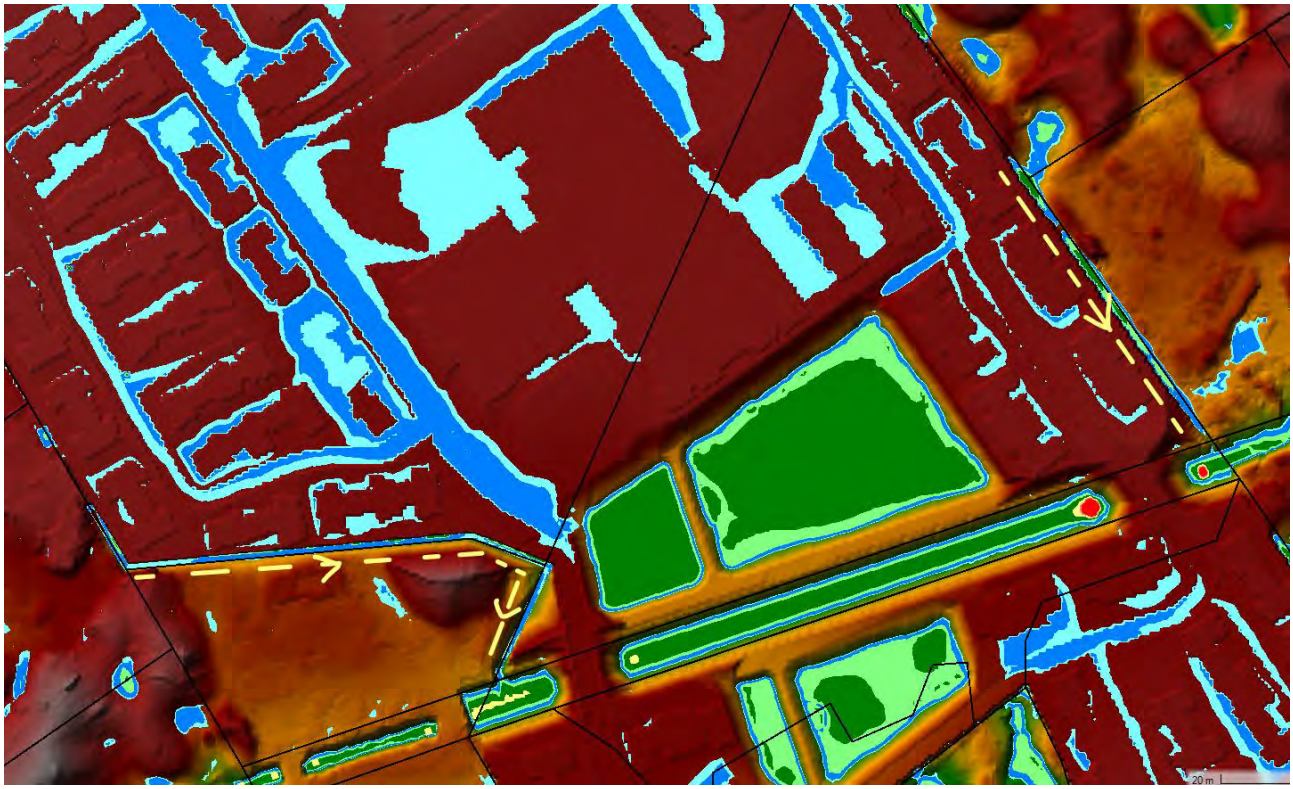



*Figure 16: External flow paths entering the stream*

The purpose of hydraulic neutrality is to avoid or minimise adverse downstream flood effects. Although the proposed peak discharge is marginally higher than the existing peak, the wetland storage, controlled outlets and modelled flood-level results demonstrate that the proposed development does not materially worsen downstream flooding. On this basis, the practical intent of hydraulic neutrality is considered to be met. The following figures compare the existing and proposed peak discharges at selected downstream locations.

In the diagrams dark blue colour represents the proposed scenario where the light blue colour represents the existing scenario. Profile Location is shown in a red arrow.

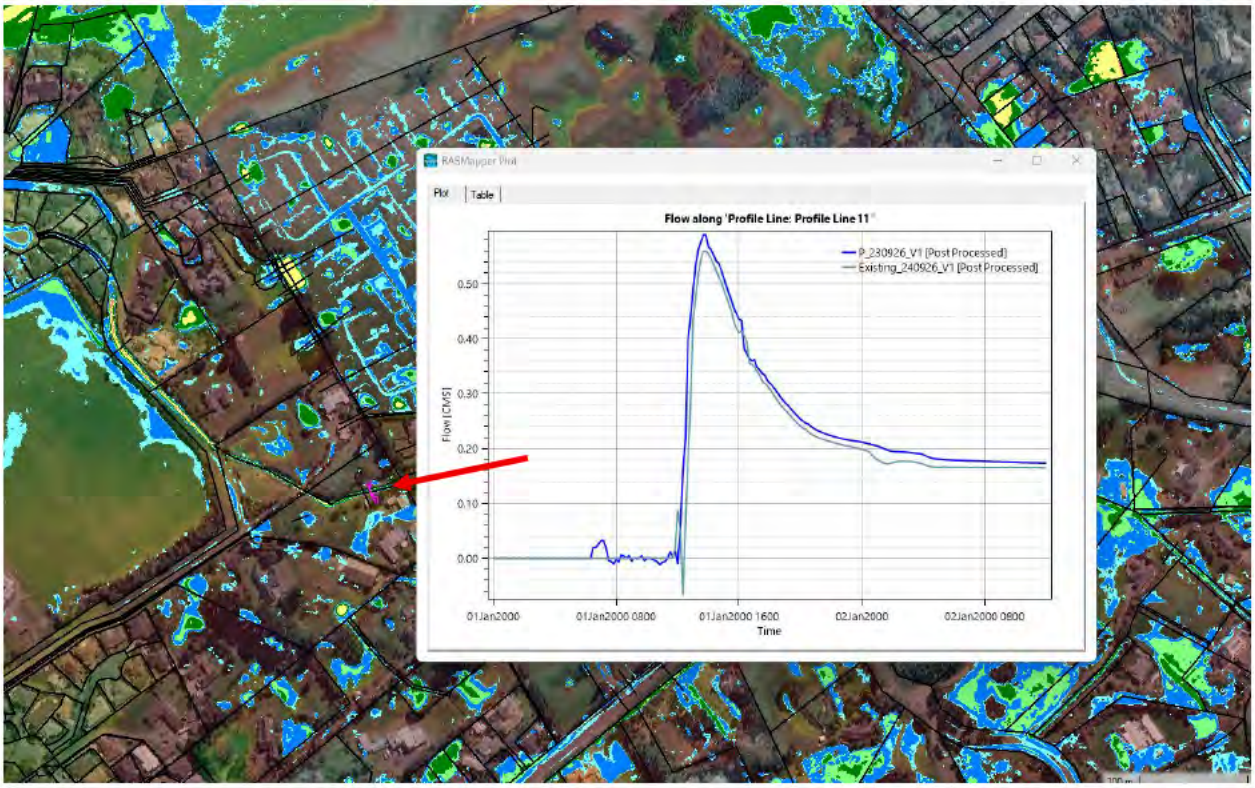



*Figure 17: Existing (light blue) and proposed (dark blue) flows following the flow attenuation immediately downstream of the site- 100yrCC*

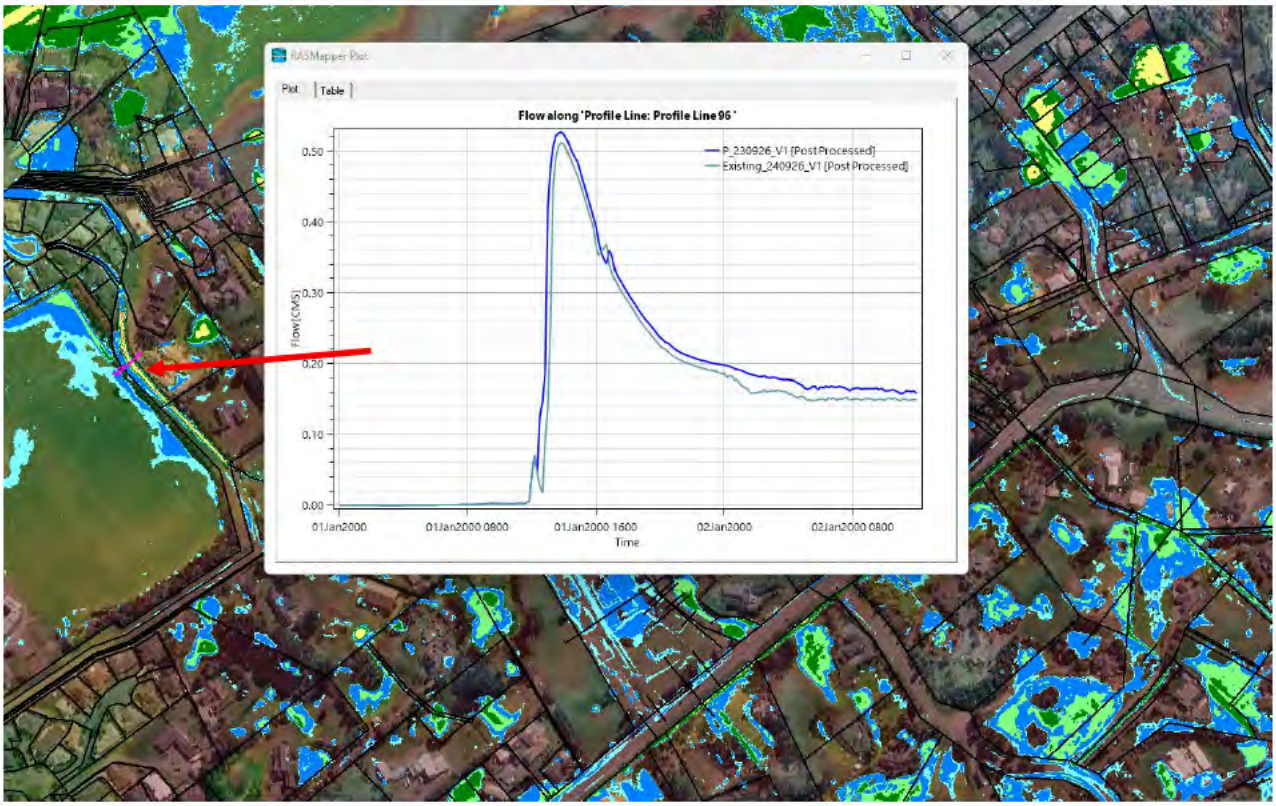



*Figure 18: Existing (light blue) and proposed (dark blue) flows following the flow attenuation at the inlet location to Mazengarb stream- 100yrCC*

### 4.3.3 Sensitivity Assessment

A sensitivity assessment was undertaken with 25% blockage applied to the western runoff conveyance culverts shown in the figure below. The function of these culverts and their role in flood mitigation are described in Appendix 2. The proposed culverts are 1500mm diameter culverts, much bigger than the modelled 100yr peak flow through them, however they are big to ensure minimal blockage. They will remain private culverts managed by the retirement village with operation and maintenance manuals provided in the Stormwater Management Assessment, to ensure they stay clear of debris. However, a 25% blockage scenario was undertaken as a sensitively check in case of blockages prior to large rainfall events.

All other culverts are considered operational in the modelling given their large size.

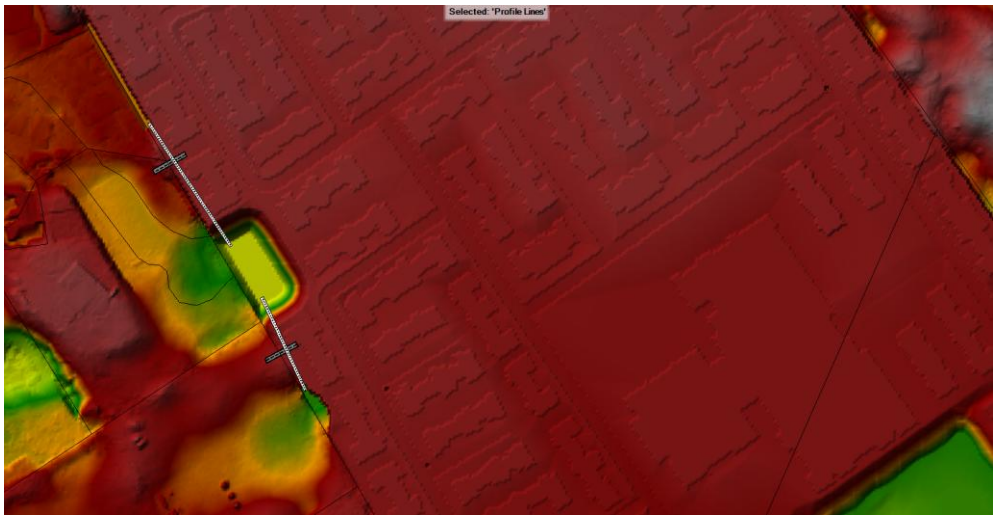



*Figure 19: Western secondary culverts*

The modelling results indicate that the culverts retain more than sufficient capacity to convey the 100yr overland peak flow under the assessed blockage condition and continue to provide the intended flood mitigation. The flood-level difference map below illustrates the effects of this scenario.

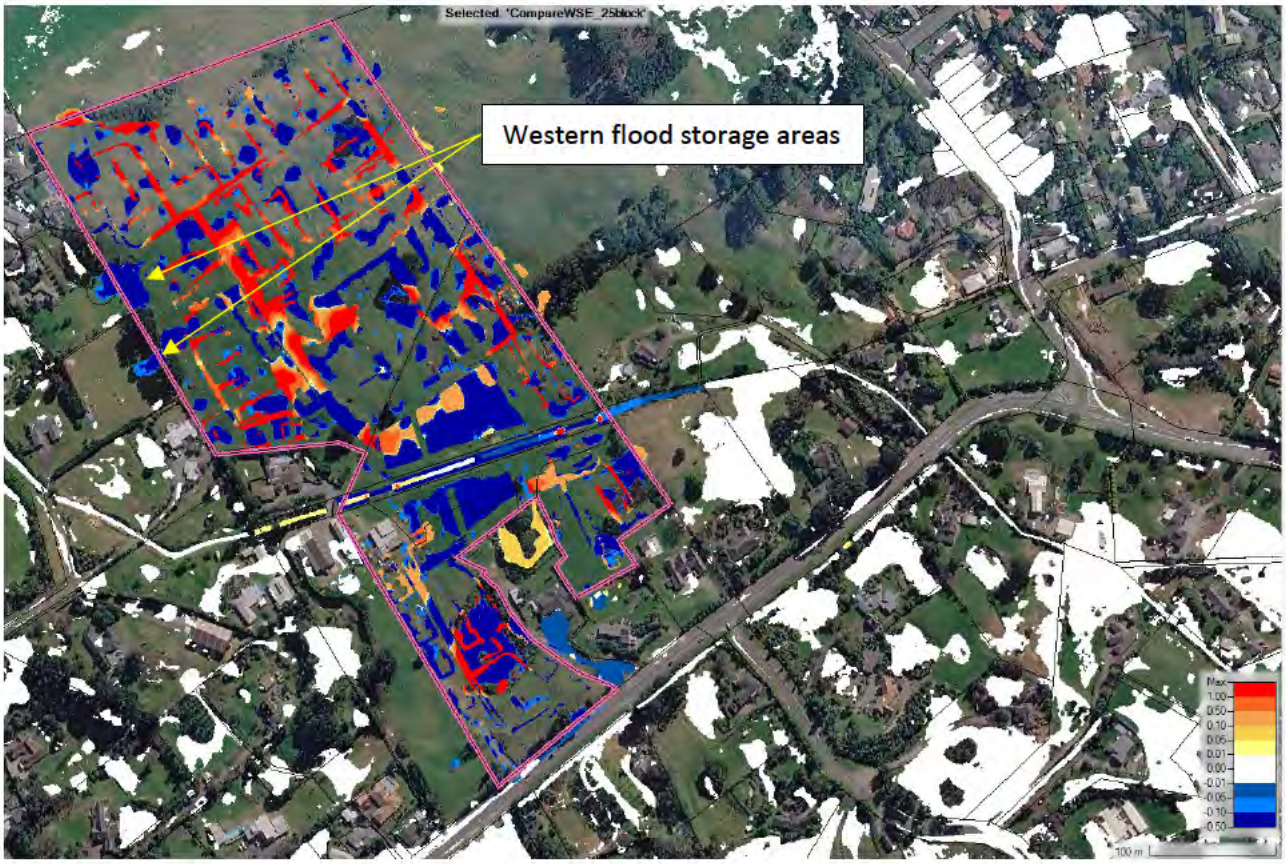



*Figure 20: Flood level difference map considering 25% blockage on western culvert*

### 4.3.4 Resilience Assessment

The 500-year ARI event was modelled to assess the availability of safe egress during an event more extreme than the 100-year ARI event, recognising the importance of safe evacuation from the retirement village. The assessment focused on the two proposed road crossings that provide an egress route from the northern development, through the southern platform, to Ratanui Road. Please see 500yr ARI flood extents at the road crossing locations shown below.

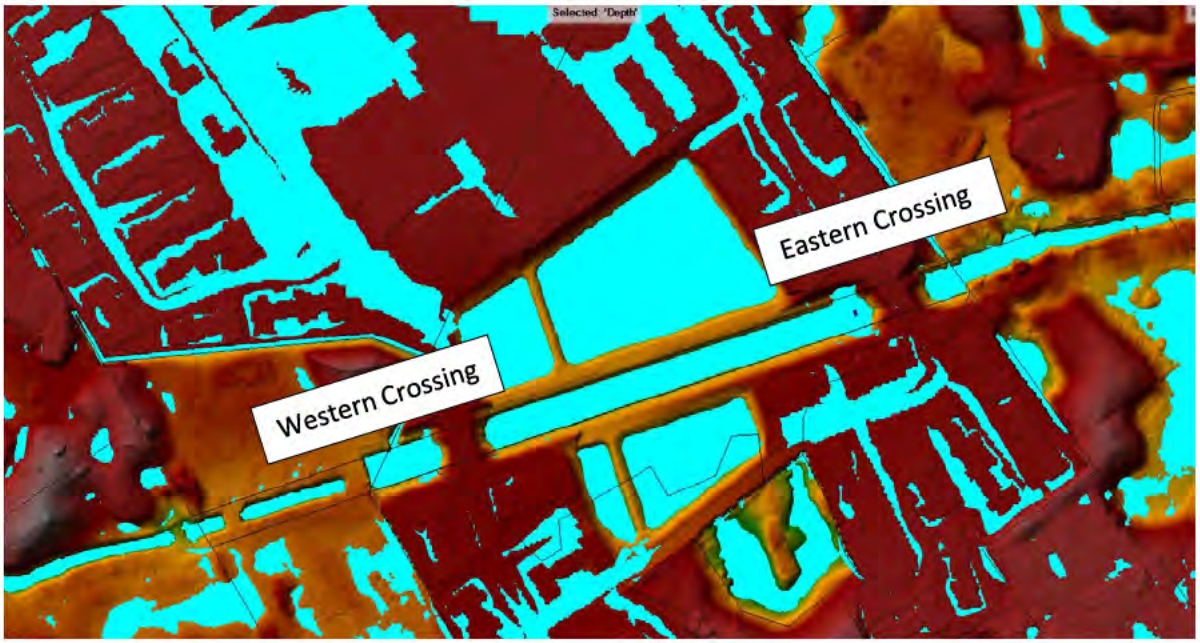



*Figure 21: 500yr ARI flood extents*

The modelled flood levels at the two crossings were compared with the corresponding road crest levels to assess available freeboard. The results are summarised below.

*Table 1: Available freeboard to the road crossing in 500yr ARI event.*

| Road Crossing | 500yr ARI Flood Level m R.L (NZVD2016) | Road Crossing Crest Level m R.L (NZVD2016) | Available Freeboard (m) |
|---|---|---|---|
| **Eastern** | 5 | 6.8 | 1.8 |
| **Western** | 5 | 6.5 | 1.5 |

# 5 Conclusion

A flood risk assessment has been undertaken for the proposed retirement village development at 65 and 73 Ratanui Road, Paraparaumu, to assess potential flood effects on the site, neighbouring properties and downstream receiving environment. The assessment considered the 1% AEP rainfall event, including climate change, with the proposed design progressively refined to minimise adverse flood effects.

The hydraulic modelling and assessment demonstrate that:

- **The proposed flood attenuation systems operate effectively**, with sufficient capacity to accommodate the 1% AEP event without surcharging and controlled outlets limiting downstream discharges.
- **Hydraulic neutrality has effectively been achieved**, with a minor increase in peak discharge of approximately 0.03 $m^3/s$ but negligible downstream flood-level changes, generally less than 10 mm.
- **Downstream flood effects are negligible**, with no material increase in flood levels or flood risk within the downstream receiving environment.
- **Residual boundary effects are localised and acceptable**, with mitigation incorporated where practicable and remaining effects assessed as low relative to existing flood risk.
- **Safe egress is maintained during the 1% AEP event**, with the resilience assessment for the 500-year ARI event confirming that safe egress is also maintained during that event.

Overall, the assessment demonstrates that the practical intent of hydraulic neutrality has been achieved and that the proposed development will not result in material adverse flood effects on surrounding properties or the downstream receiving environment. The proposal is therefore considered appropriate from a flood risk perspective.

# 6 Limitations

This report has been prepared for our client in accordance with the agreed brief and for submission in support of a Fast-track application for consideration by an expert consenting panel. It may not be relied upon in any other context or for any other purpose without CKL's express written approval. Except for its intended use in the Fast-track consenting process, neither CKL nor any employee or sub-consultant accepts responsibility for reliance on this report, in whole or in part, by any other person or entity. Making this report available to Council or other parties for consent, approval or statutory purposes does not extend its intended scope or purpose.

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# FRA Appendix 1:  Hydraulic Modelling Summary

| | | | |
|---|---|---|---|
| **Job number:** | A25556 | | |
| **Job description:** | Flood modelling Summary_ Summerset Paraparaumu | | |
| **Calculations by:** | SK | **Date:** | 09.10.26 |
| **Checked by:** | ZW | **Date:** | 19.10.26 |

# 1 Aim

- The hydraulic model has been developed to assess the hydraulic response of the proposed development during the considered design events, particularly the 100-year ARI event. The assessment is undertaken to demonstrate hydraulic neutrality and evaluate any upstream/ downstream effects in accordance with the requirements of Kapiti Coast District Council (KCDC) and Greater Wellington Regional Council (GWRC).

# 2 Methodology

### 1. Hydrological Modelling

A two-dimensional (2D) rain-on-grid hydraulic model was developed, with the model domain covering the proposed development and the surrounding residential area.

The model incorporates upstream catchment inflows as point source hydrographs within the 2D domain. A stage hydrograph has been used as the downstream boundary condition. The hydraulic assessment adopts the inflow and stage hydrographs from the existing flood model developed by AWA on behalf of Kapiti Coast District Council (KCDC).

### 2. Hydraulic Modelling

The 2D model geometry was developed in HEC-RAS and includes the following components:

- Terrain
- 2D flow area (model domain)
- Manning's roughness coefficients
- Infiltration parameters
- Boundary Conditions

### 3. Scenario Modelling

Two scenarios have been assessed as follows.

TABLE 1: MODELLING SCENARIOS

| Scenario | Geometry |
|---|---|
| Existing Condition | Use existing ground levels and land use across development site |
| Proposed Condition | Use proposed ground levels and land use across development site |

existing scenario represents the pre-development conditions, while the proposed scenario represents the post-development conditions.

### 4. Design Events

100yr and 500yr ARI design event have been used to assess the flood conditions. All rainfall data corresponding to each event have been adjusted for climate change using RCP 8.5 2081-2100 approach in line with AWA model.

# 3 Model Build - Hydraulics

## 3.1 Software

The Hydrologic Engineering Centre's River Analysis System (HEC-RAS) developed by the U.S. Army Corps of Engineers (version 6.5) has been used for hydraulic modelling.

The link to the model project is below:

12DSynergy\data\CKL-AZU-SYN-1\A25556_29126\31 Enviro\00 Modelling\HEC RAS

## 3.2 Naming Conventions

Corresponding HEC RAS files for each scenario are saved under following naming convention.

TABLE 2: NAMING CONVENTIONS FOR EXISTING AND PROPOSED SCENARIOS

| Component | Existing Condition | Proposed Condition |
|---|---|---|
| 100yr (Basis for Impact Assessment and hydraulic Neutrality) | | |
| Terrain | Existing LiDAR | Design 230926 |
| Geometry Title | Proposed V35[1]_ Existing | Proposed V36 |
| Flow Title | 100yr_V1_AWA[2] | 100yr_V1_AWA |
| Plan Title | Existing 240926 | Proposed 230926 |
| Plan Short ID | Existing_240926_V1 | P_230926_V1 |

## 3.3 Geometry

### 3.3.1 Terrain

Existing and proposed terrain consists of following characteristics.

TABLE 3: TERRAIN DETAILS

| Terrain Information | Existing | Proposed |
|---|---|---|
| Source | LiDAR 1m DEM downloaded from LINZ | Proposed design surface by CKL merged with the existing LiDAR |
| Datum | Horizontal: NZTM 2000 | |
| | Vertical: NZVD2016 | |

[1] Proposed 2D mesh has been retained in both existing and proposed conditions, with other parameters varying across two scenarios.

[2] Proposed rainfall and flow conditions have been used across both scenarios for consistency.

### 3.3.2 2D Flow Area

A two-dimensional flow area encompassing the site and downstream area was established using a base grid size of 5 × 5 m. Break lines were applied where required, with finer cell sizes as applicable.

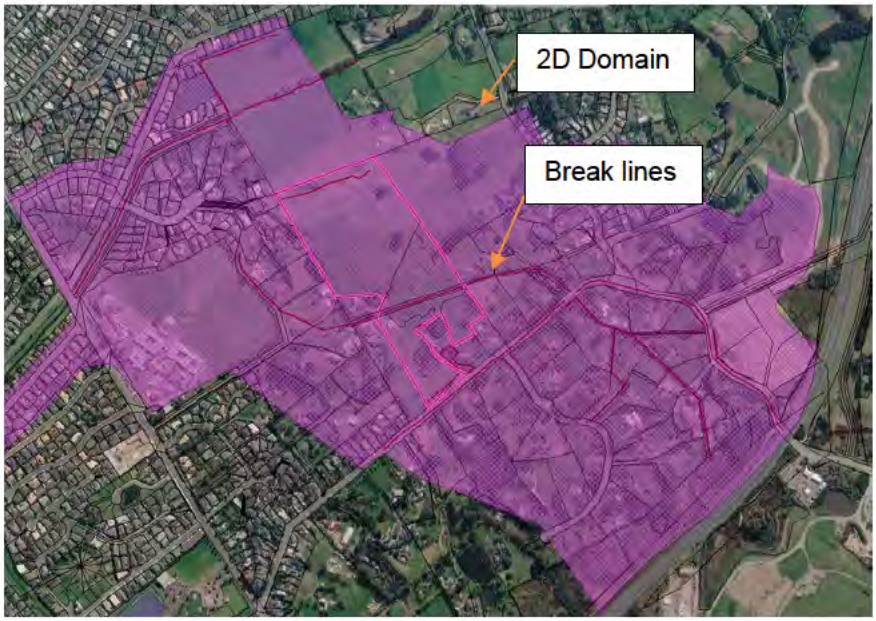



FIGURE 1: 2D FLOW AREA

### 3.3.3 Boundary Conditions

An upstream flow hydrograph, a downstream stage hydrograph, a precipitation boundary applied across the 2D model domain, and six normal depth boundary conditions were used. The locations of the boundary conditions are shown in Figure 2.

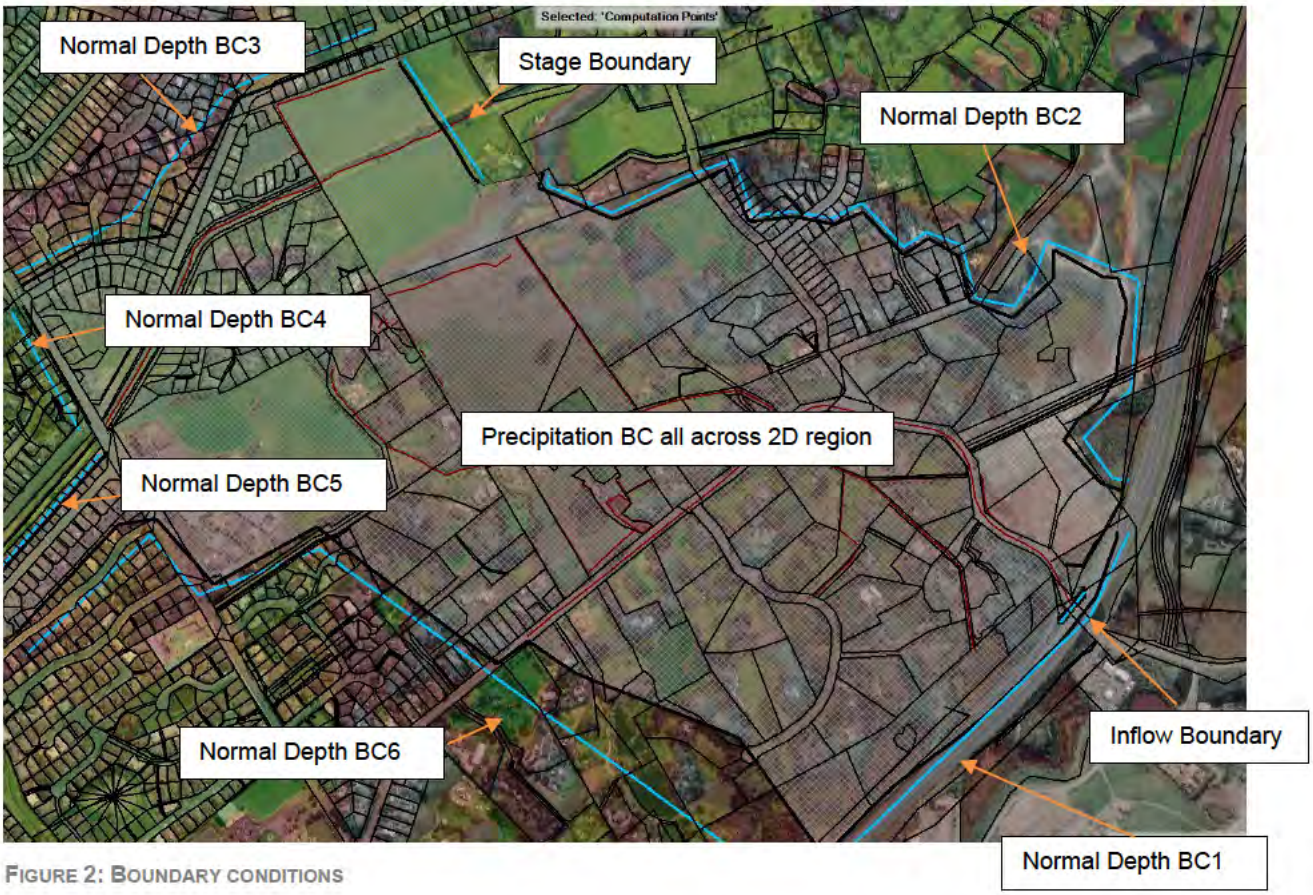



FIGURE 2: BOUNDARY CONDITIONS

### *3.3.3.1 Precipitation Boundary Conditions*

A nested rainfall distribution, as recommended in the *Reference Guide for Design Storm Hydrology* developed by Cardno NZ Ltd., was used to develop the rainfall hyetograph for the design event using the corresponding design rainfall depth. The resulting hyetograph was applied as a precipitation boundary condition across the 2D model domain.

Design rainfall depth was obtained from HIRDS v4, adopting the RCP 8.5 climate change scenario for the 2081–2100 period. A 36-hour storm duration with a 5-minute temporal interval was adopted, consistent with the existing AWA model.

Corresponding hyetograph for 100yr and 500yr ARI events are shown in Figure 3 and 4 respectively.

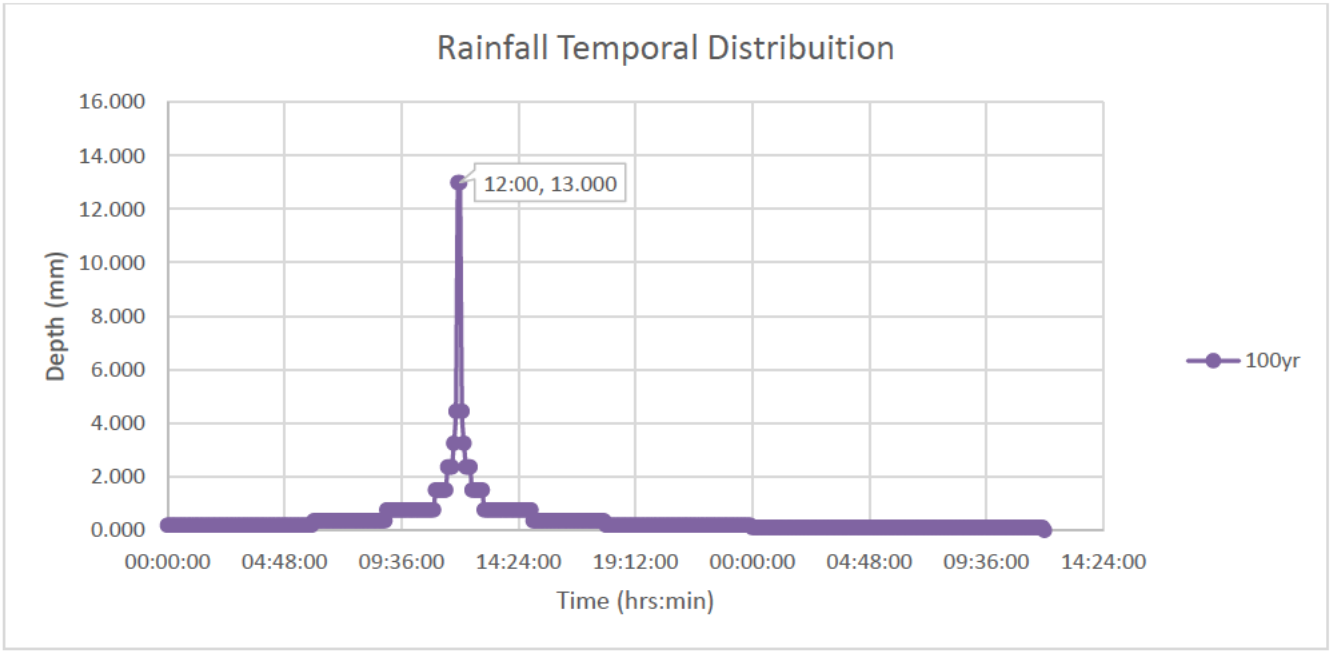



FIGURE 3: RAINFALL PATTERN 100YRCC DESIGN EVENT

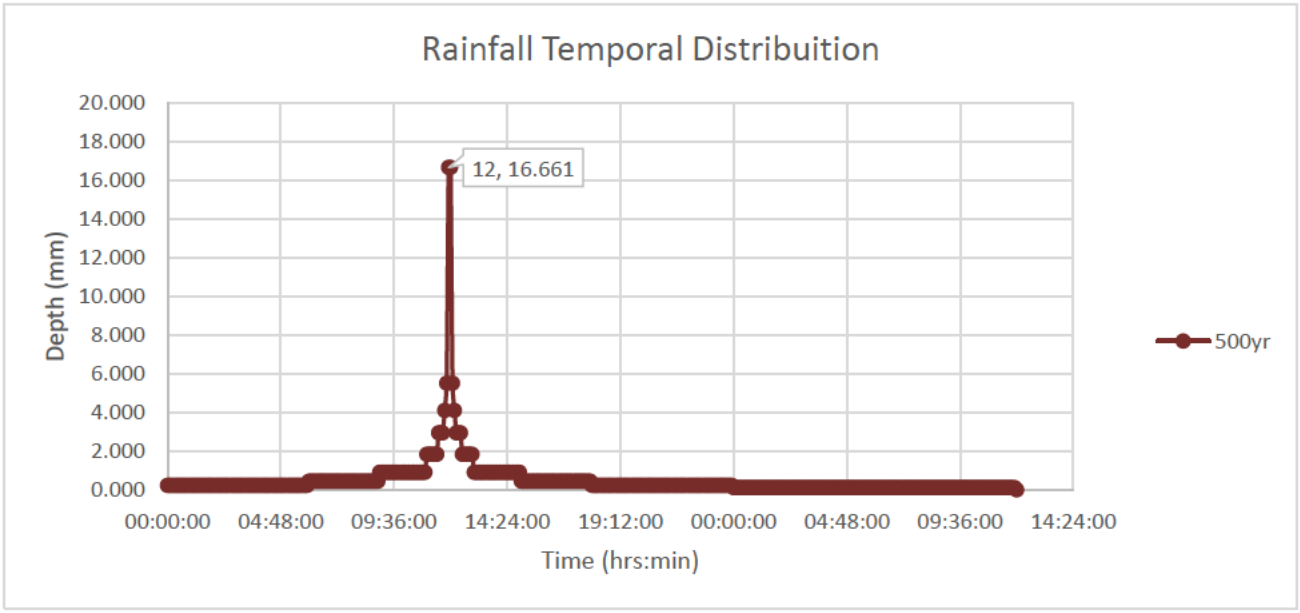



FIGURE 4: RAINFALL PATTERN 500YRCC DESIGN EVENT

500yr rainfall distribution has been obtained using the same temporal pattern by Cardno NZ Ltd corresponding to 500yr rainfall depths derived from NIWA HIRDS V4.

Following method has been used to extrapolate 500 yr rainfall depths.

The 500-year ARI rainfall depths for the existing climate were first derived by extrapolating the HIRDSv4 depth–duration–frequency equation (Equation 3 of High Intensity Rainfall Design System Version 4, August 2018) using the site-specific coefficients. Equation and corresponding site-specific parameters are shown below.

$$\ln R = cxy + dx + ex^2 + fx^3 + gy + hy^2 + i \qquad (3)$$

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| HIRDS V4 Depth-Duration-Frequency Results | | | | | | | | |
| Sitename: Summerset Paraparaumu | | | | | | | | |
| Coordinate system: WGS84 | | | | | | | | |
| Longitude: 175.0121 | | | | | | | | |
| Latitude: -40.8865 | | | | | | | | |
| DDF Model | Parameters: | c | d | e | f | g | h | i |
| | Values: | -0.00535922 | 0.44796609 | -0.00621015 | -0.00310054 | 0.24275222 | -0.00912228 | 2.794358 |
| | Example: | Duration (hrs) | ARI (yrs) | x | y | Rainfall Depth (mm) | | |
| | | 24 | 500 | 3.17805383 | 6.213607264 | 165.0269813 | | |

FIGURE 5: EQUATION AND COEFFICIENTS USED TO EXTRAPOLATE 500 YEAR RAINFALL DEPTHS

The resulting depths were then adjusted for RCP8.5 for 2081–2100 using the duration-specific augmentation factors in Table 6 and the projected temperature increase of 2.58°C in Table 8 of the HIRDSv4 Technical Report. Accordingly following depths are obtained.

FIGURE 6: 500YRCC RAINFALL DEPTHS

| Duration | Historical 500-year depth (mm) | Increase per °C | Total increase | RCP8.5 2081–2100 depth (mm) |
|---|---|---|---|---|
| 10 minutes | 24.7 | 13.6% | 35.09% | **33.3** |
| 20 minutes | 32.8 | 13.6% | 35.09% | **44.4** |
| 30 minutes | 38.9 | 13.6% | 35.09% | **52.6** |
| 1 hour | 52.0 | 13.6% | 35.09% | **70.2** |
| 2 hours | 69.0 | 13.1% | 33.80% | **92.3** |
| 6 hours | 105.2 | 11.5% | 29.67% | **136.4** |
| 12 hours | 133.6 | 10.1% | 26.06% | **168.4** |
| **24 hours** | **165.0** | **8.6%** | **22.19%** | **201.6** |
| 48 hours | 196.9 | 7.5% | 19.35% | **235.0** |
| 72 hours | 214.4 | 6.9% | 17.80% | **252.6** |
| 96 hours | 225.7 | 6.5% | 16.77% | **263.5** |
| 120 hours | 233.5 | 6.1% | 15.74% | **270.2** |

### *3.3.3.2 Upstream Boundary Conditions - Flow Hydrograph*

CKL obtained the upstream flow hydrographs for the 100-year design event from the existing Council model developed by AWA Environmental.

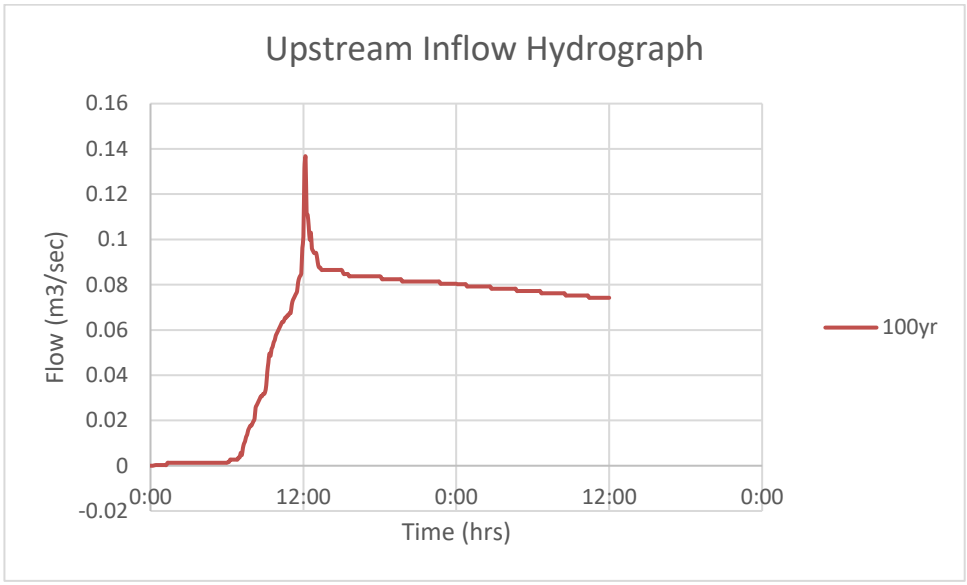



FIGURE 7: FLOW HYDROGRAPH 100YRCC DESIGN EVENT

The 100-year flow hydrograph was scaled based on the ratio of the 500-year rainfall depth to the 100-year rainfall depth. For example, the 500-year flow hydrograph was calculated as follows:

500-year flow hydrograph = AWA 100-year flow hydrograph × (500-year rainfall depth / 100-year rainfall depth)

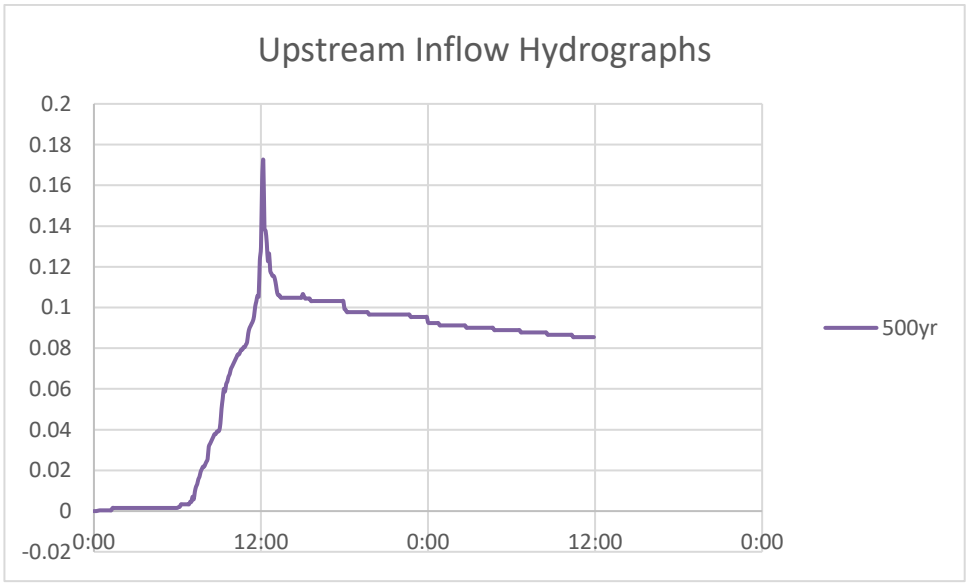



FIGURE 8: FLOW HYDROGRAPH 100YRCC DESIGN EVENT

### *3.3.3.3 Downstream Boundary Condition -Stage Boundary*

A stage hydrograph at the location shown in Figure 2 was obtained from AWA Environmental for the 100-year design event. The stage data were provided relative to the Wellington Vertical Datum 1953 (WVD53) and were converted to the New Zealand Vertical Datum 2016 (NZVD2016) by applying an offset of -0.355 m.

Corresponding stage hydrographs are shown in Figure 9.

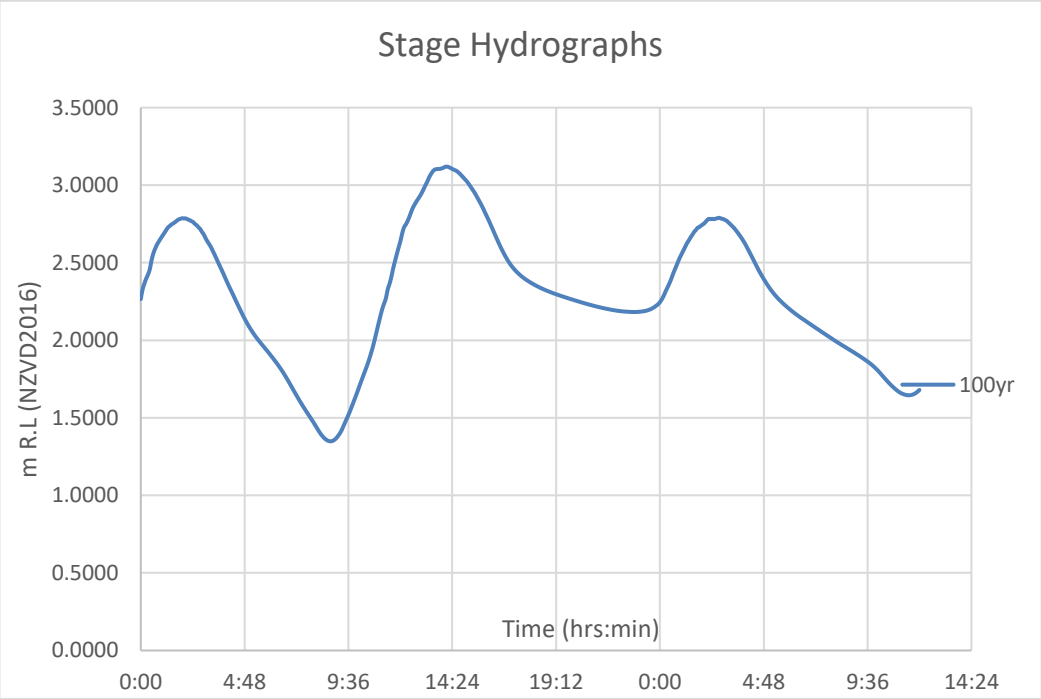



FIGURE 9: STAGE HYDROGRAPHS CORRESPONDING TO 100YRCC EVENT

For the 500-year design events, the downstream stage hydrographs was derived from the 10-year and 100-year AWA stage hydrographs using linear extrapolation based on the corresponding design peak flows. At each time step, the difference between the 10-year and 100-year stage hydrographs was scaled by the relative increase in design peak flow and added to the 10-year stage hydrograph.

For example, the 500-year stage hydrograph was calculated using the following relationship:

$R = (Q500 - Q10) / (Q100 - Q10)$

$H500 = H10 + R \times (H100 - H10)$

where Q10, Q100 and Q500 are the peak design flows for the 10-year, 100-year and 500-year events, respectively, and H10, H100 and H500 are the corresponding stage hydrograph values at each time step.

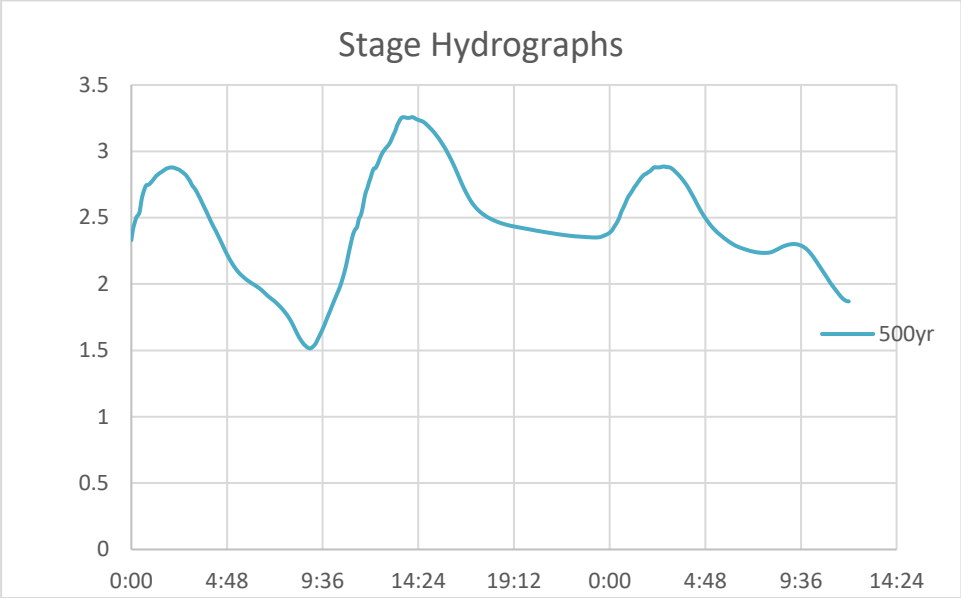



FIGURE 10: STAGE HYDROGRAPHS CORRESPONDING TO 100YRCC EVENT

It is to be noted that the Waikanae river level at 500yr ARI design event has not been explicitly assessed within the scope of this assessment. These levels might affect the 500yr model results, as the Waikanae river levels might differ from what has been assessed using interpolated tailwater conditions.

#### *3.3.3.4 Downstream Boundary Condition -Normal Depth*

Six normal depth boundary conditions were defined as per Figure 2 with following normal depths.

TABLE 4: NORMAL DEPTHS

| Boundary Condition (refer figure 2) | Normal Depth |
|---|---|
| BC Line 1 | 0.10 |
| BC Line 2 | 0.10 |
| BC Line 3 | 0.01 |
| BC Line 4 | 0.01 |
| BC Line 5 | 0.07 |
| BC Line 6 | 0.01 |

### 3.3.4 Mannings Roughness

Following Manning's Roughness values have been used for both existing and proposed scenarios with calibration regions defined to represent proposed changes in land use.

TABLE 5: MANNING'S ROUGHNESS VALUES BASED ON LAND USE

| Land Cover / Surface Type | Manning's n |
|---|---|
| Forest | 0.100 |
| Impervious | 0.020 |
| Scrub Flax | 0.200 |

| | |
|---|---|
| Wetland | 0.050 |
| Pasture-Cr | 0.030 |
| Water | 0.025 |
| Alpine tus | 0.100 |
| Bare | 0.023 |
| Urban Open | 0.030 |

Variation of hydraulic roughness between ED and MPD scenarios are shown in Figure 11 and Figure 12 respectively.




FIGURE 11: VARIATION OF MANNINGS ROUGHNESS-EXISTING

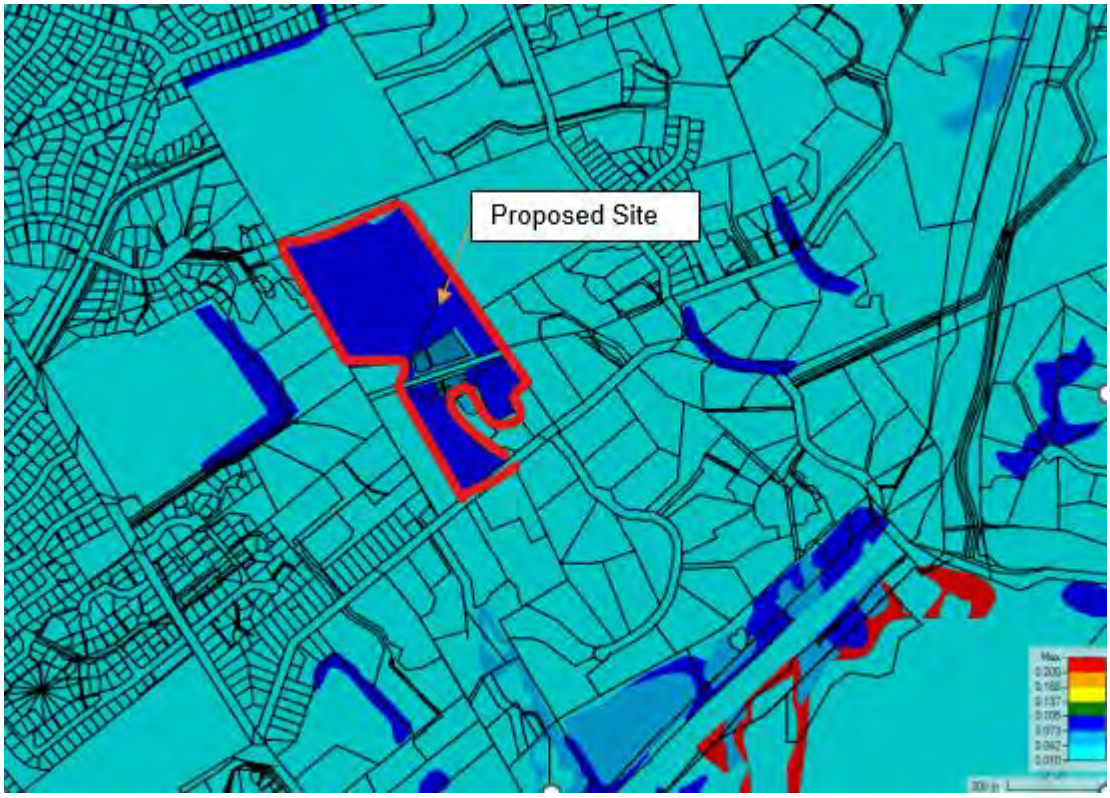



FIGURE 12: VARIATION OF MANNINGS ROUGHNESS-PROPOSED

### 3.3.5 Infiltration Parameters

The infiltration parameters adopted for the HEC-RAS model were based on the initial and continuing loss parameters from the existing AWA TUFLOW model. The TUFLOW initial loss was adopted as both the HEC-RAS initial and maximum deficit, while the continuing loss rate was adopted as the potential percolation rate. This parameterisation was adopted to provide a comparable representation of the AWA initial/continuing loss approach for the design storm simulations, rather than as a physically based representation of soil properties.

The adopted potential percolation rates range from 0 to 400 mm/hr, while the initial and maximum deficits range from 5 to 20 mm. Due to the spatial variability and large number of individual polygons, the adopted infiltration parameters are presented spatially in Figure 13 rather than tabulated individually.

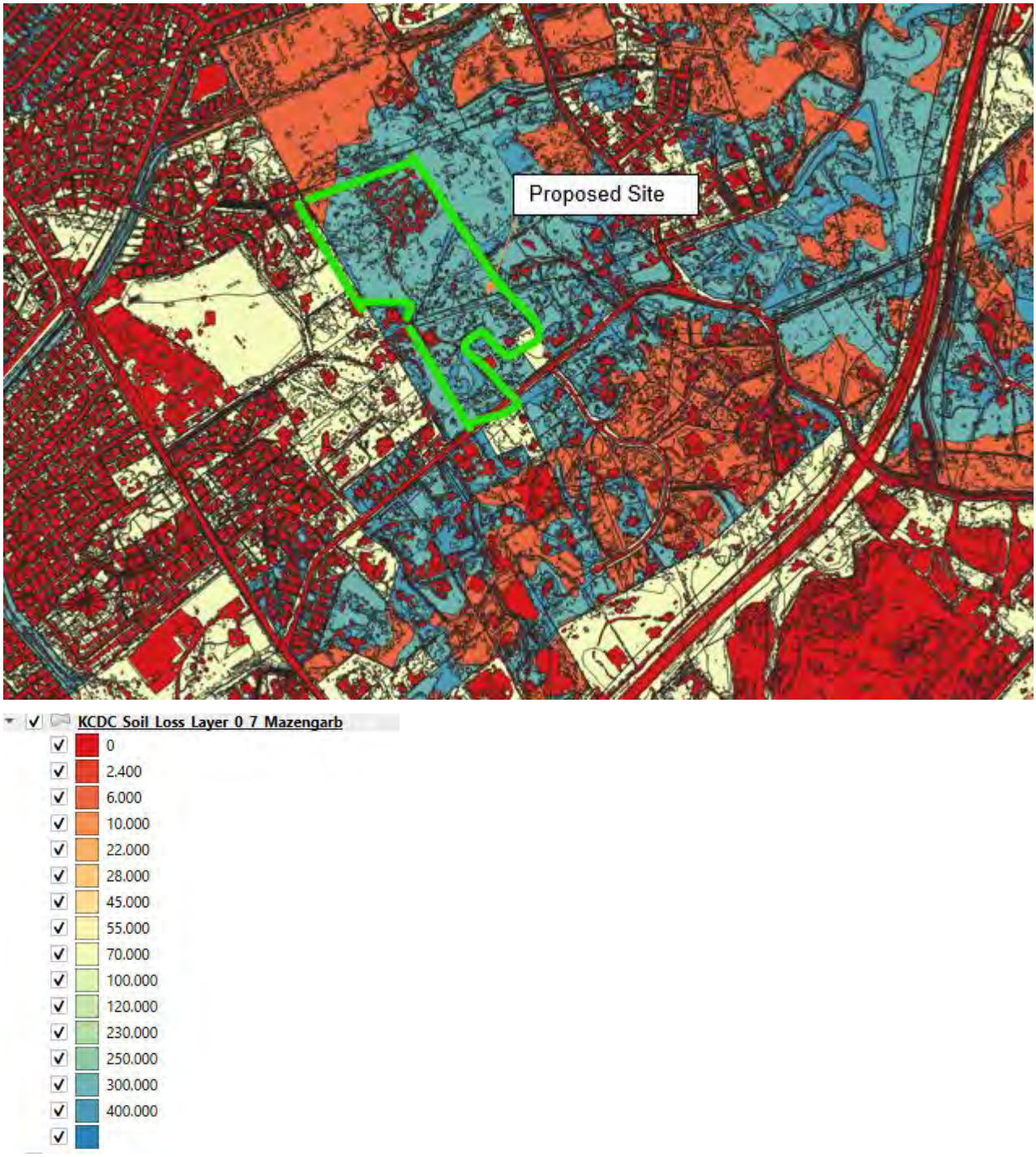



FIGURE 13 SPATIAL VARIATION OF THE PERCOLATION RATE IN MM/HR

### 3.3.6 Culverts-Existing

Several culverts located upstream, downstream, and within the site have been incorporated into the hydraulic model.

Where survey information was unavailable, culvert dimensions were assumed based on upstream structures, and invert levels were approximated using the existing ground surface. Culverts must be surveyed accurately for detailed design purposes to finalize the flood modelling.

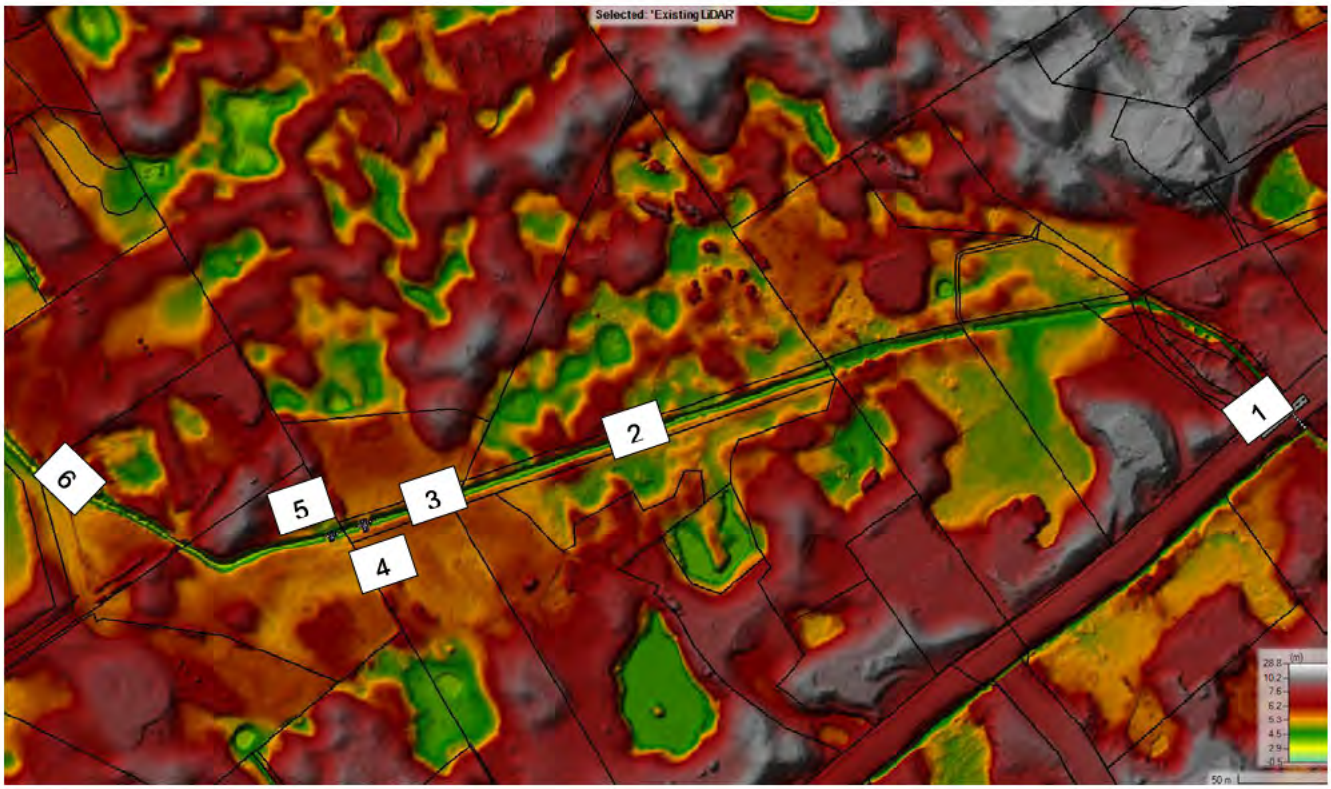



FIGURE 14: CULVERTS

Following parameters have been used in the model.

TABLE 6: PARAMETERS USED IN MODELLING CULVERTS AND BRIDGES

| Location ID (as per figure 14) | Size (m diameter) | Upstream Invert_NZVD2016 | Downstream Invert_ NZVD2016 | Source |
|---|---|---|---|---|
| **1** | 1.05 | 4.8 | 4.6 | Assumed |
| **2** | 1.05 | 3.44 | 3.33 | AWA Model |
| **3** | 1.05 | 3.22 | 3.25 | AWA Model |
| **4** | 1.05 | 3.51 | 3.24 | Surveyed |
| **5** | 1.05 | 3.05 | 3.28 | Surveyed |
| **6** | 1.05 | 2.25 | 2.15 | AWA Model |

### 3.3.7 Culverts-Proposed

Existing Culverts 1, 3, 4, 5 and 6 were retained under proposed conditions. Six additional culverts were incorporated:

- two serving as outlets from the northern and southern attenuation basins -100mm Ø
- two at road crossings over the mainstream -2m x 2m box culverts
- two along western site boundary – 1.5m Ø

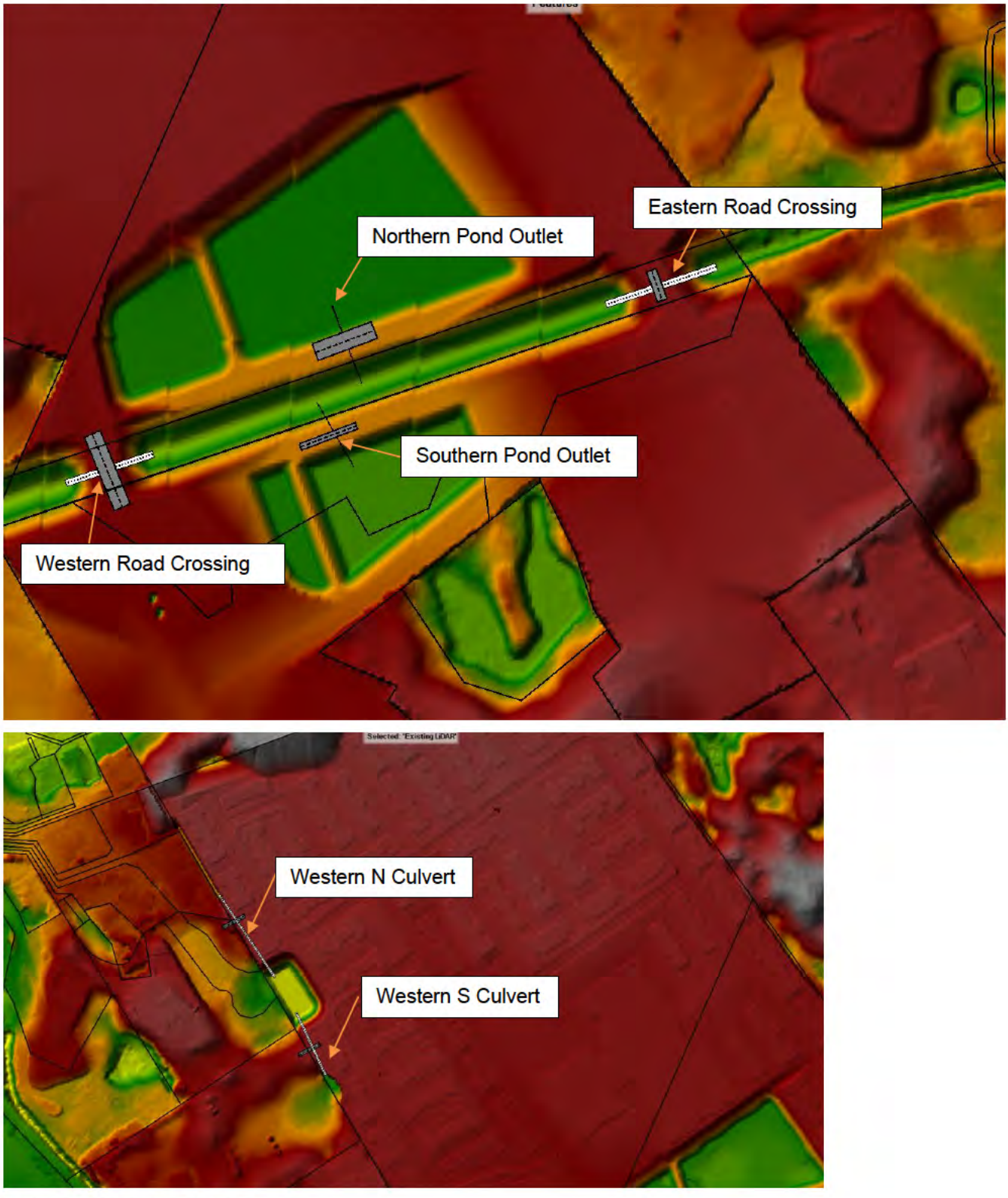


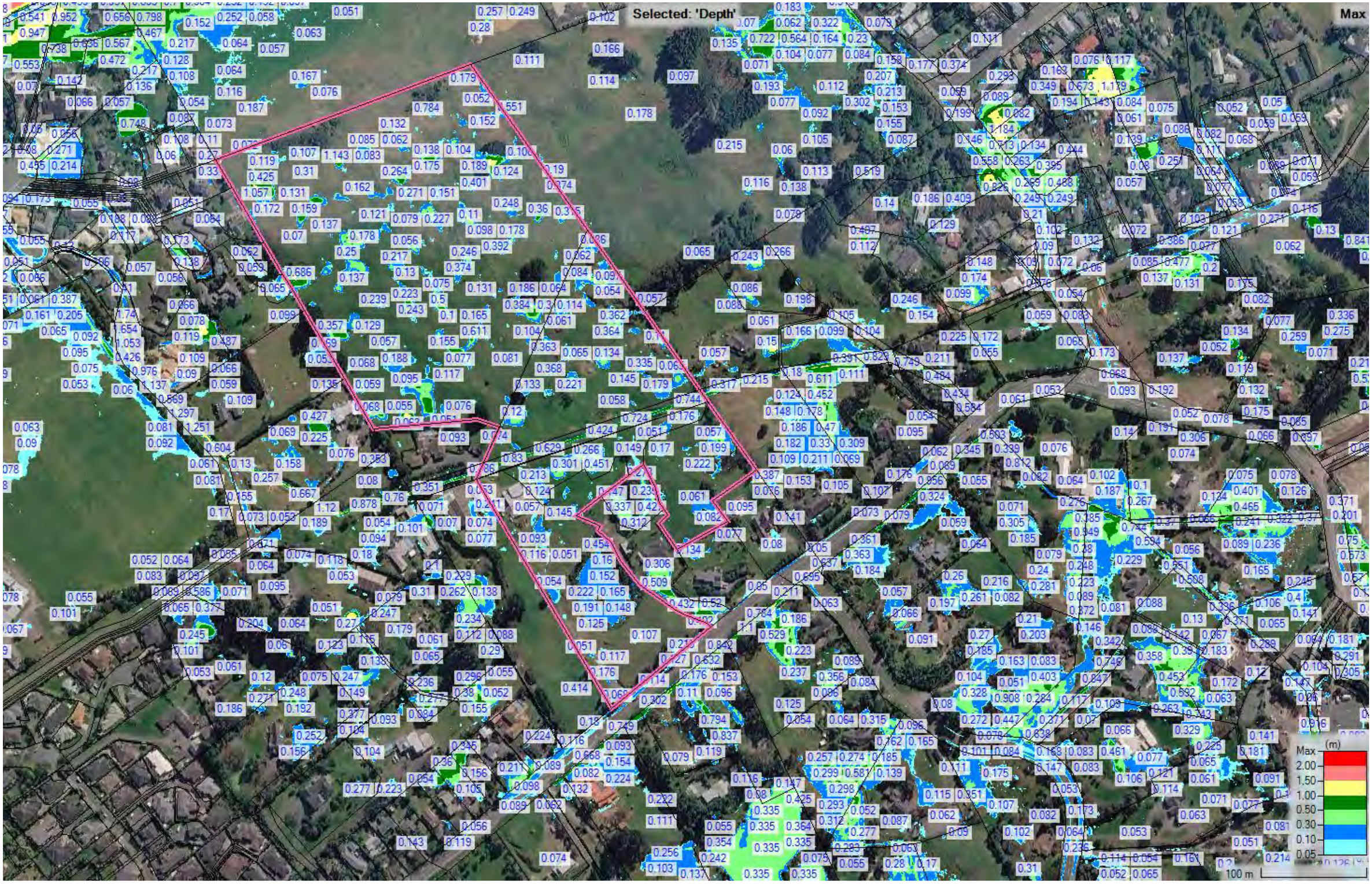

FIGURE 15: PROPOSED CULVERTS

TABLE 7: PARAMETERS USED IN MODELLING CULVERTS AND BRIDGES

| Location ID (as per figure 14) | Size | Upstream Invert_NZVD2016 | Downstream Invert_ NZVD2016 |
|---|---|---|---|
| **Northern Pond Outlet** | 0.1m diameter | 4.3 | 4.25 |
| **Southern Pond Outlet** | 0.1 m diameter | 4.3 | 4.25 |
| **Eastern Road crossing** | 2m x 2m box culvert | 3.4 | 3.35 |
| **Western Road Crossing** | 2m x 2m box culvert | 3.14 | 3.13 |
| **Western N Culvert** | 1.5 m diameter | 5.45 | 3 |
| **Western S Culvert** | 1.5 m diameter | 4.25 | 3 |

## 3.4 Model Results

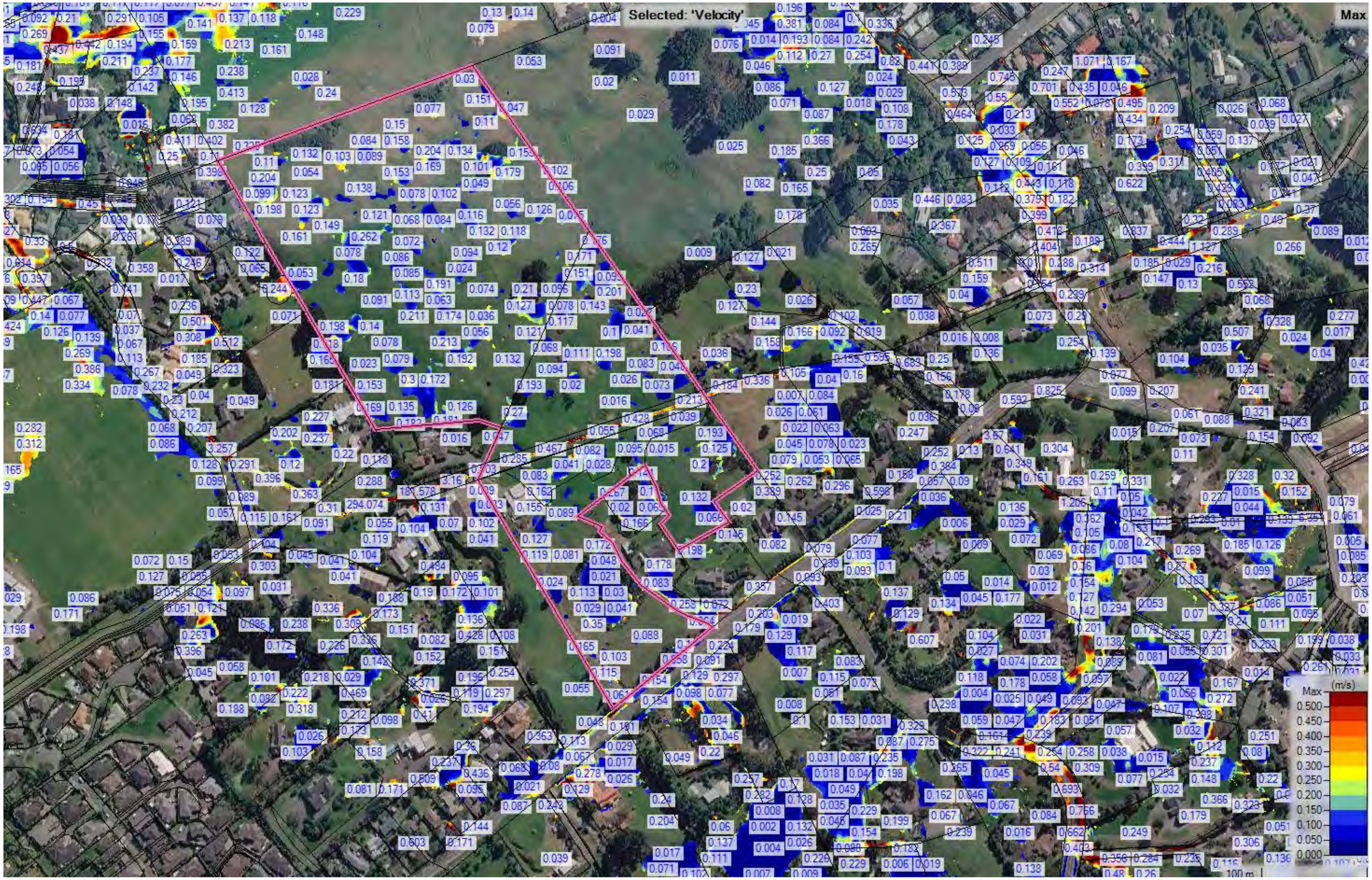
**100yrCC- Existing Scenario- Flood Depths (m)**



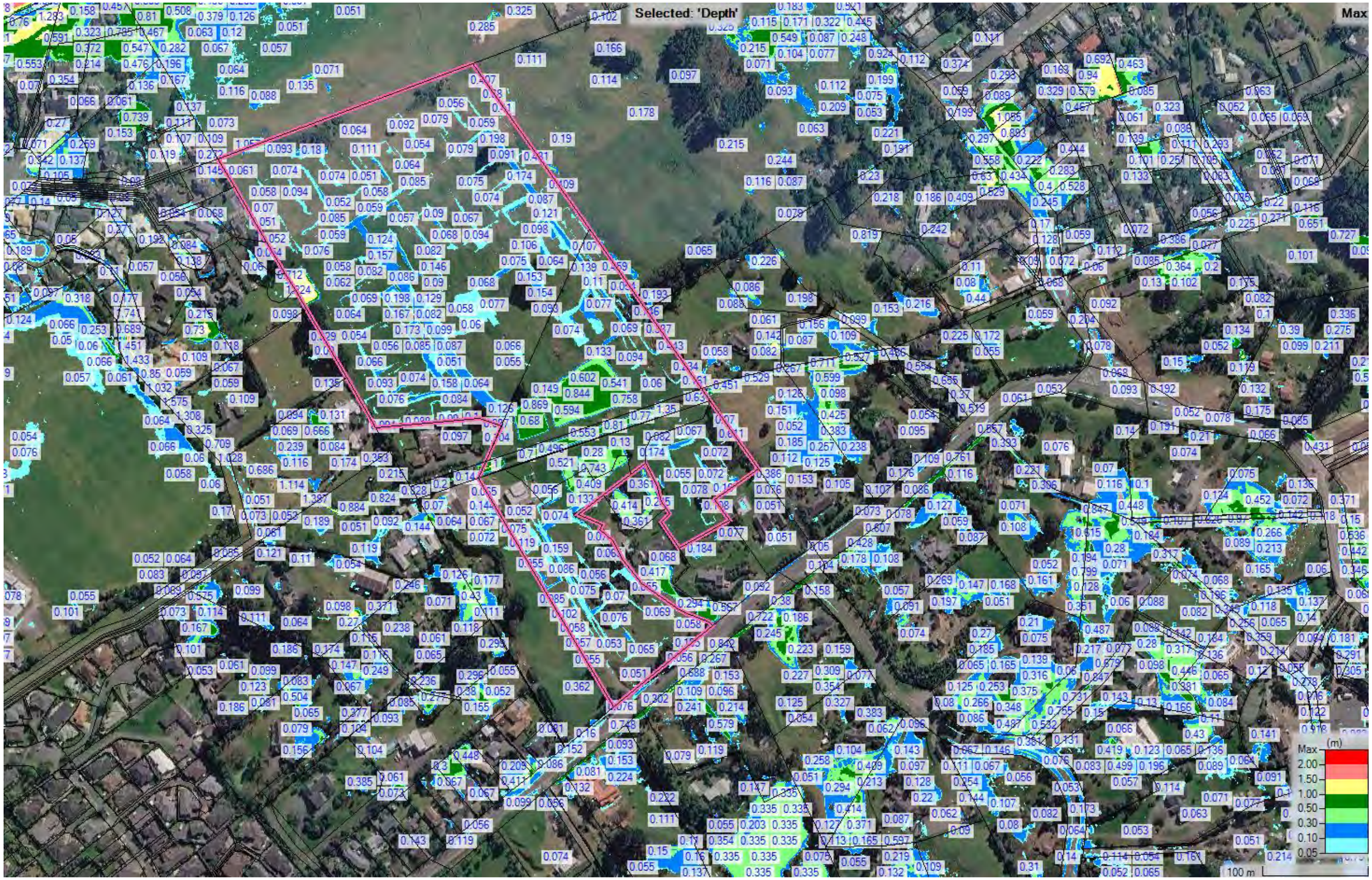
**100rCC- Existing Scenario- Flood Levels (m R.L NZVD 2016)**



**100rCC- Existing Scenario- Flood Velocities (m/sec)**

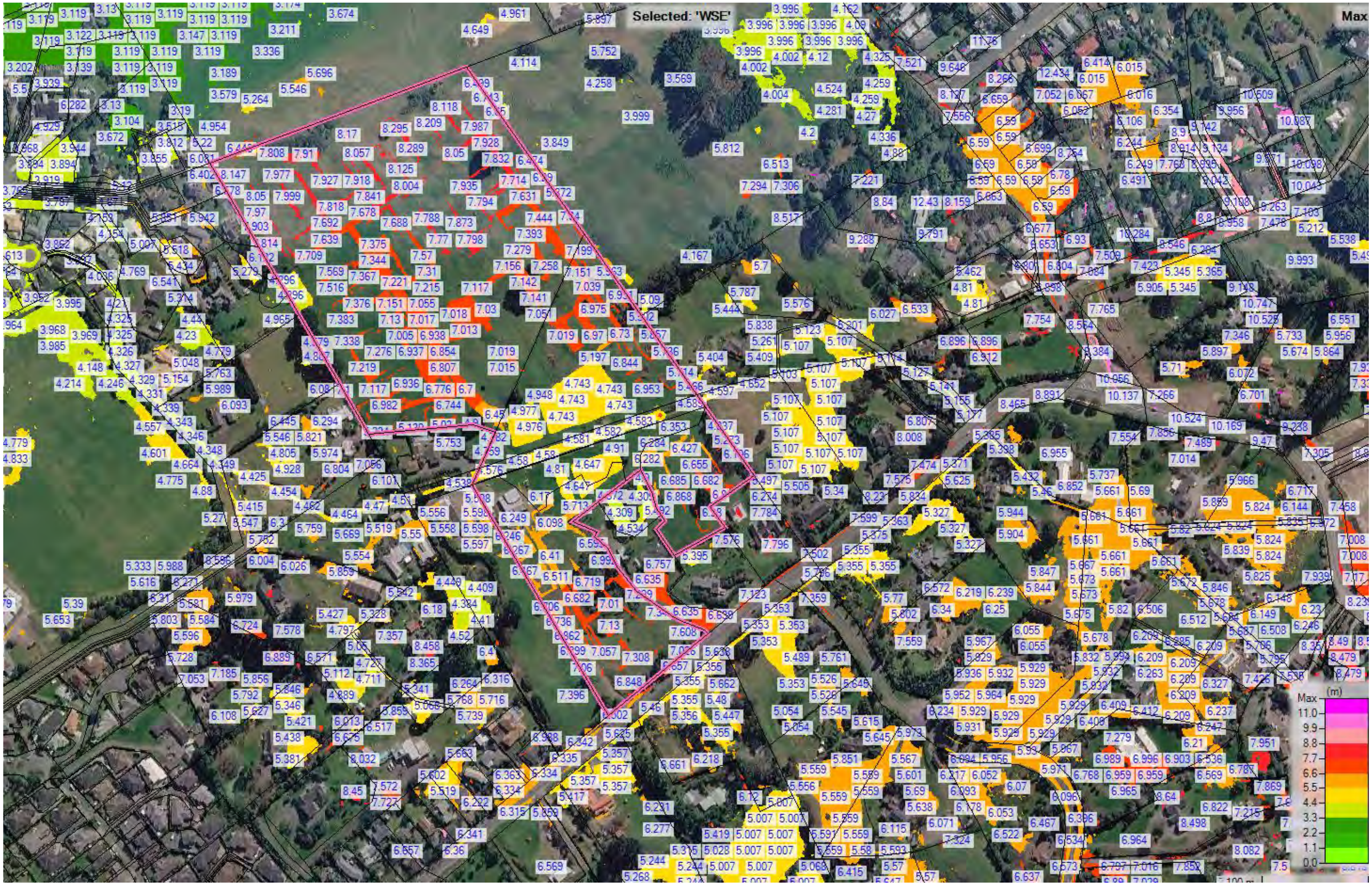


**100yrCC- Proposed Scenario[3]- Flood Depths (m)**

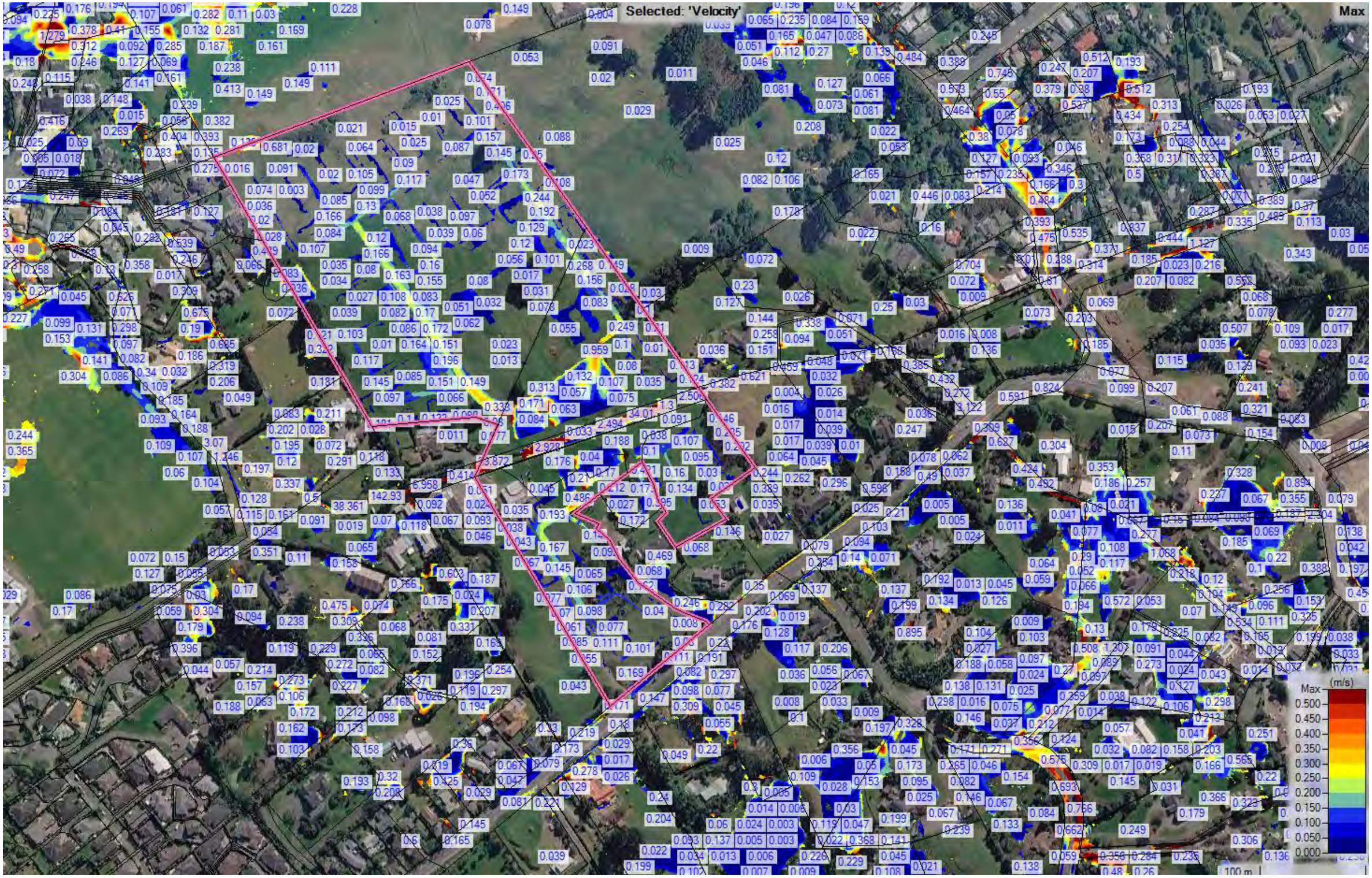



[3] Proposed surface dated 23.09.26

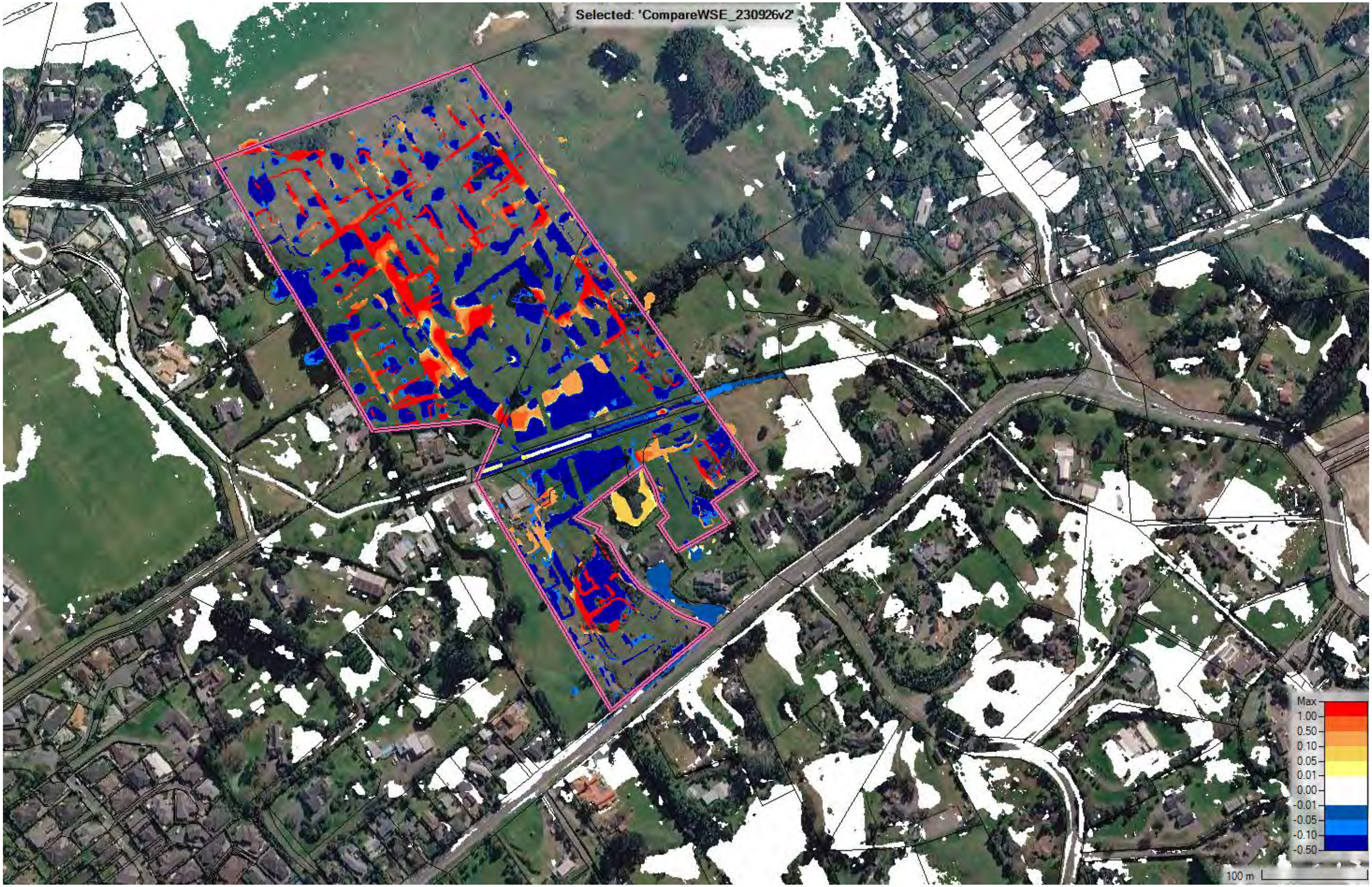
**100rCC- Proposed Scenario- Flood Levels (m R.L NZVD 2016)**



**100rCC- Proposed Scenario- Flood Velocities (m/sec)**



**100yrCC- Flood level Difference Map- Proposed vs Existing**



# FRA Appendix 2:  Boundary Flood Hazard Memo

# MEMO

Planning | Surveying | Engineering | Environmental

| | | | |
|---|---|---|---|
| **To:** | **Summerset Holdings** | **Date:** | **09.10.26** |
| **From:** | **Sarala Karunarathna (Environmental Engineer)** | **CC:** | |
| **Reviewed:** | **Frances Deamer-Phillips (Senior Environmental Engineer)** | **CKL Ref:** | **A25556** |
| **Re:** | **Summerset Paraparaumu: Flood Hazard along Development Boundary** | | |

# 1 Introduction

CKL has been engaged by Summerset Holdings Ltd to undertake a flood risk assessment for the proposed Summerset Retirement Village development at Paraparaumu. The assessment has been prepared to support the concept design by evaluating the hydraulic performance of the proposed development under the 1% AEP (100-year) design flood event and to inform the ongoing design process.

As part of the assessment, several design iterations have been modelled and assessed to achieve an acceptable hydraulic outcome while satisfying the requirements of the Kapiti Coast District Council (KCDC) and Greater Wellington Regional Council (GWRC). The assessment considers the effects of the proposed development on flood behaviour, including upstream and downstream flood levels, flood extents and peak flows, with the objective of avoiding or minimising adverse hydraulic effects.

During the design process, several locations have been identified where flood impacts are difficult to manage due to existing topographical constraints and practical engineering limitations. While a range of mitigation measures have been investigated, further improvements at these locations would require significant intervention that may not be practicable or proportionate to the level of risk.

This memorandum has been prepared to present the hydraulic behaviour at these locations, discuss the constraints associated with implementing additional mitigation measures, and assess whether the proposed development results in any material increase in flood risk compared with the existing condition. The intent is to provide a transparent technical assessment to assist discussions with KCDC and GWRC and to establish an agreed understanding of whether further mitigation is warranted.

The memorandum is structured as follows:

- **Section 2** describes the existing flood conditions.
- **Section 3** presents the predicted flooding under the proposed development and discusses the contributing factors.
- **Section 4** summarises the mitigation measures investigated and explains the engineering constraints that limit further improvements.
- **Section 5** compares the flood risk under the existing and proposed scenarios to determine the flood risk and hazard assessment and whether the proposal results in any material increase in risk.
- **Section 6** provides conclusions and recommendations based on the findings of the risk assessment.

# 2 Existing Site Conditions and Flooding

The existing site is characterised by undulating topography, with a series of natural dune formations creating multiple low-lying depressions within and along the site boundary. These depressions provide localised storage during significant rainfall events, resulting in isolated areas of ponding under the existing condition.

Hydraulic modelling indicates that flood depths within these ponding areas are approximately 0.05 m and 2m during the 1% AEP (100-year) design event. The flooding is largely confined to the natural depressions and is primarily influenced by the site's existing topography, with limited overland flow between the individual ponding areas.

The existing flood extents and depths are illustrated in Figure 1 and Figure 2 respectively.

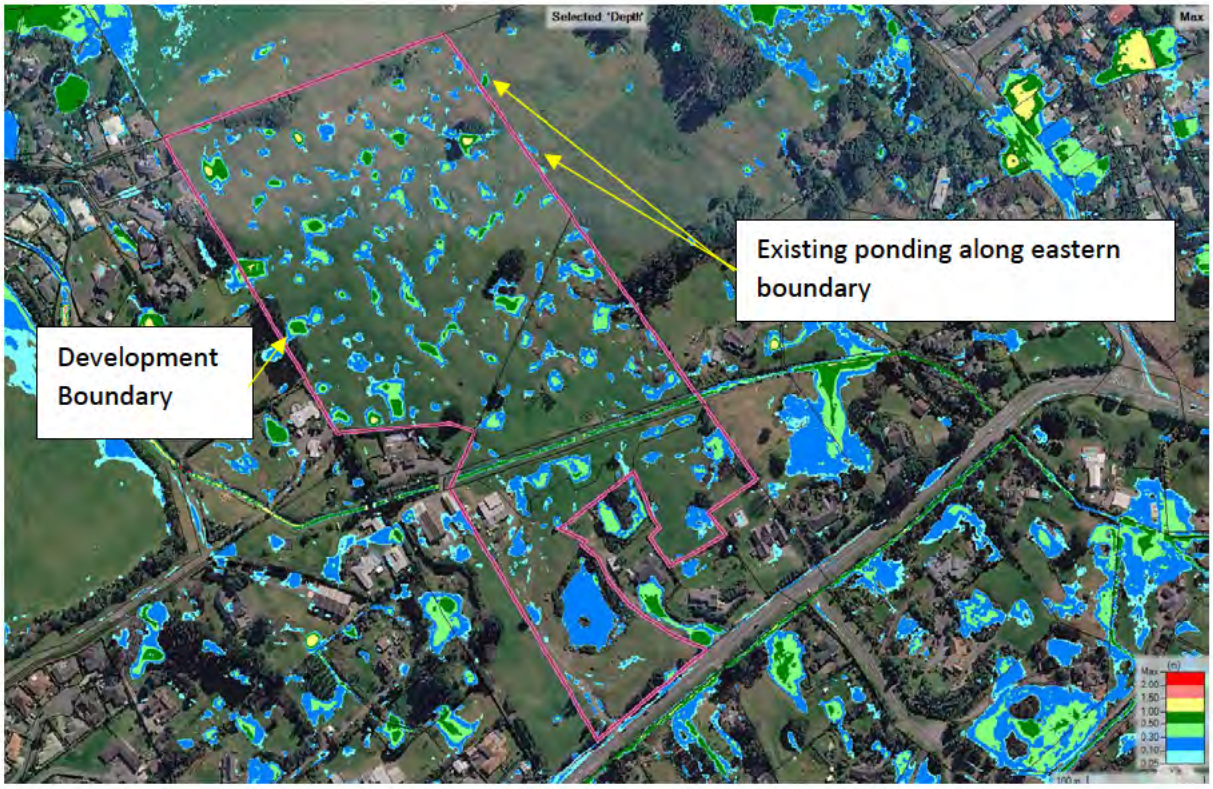



Figure 1: Flood extents under existing conditions - 100yrCC

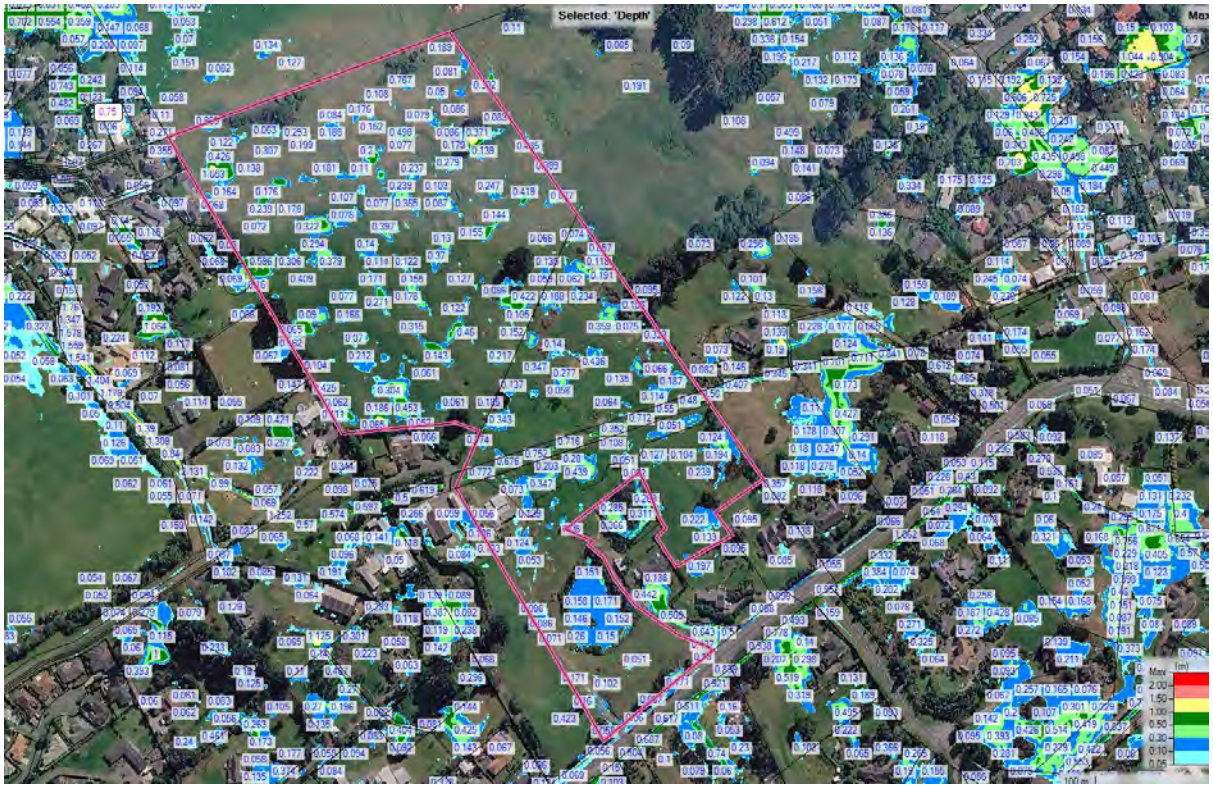



Figure 2: Flood depths under existing conditions - 100yrCC

# 3 Proposed Conditions and Flooding

As part of the assessment, the proposed design surface was incorporated into the hydraulic model to evaluate the flood behaviour under the developed site condition. The modelling reflects the proposed earthworks and finished ground levels associated with the concept design.

The results indicate that localised ponding occurs along the site boundaries where the proposed raised building platforms intercept the existing natural depressions and reduce the available flood storage within these depressions. Consequently, runoff that would naturally occupy these depressions is redistributed, resulting in increased ponding in several low-lying areas adjacent to the site boundary.

The proposed surface design retains most of the displaced flood storage within the site, particularly along the western boundary. However, there are several isolated locations along the eastern boundary where it is not practicable to fully replace the existing depression storage within the development. This is primarily due to the existing undulating topography and the associated engineering constraints.

Under the proposed condition, flood depths within these ponding areas range from 0.05 m to 0.51 m during the 1% AEP (100-year) design event. The predicted flood extents and depths for the proposed scenario are shown in Figure 3 and Figure 4 below.

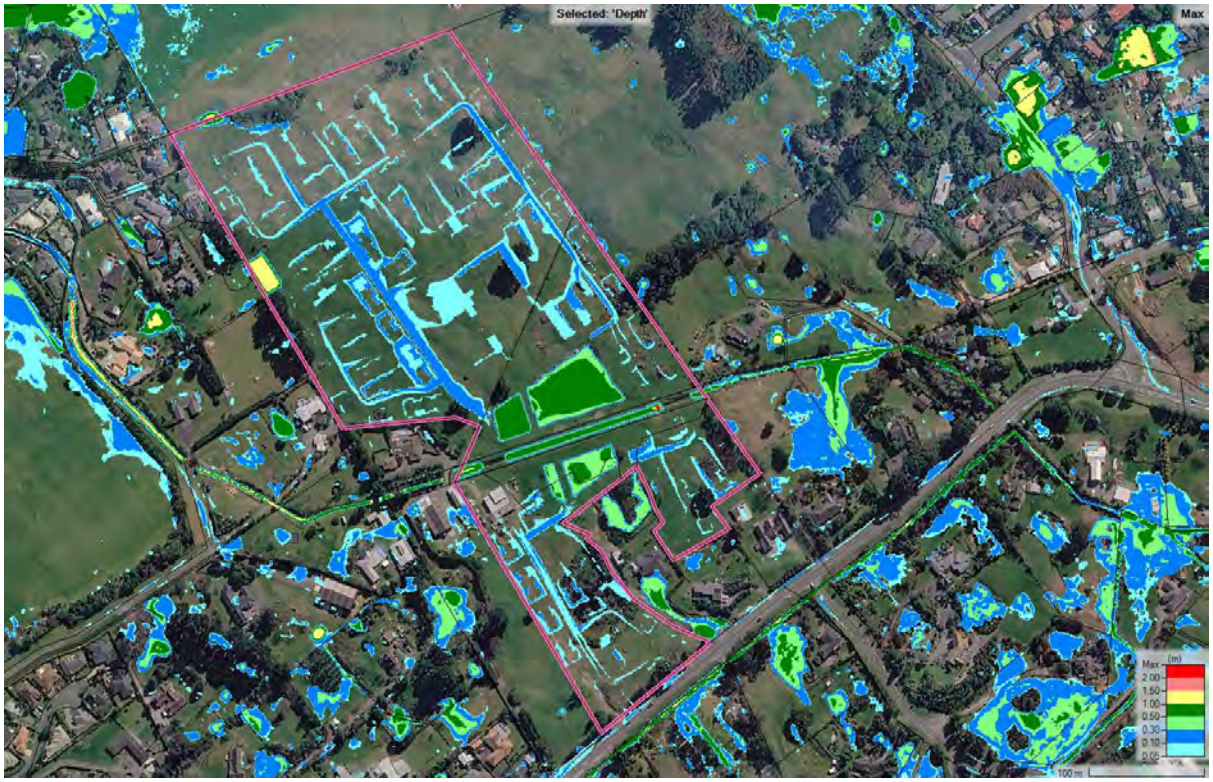



Figure 3: Flood extents under proposed conditions - 100yCCC

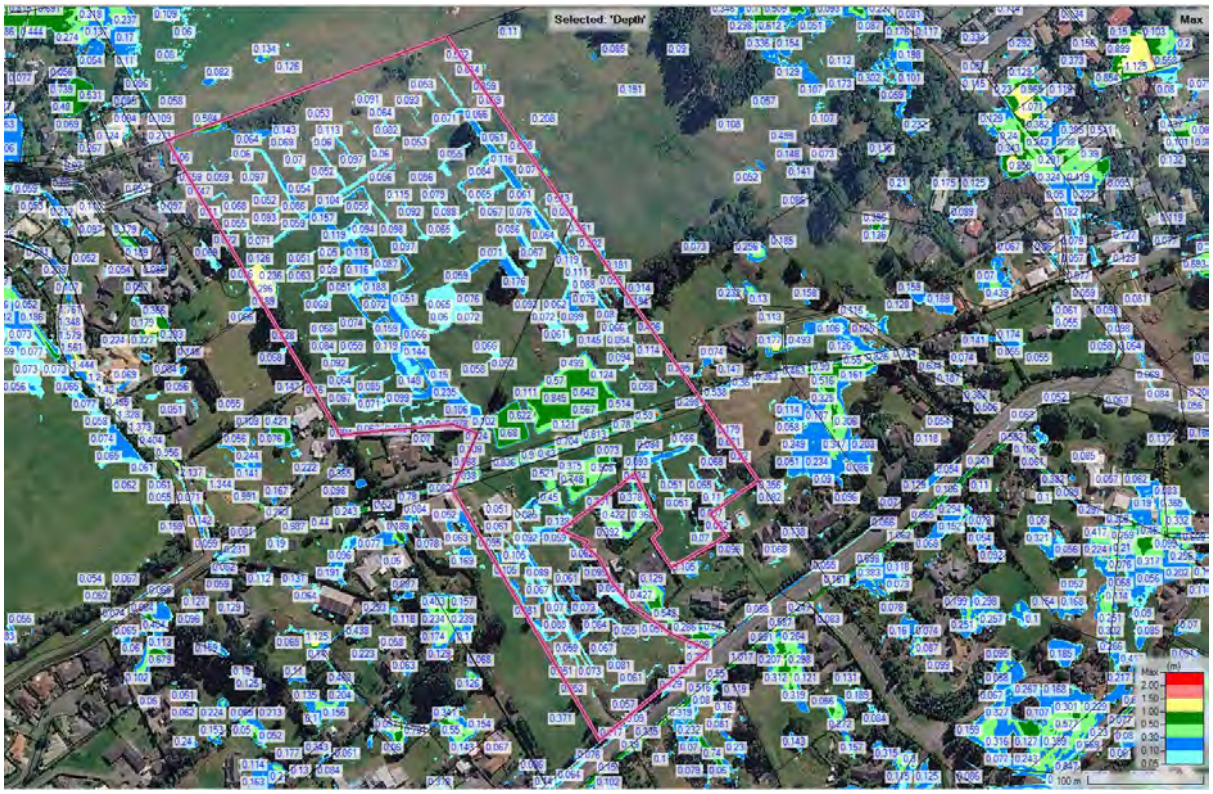



Figure 4: Flood depths under proposed conditions - 100yrCC

# 4 Changes in Flooding During Proposed Conditions

A flood level difference map has been prepared to assess the changes in flood levels between the existing and proposed scenarios. The map highlights areas where flood levels increase or decrease as a result of the proposed development to identify the magnitude and extent of modelled changes.

The increase flood depth is represented by the yellow-shaded areas on the difference map. Conversely, the blue-shaded areas indicate locations where flood levels decrease under the proposed condition.

The overall difference map demonstrates that the proposed surface modifications have successfully retained and redistributed flood storage within the site wherever practicable, particularly along the western boundary.

However, localised increases in flood levels of approximately 100 mm to 300 mm are predicted within the existing depression areas along sections of the eastern site boundary. These increases are confined to the natural low-lying areas where floodwater is redistributed following the proposed ground level modifications.

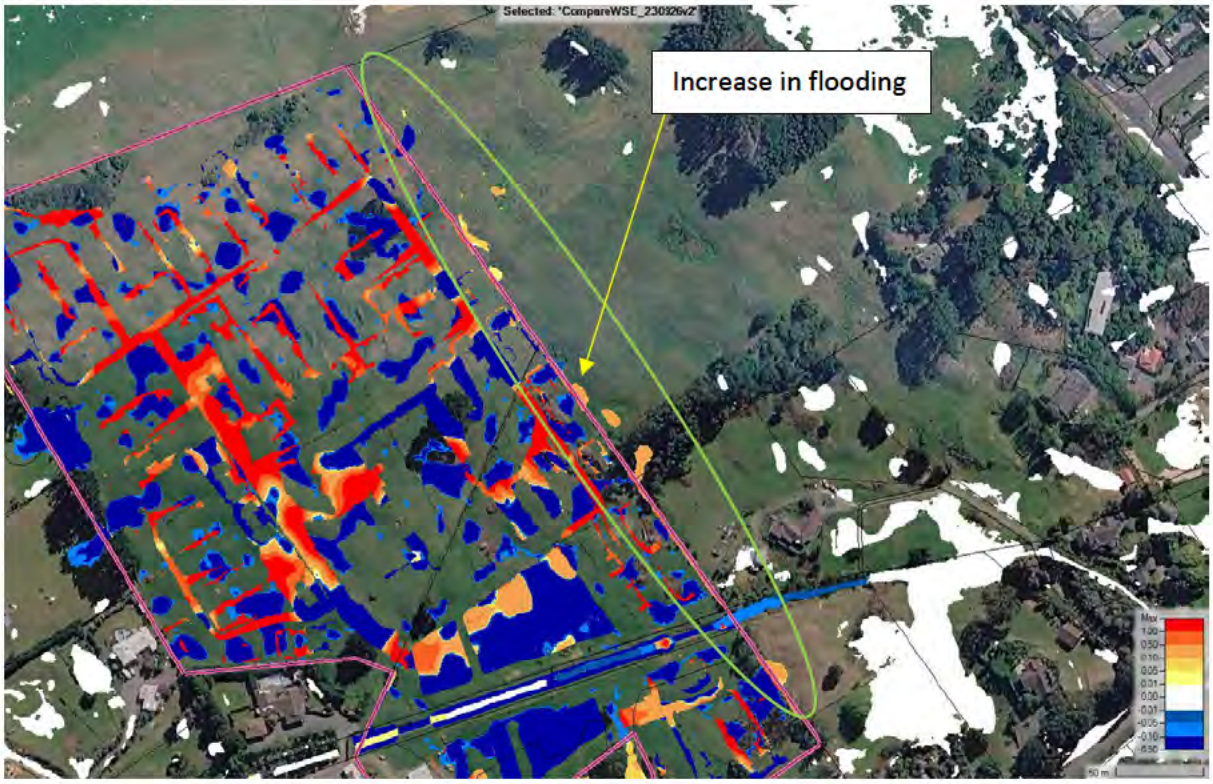



Figure 5: Increase in flood level by 200mm under proposed conditions within Eastern Site Boundary

As discussed previously, the eastern boundary is constrained by the existing terrain, where ridges and elevated ground along the site boundary make it impracticable to intercept and accommodate the external flows within the proposed site through conventional drainage measures. These areas are further discussed in Section 5 highlighting the mitigation measures that have already been considered where practicable.

These constrained areas form the focus of this memorandum. Given that further mitigation is not considered practicable, a flood hazard assessment has been undertaken to determine whether the observed increases in flood level result in a material increase in flood hazard level and extent compared with the existing condition, and consequently whether additional mitigation measures are warranted. This is further discussed in Section 6.

# 5 Mitigation Measures Investigated

Notwithstanding the topographical constraints, a range of mitigation measures were investigated during the concept design with the following objectives:

1. To manage site drainage such that no runoff generated within the proposed development discharges onto neighbouring properties.
2. To intercept and convey external overland flows that become impounded against the proposed raised boundary, where practicable, through an internal swale or drainage system.

## 5.1 Flows within the Proposed Development will be Managed Internally

The first objective has been successfully achieved. The proposed surface design ensures that runoff generated within the development is captured and managed internally (with controlled orifice discharge), with no discharge onto adjoining properties.

## 5.2 Accommodation of External Flows and Retention of Existing Storage

### 5.2.1 Western Boundary

The second objective has been achieved at key secondary flow paths along the western boundary.

**Location 1:**

Two culverts intercept external runoff and discharge into an internal depression area which will allow infiltration to the ground over time. Please see Figure 6 for the proposed arrangement. There is a small depression area where the existing southern overland flow path enters site to accept this flow prior to discharging to the culvert and the flood storage area. A northern culvert accepts smaller overland flow paths to the north.

The flood storage area is set below the boundary elevation to provide storage within the site prior to infiltrating to ground, as it does naturally. Recent groundwater testing in September 2026 indicated higher groundwater (RL 2.82m) after heavy winter rainfall than previous groundwater testing had indicated. The base of this flood storage area is set above the seasonally high groundwater (RL 3.0m) to ensure there is available flood storage in the basin year round.

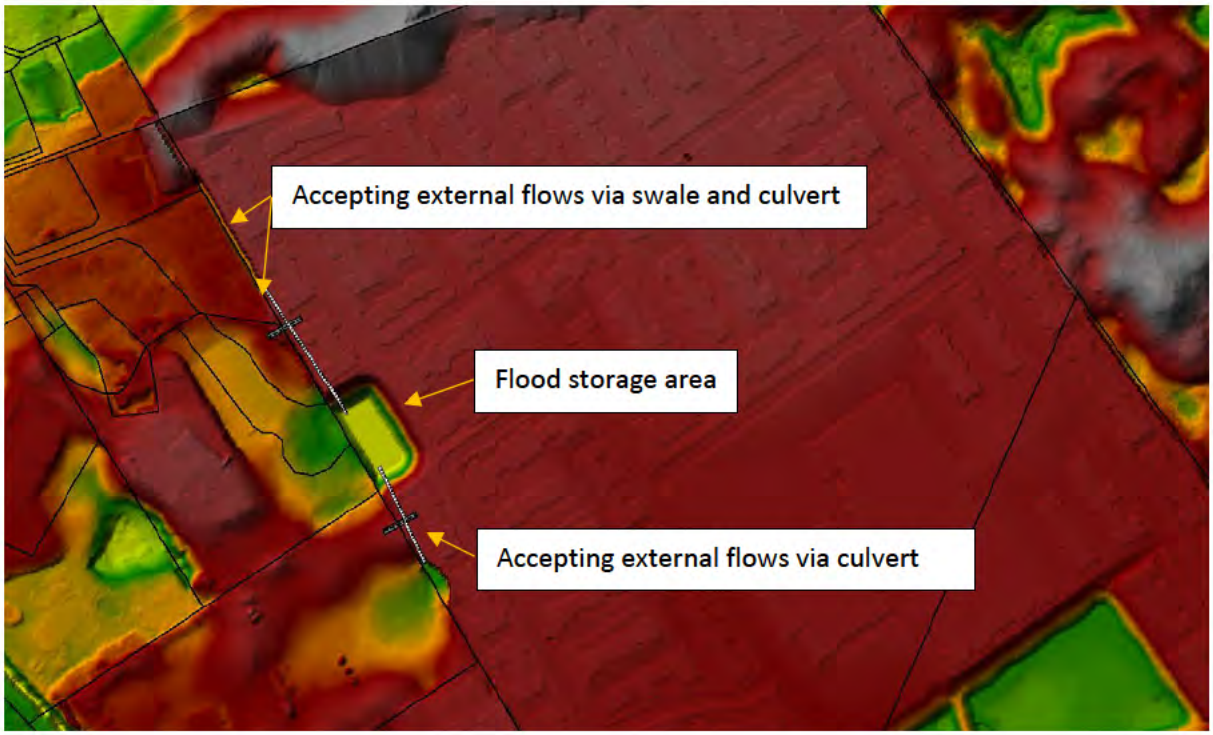



Figure 6: Mitigation measures along northwestern site boundary- Location 1

**Location 2:**

Similar to the above arrangement a 1m base width swale has been incorporated along the southwestern boundary to accept the external flows intercepted by the proposed development and discharge to the Drain. Please see the proposed arrangement in Figure 7.

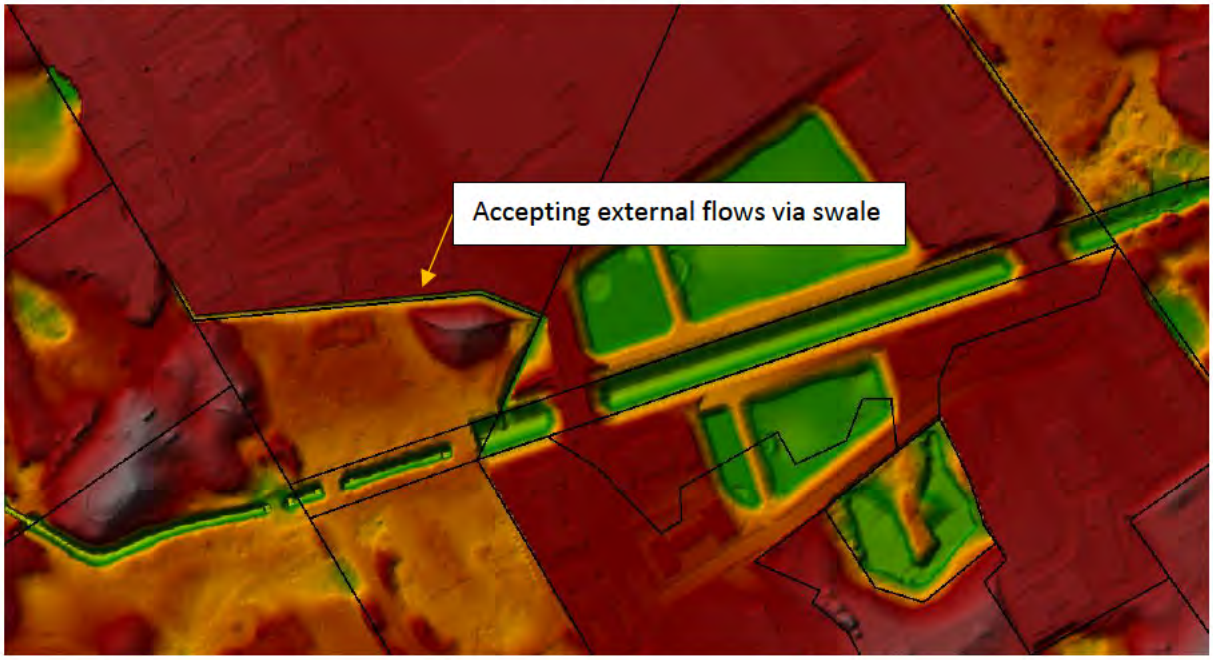



Figure 7: Mitigation measures along southwestern site boundary- Location 2

### 5.2.2 Eastern Boundary

#### 5.2.2.1 Swales

Implementing a similar solution along the eastern site boundary has proven significantly more challenging. The existing ground is highly undulating, comprising a series of ridges, knolls and small natural depressions. In several locations, the property boundary follows elevated ground, making it impracticable to construct a continuous swale capable of intercepting and conveying the external overland flows.

#### 5.2.2.2 Accommodation of External Flows within the Pipe Network

The peak runoff associated with these isolated ponding areas is expected to be relatively small. However, capturing and conveying these flows through the proposed piped drainage network is not considered hydraulically practical. Any such pipe would require a very shallow grade be difficult to implement considering the existing site constraints.

Accordingly, the following risk assessment focuses on the eastern site boundary, where the hydraulic modelling predicts localised increases in flood levels under the proposed condition and where opportunities for further mitigation are constrained by the existing topography and practical engineering limitations.

# 6 Flood Hazard Assessment

Flood hazard has been assessed using the depth–velocity product (d × V) in accordance with the Greater Wellington Regional Council (GWRC) Flood Hazard Modelling Standard. The depth–velocity product combines flood depth and flow velocity to provide an indication of the potential hazard posed by floodwaters.

The flood hazard classification adopted for this assessment is based on the Australian Rainfall and Runoff (ARR, 2016) flood hazard methodology, as referenced in the GWRC Flood Hazard Modelling Standard. The ARR methodology categorises flood hazard into hazard classes based on the relationship between flood depth and velocity, providing an indication of the relative risk to people, vehicles and buildings.

The ARR flood hazard classification curve adopted by the GWRC Flood Hazard Modelling Standard is presented in Figure 8 below and forms the basis of the flood hazard assessment presented in this memorandum.

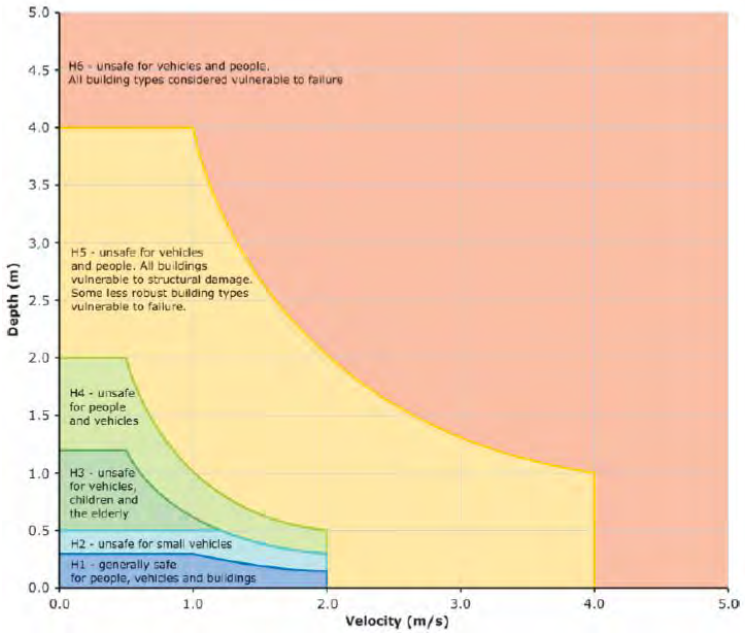



**Figure 8: Flood Hazard Classification adopted from ARR 2016 and presented in Flood Hazard Modelling Guidelines by GWRC**

The hazard classification was applied to the modelled existing and proposed flood depths and velocities at the assessed locations.

The results indicate that the affected areas fall within the H1 hazard category, which is classified by the ARR (2016) methodology as generally safe for people, vehicles and buildings, under both existing and proposed conditions.

## 6.1 Existing and Proposed Flood Hazard_ Eastern Boundary

Under existing conditions, flood depths along the eastern boundary (on the neighbouring land) are generally less than 0.50 m and velocities are typically below 0.30 m/s. Therefore, maximum depth–velocity product is approximately 0.15 $m^2/s$, which is well below the 0.40 $m^2/s$ upper threshold for the H1 hazard category. The resulting hazard classification is shown below in Figure 9, with purple areas demonstrates hazard category H1.

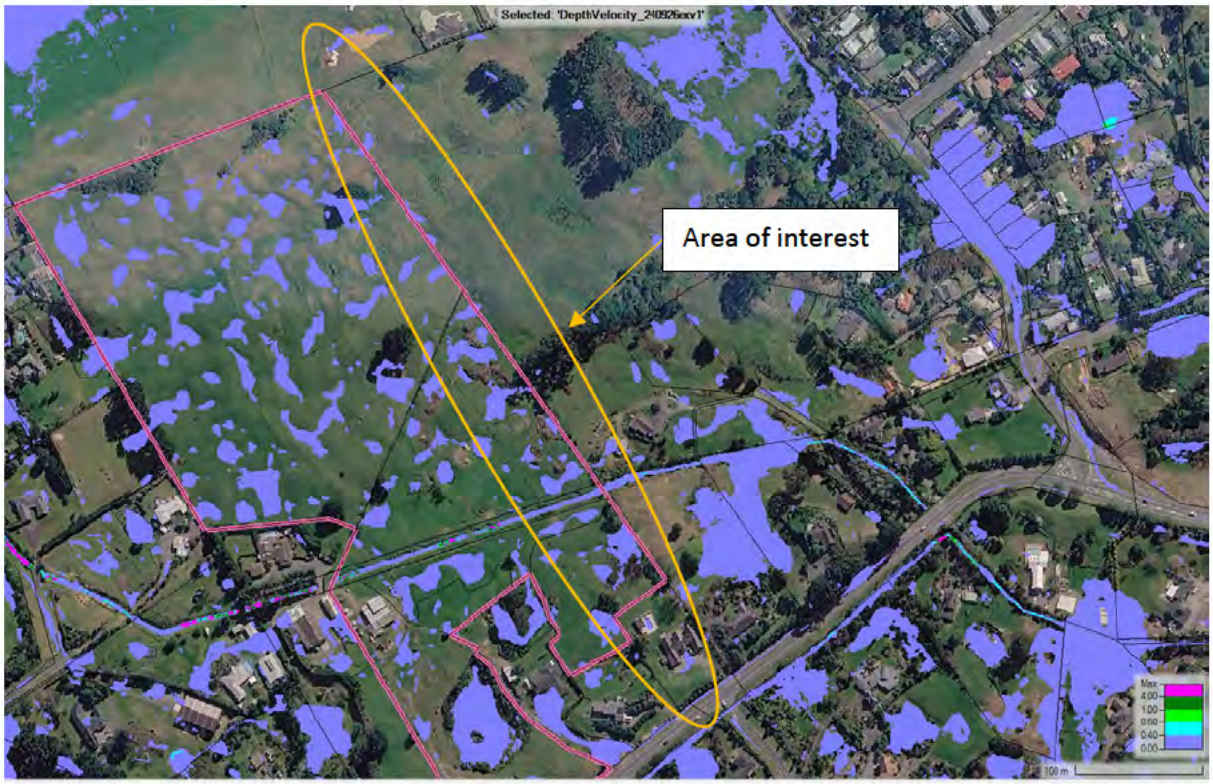



Figure 9: Flood Hazard under existing conditions_ Eastern Boundary

Although the proposed condition results in a localised flood-depth increase of up to approximately 300 mm, the affected area remains within the H1 hazard category. The proposed development therefore does not materially change the modelled flood hazard classification at the eastern boundary.

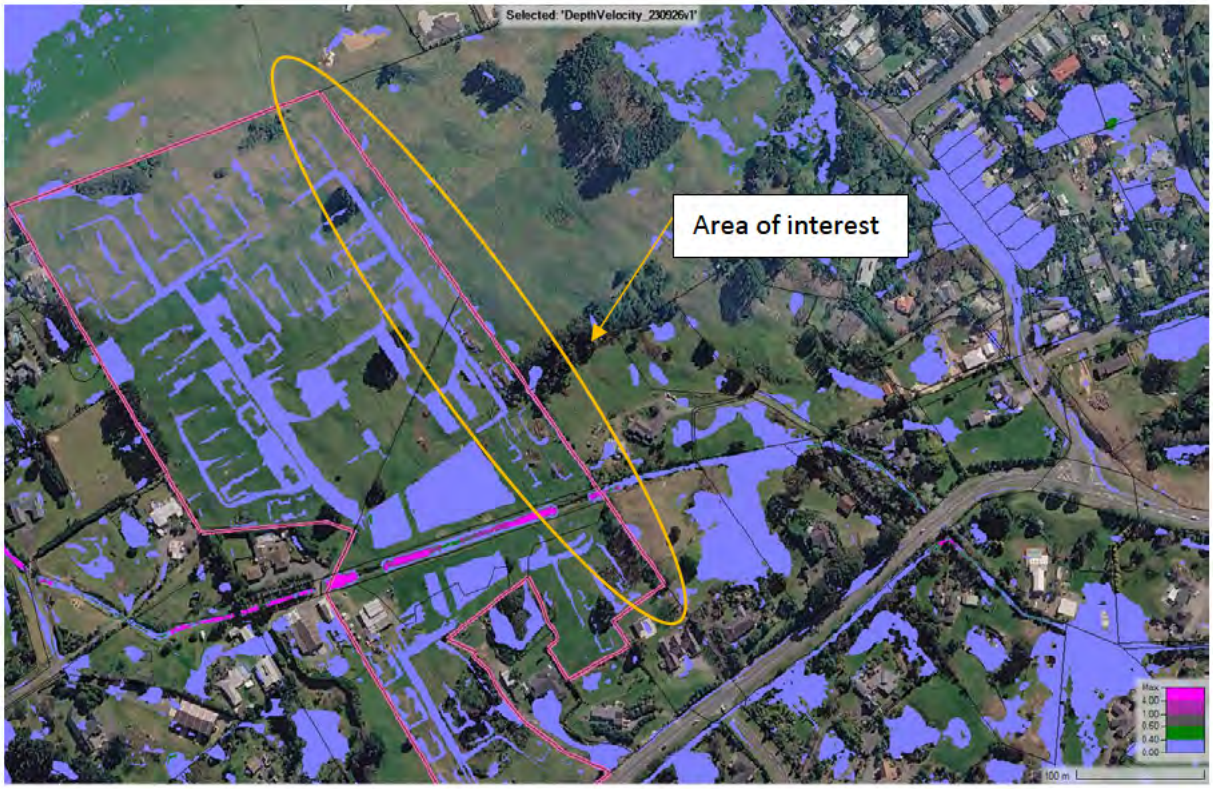



Figure 10: Flood Hazard under proposed conditions_ Eastern Boundary

It should also be noted that the proposed development does not increase runoff to neighbouring properties or obstruct existing overland flow paths. The localised increase in flood depth predicted by the modelling is primarily attributable to the reduction in storage volume within small natural depressions.

Accordingly, the proposed development does not transfer additional flood flows to adjoining properties or reduce their ability to be developed in the future by requiring them to accommodate increased runoff from the site. Rather, the effect is limited to an increase in flood depth within existing low-lying areas adjacent to the site, without materially altering the overall flood regime and hydraulic performance.

The flood extent remains consistent with the existing scenario. Localised increases in flood level are confined to existing depressions by the development boundary and the narrow corridor between the development footprint and the site boundary. These ponding areas are expected to infiltrate gradually, as occurs under existing conditions.

The model was simulated for 36 hours, and ponding remained in both the existing and proposed scenarios at the end of the simulation. The modelling therefore does not establish the change in total ponding duration

# 7 Conclusions and Recommendations

The proposed development results in localised increases in flood depth along portions of the site boundary, where proposed earthworks affect existing natural depressions and associated flood storage. Design measures have been incorporated to retain storage and accommodate external flows within the site where practicable, particularly along the western boundary. Further mitigation along the eastern boundary is constrained by the existing topography, limited available gradients. Additional works would require substantial earthworks or drainage infrastructure and are not considered proportionate to the modelled effects.

The flood hazard assessment demonstrates that, despite the minor increases in flood levels, the affected areas remain within the H1 flood hazard category under both the existing and proposed conditions and that there is no material change in flood extents. Therefore, it is considered safe for people, buildings and vehicles. Flood depths and velocities remain low, resulting in no material increase in flood hazard.

Overall, the proposed development does not materially alter the existing overland flow regime or increase the modelled flood hazard classification at the assessed locations. Given the practical constraints associated with implementing additional mitigation and the absence of any increase in flood hazard classification, the residual effects are considered acceptable from a flood hazard perspective.

# FRA Appendix 3: Statement of Experience

**Statement of Experience**

P. A. Sarala Karunarathna

**Qualifications and professional experience**

My full name is P. A. Sarala Karunarathna. I am an Intermediate Environmental Engineer at CKL New Zealand Limited, having joined the company in September 2024.

I hold a Bachelor of Engineering (Hons) in Civil Engineering and a Master of Engineering in Structural Engineering. I am a member of Engineering New Zealand (MEngNZ).

I have more than six years of engineering experience, including approximately five years specialising in stormwater management, hydrological and hydraulic modelling. My experience includes assessment of development effects, flood-mitigation optioneering, risk assessment, stormwater quantity and quality management, and preparation of technical reports supporting consent applications.

**Relevant technical experience**

My technical experience includes the development of hydrological and hydraulic models to assess flood behaviour and inform stormwater management and development design. I develop flood models from initial terrain and catchment data through model validation and calibration where required. I use HEC-HMS for hydrological assessments and HEC-RAS, TUFLOW and SWMM for hydraulic modelling.

My work includes establishing existing flood conditions and incorporating proposed terrain, infrastructure and development layouts into hydraulic models to assess development effects. This involves comparing existing and proposed flood levels, depths, velocities and flow patterns to identify changes within the site and effects on neighbouring properties and downstream areas.

I interpret model results and hydraulic behaviour to identify the causes of flooding and develop practicable mitigation options. These options are incorporated into the hydraulic model to evaluate their effectiveness, assess potential adverse effects and inform selection of the best practicable option. I then assess the residual flood risk following implementation of the preferred mitigation measures, including risks to buildings, surrounding properties, and access and egress.

Recent relevant assignments include the proposed McDonald's developments at 732 Beach Road, Browns Bay, and 450 Broadway, Newmarket. For both projects, I undertook HEC-RAS 2D rain-on-grid modelling to assess existing and proposed flood conditions, evaluate development effects and test mitigation options. At Browns Bay, the assessment informed selection of a suspended driveway that allows floodwater to pass beneath the platform, supported by a residual-risk assessment and an operation and maintenance manual. At Newmarket, mitigation focused on refining the proposed site

levels and assessing the resulting flood effects and residual risk. For both projects, I prepared technical documentation and responded to Council requests for further information through to consent approval.

**Role in the Paraparaumu Fast-track application**

For the Paraparaumu Retirement Village project, I undertook the 2D rain-on-grid hydraulic assessment of existing and proposed conditions. My work included assessment of development-related changes in flooding, maintain hydraulic neutrality, evaluation of mitigation options and assessment of residual flood risk.

I prepared the flood-risk assessment report, flood model build summary and residual-risk assessment memorandum to support the Fast-track application. My work was reviewed by Frances Deamer Philip, Senior Engineer, and Zeb Worth, Principal Engineer.

**Code of Conduct**

Although this statement and the associated stormwater assessment have been prepared to support an EPA application under the Fast-track Approvals, rather than as evidence in an Environment Court proceeding, I have prepared my technical assessment and opinions on an independent professional basis.

I have read the Code of Conduct for Expert Witnesses contained in section 9 of the Environment Court Practice Note 2023 and agree to comply with that Code to the extent that it is applicable to my involvement in this project. I understand that an expert's primary obligation when providing expert opinion is to provide impartial assistance on matters within their expertise and not to act as an advocate for the party engaging them.

Except where I expressly identify reliance on another specialist, the matters addressed by me are within my area of expertise. I have identified the information, assumptions and limitations material to the opinions I have expressed, and I have not knowingly omitted to consider any material fact that might alter or detract from those opinions. Where information remains incomplete at referral stage, I have identified the further investigation or assessment required rather than assuming that issue to have been resolved.

P. A. Sarala Karunarathna
BEng Civil (Hons), MEng Structural, MEngNZ
Intermediate Environmental Engineer
CKL New Zealand Limited

# Appendix 5 Draft Operation and Maintenance Plans

CKL

Planning | Surveying | Engineering | Environmental

# Summerset Retirement Village Paraparaumu

**Summerset Group Holdings Ltd**

**P O Box 5187, Wellington , 6140**

# Document Information

| | |
|---|---|
| **Client** | **Summerset Group Holdings Ltd** |
| **Site Location** | 65 & 73 Ratanui Road, Paraparaumu, Wellington, WELL, NZ |
| **Legal Description** | Lot 4 DP 58017, Certificate of Title 29D/823 (65 Ratanui Road)<br>Lot 3 DP 497389, Certificate of Title 734643 (73 Ratanui Road) |
| **CKL Reference** | A25556 |
| **Office of Origin** | Auckland |

| | | | |
|---|---|---|---|
| **Author** | Sarala Karunarathna | | |
| **Signed** | | **Date** | 24/09/2026 |

| | | | |
|---|---|---|---|
| **Reviewed By** | Frances Deamer-Phillips | | |
| **Signed** | | **Date** | 28/09/2026 |

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|---|---|---|---|
| **Authorised By** | Frances Deamer-Phillips | | |
| **Signed** | | **Date** | 28/09/2026 |

| Revision | Status | Date | Author | Reviewed By | Authorised By |
|---|---|---|---|---|---|
| 0 | DRAFT | 28/09/26 | SK | FDP | FDP |
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# 1 Introduction

This report provides an Operation and Maintenance Plan for the stormwater management devices proposed for the Summerset Retirement Village development at 65 and 73 Ratanui Road, Paraparaumu. It sets out the inspection and maintenance requirements necessary to maintain the treatment, retention, conveyance and flood-mitigation functions of these devices.

This report should be read in conjunction with the proposed Stormwater Management Plan, which provides further details on the design and operation of the stormwater management system.

# 2 Background

The plan covers the northern and southern bioretention devices and attenuation wetlands, associated outlet culverts and flap gates, secondary flood-mitigation swales and culverts, and the western flood-storage area. Please see details below in Figure 1.

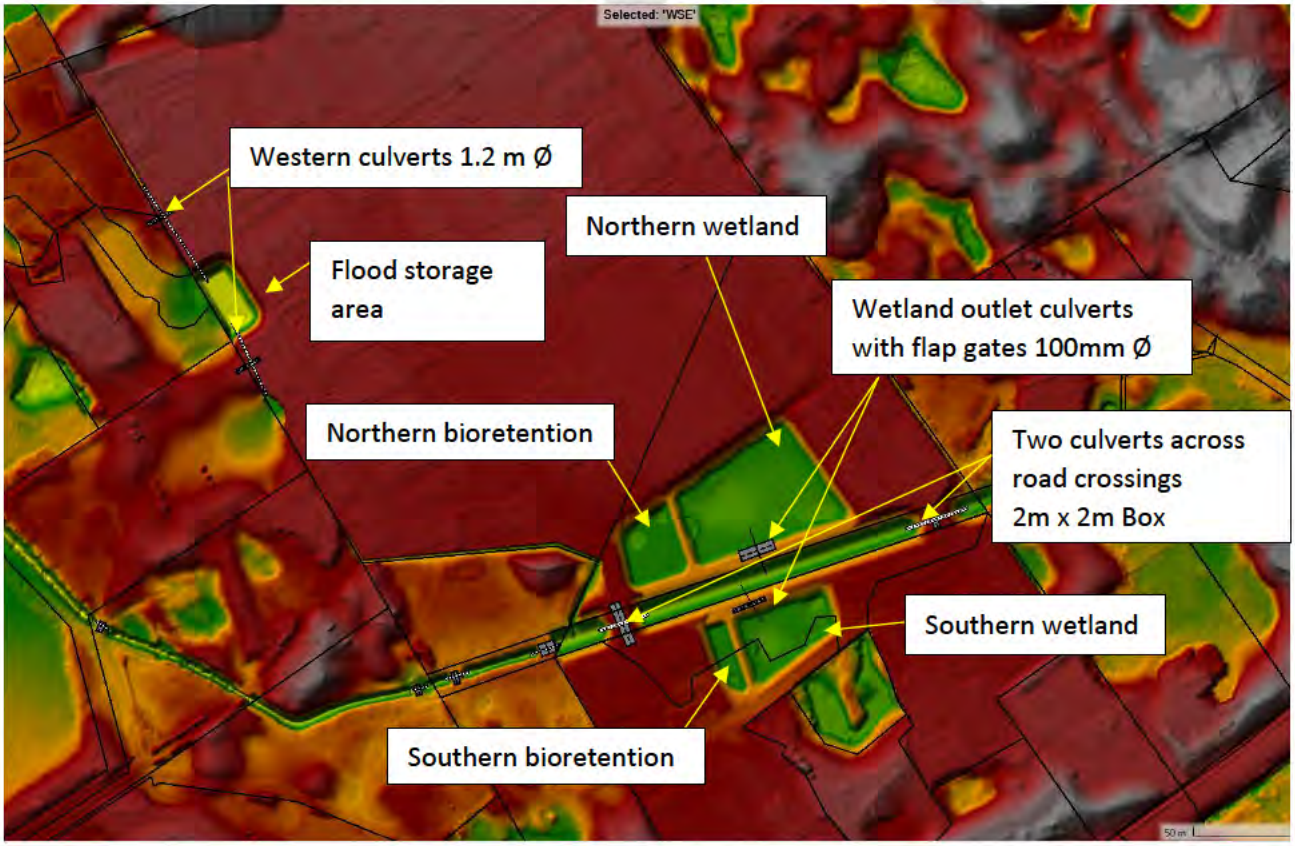



*Figure 1: Stormwater Management Devices*

Functionality of each device is detailed in Table 1

*Table 1: Functionality of stormwater devices shown above*

| Device | Functionality |
|---|---|
| **Bioretention Devices** | The upstream bioretention devices capture and treat first-flush runoff before it enters the downstream wetlands. It provides retention and treatment. |
| **Wetlands** | The northern and southern wetlands receive treated runoff from the upstream bioretention devices. They provide detention and attenuation of stormwater flows for flood protection, with stored water discharged at a controlled rate to the Drain. They also provided habitat for offsetting of lost wetlands |
| **Secondary Flow-Conveyance Swales and Culverts** | The secondary swales and culverts are designed to intercept and safely convey secondary flows during the 1% AEP design storm event. These devices maintain the required overland flow path and direct runoff towards the designated flood-storage and downstream discharge areas. |
| **Flood-Storage Area** | The western flood-storage area provides temporary storage for stormwater during significant rainfall events. It reduces peak flows by retaining runoff and allowing the stored water to drain or infiltrate over time. |

# 3 Operation and Maintenance Requirements

Operational and maintenance requirements for each device are listed below.

## 3.1 Bioretention Devices

Regular inspection and maintenance shall be undertaken to ensure that the bioretention devices continue to provide effective stormwater retention and treatment. The following requirements apply to both the northern and southern bioretention devices.

### 3.1.1 Flow Through the Bioretention Devices

The devices shall be inspected monthly and following significant storm events. Maintenance shall include:

- Clearing flow paths and inlets leading to the devices.
- Removing litter, sediment and debris that may obstruct flow or inhibit plant growth.
- Clearing blocked inlets, outlets and overflow structures.
- Checking for prolonged surface ponding. Ponded water should generally drain within 24 hours; longer drawdown times may indicate surface clogging or reduced infiltration.

### 3.1.2 Erosion

Forebays shall be inspected at six-monthly intervals and following significant storm events. Any erosion, scour or damage to the side slopes, inlet areas or outlet structures shall be repaired using suitable material and replanted where required.

### 3.1.3 Sediment Management

Sediment shall be removed when it obstructs inflows, reduces the surface storage volume or causes prolonged ponding. When sediment has built up to 1/3$^{rd}$ of the forebay height, it should be removed and disposed of appropriately. Where the treatment surface has become clogged, the affected surface media shall be removed and replaced with suitable bioretention media, with mulch and vegetation reinstated to

the original design condition. Removed sediment and media shall be disposed of at an appropriate authorised facility.

### 3.1.4 Vegetation Maintenance

During the first three years, the bioretention planting shall be inspected at least twice annually to confirm that vegetation is establishing and remains healthy. Thereafter, inspections shall be undertaken annually, with additional maintenance as required. Maintenance shall include:

- Removing weeds, invasive plants and litter.
- Replacing dead or poorly established plants.
- Watering during extended dry periods.
- Maintaining vegetation without obstructing inlets, outlets or overflow paths.
- Avoiding phosphorus-based fertilisers.
- Preventing grass clippings and other landscaping waste from entering the devices.

Herbicides shall be used only where necessary, applied by an appropriately qualified person and in accordance with the manufacturer's instructions and applicable regulatory requirements.

### 3.1.5 Inlet and Outlet Protection

Inlet and outlet areas shall be inspected monthly and following significant storm events for structural integrity, blockage, erosion and scour. Damaged erosion protection shall be reinstated, and any recurring erosion shall be assessed to determine whether additional rock protection, geotextile, flow distribution or revegetation is required.

## 3.2 Wetland Operation and Maintenance

Regular inspection and specific maintenance shall be undertaken to ensure that the northern and southern wetlands continue to provide the required stormwater attenuation functions.

### 3.2.1 Flow, Erosion and Vegetation

Inspection and maintenance of flow paths, erosion-prone areas and vegetation shall generally be undertaken in accordance with the requirements specified for the bioretention devices in Section 3.1. Particular attention shall be given to the wetland forebays and wetland side slopes.

### 3.2.2 Sediment Management

When sediment has built up to 1/3$^{rd}$ of the forebay height, it should be removed and disposed of appropriately. Sediment monitoring shall occur where it affects vegetation or reduces the available wetland storage volume.

### 3.2.3 Structural Components

Inlets pipes and associated erosion protection and the outlet structures shall be inspected annually for cracking, leakage, displacement or deterioration. Any defects affecting the safety or hydraulic performance of the wetland shall be repaired by suitably qualified personnel.

#### 3.2.3.1 Flap-Gate Inspection and Testing

Each flap gate shall be inspected monthly and following significant storm events for debris, sediment, corrosion, physical damage and obstructions around the hinges and seating surfaces. At least six-monthly, the flap shall be safely operated through its available range of movement to confirm that it:

- Opens freely under forward flow.
- Closes fully when forward flow ceases.
- Seats evenly without binding.
- Prevents reverse flow.
- Remains securely attached to the outlet.

Personnel shall remain clear of the gate and other moving components during testing.

Debris shall be removed from the hinges and between the flap and seating surface. Hinges, pins, bushes, seals, fixings and protective coatings shall be checked for wear, looseness or deterioration and maintained in accordance with the flap-gate manufacturer's instructions.

A flap gate that does not open freely, close fully or prevent reverse flow shall be cleaned, adjusted or repaired promptly. Where the gate, culvert or supporting structure is damaged or cannot be restored through routine maintenance, it shall be assessed and repaired or replaced by suitably qualified personnel.

### 3.2.4 Erosion and Scour of Outlets

The areas immediately downstream of each outlet pipe shall be inspected monthly and following significant storm events for erosion, scour or undermining. Displaced or damaged erosion protection shall be reinstated, and recurring scour shall be assessed to determine whether additional protection is required.

### 3.2.5 Embankments and Spillways

Wetland embankments and spillways shall be inspected annually and following significant storm events for settlement, cracking, erosion, scour, seepage and changes in level or gradient. Evidence of significant seepage or instability shall be assessed by a suitably qualified geotechnical engineer.

## 3.3 Drain Box Culvert Operation and Maintenance

The 2 m × 2 m box culverts convey the main Drain beneath the proposed road crossings. The culverts are embedded approximately 25% below the streambed level to retain a natural bed through the culvert and facilitate fish passage.

### 3.3.1 Flow and Blockage Management

The culvert openings and approaches shall be inspected quarterly and following significant storm events. Debris, litter or vegetation that materially obstructs streamflow shall be removed without unnecessarily disturbing the natural streambed.

### 3.3.2 Sediment and Streambed Condition

Sediment movement through and around the culverts shall be monitored to confirm that the culvert bases remain embedded and covered by natural bed material. Maintenance shall avoid removing the embedded streambed layer unless required to address an obstruction or structural issue. If erosion exposes the culvert base or excessive deposition significantly reduces the available waterway area, appropriate reinstatement shall be undertaken and structural sediment components can be considered.

### 3.3.3 Structural Condition

The culvert barrels, joints, headwalls, wingwalls and road-crossing structures shall be inspected annually for cracking, displacement, leakage or deterioration. Any defects affecting structural integrity or hydraulic performance shall be assessed and repaired by suitably qualified personnel.

### 3.3.4 Erosion and Scour

The upstream and downstream approaches, culvert edges and erosion-protection measures shall be inspected for scour, bank erosion and undermining. Any damage shall be repaired using suitable materials while maintaining the stream alignment and embedded culvert condition.

### 3.3.5 Fish Passage

The culvert bed shall remain embedded and continuous with the upstream and downstream streambed to facilitate fish passage. Any maintenance works shall retain or reinstate this condition following completion.

## 3.4 Secondary Swale Operation and Maintenance

The secondary swales convey overland flows through the western part of the site during events up to the 1% AEP design storm. The swales shall be maintained to preserve their design alignment, levels, cross-sectional area and hydraulic capacity.

### 3.4.1 Flow and Obstruction Management

The swales shall be inspected six-monthly and following significant storm events. Litter, debris, excessive vegetation and other obstructions that may restrict or divert flow shall be removed.

### 3.4.2 Vegetation Maintenance

Vegetation shall be maintained at a suitable height to support stable flow conveyance and protect the swale surface from erosion. Bare areas, damaged vegetation and areas of plant dieback shall be reseeded or replanted as required.

### 3.4.3 Sediment Management

Accumulated sediment shall be removed where it materially reduces the swale capacity, obstructs flow or alters the design gradient. Following sediment removal, the swale profile and vegetation shall be reinstated.

### 3.4.4 Erosion and Settlement

The swales shall be inspected for erosion, scour, rutting, settlement and unintended ponding. Damaged areas shall be regraded, stabilised and revegetated to restore the design profile and continuous flow path.

### 3.4.5 Protection of the Flow Path

Filling, landscaping, structures, fencing or other modifications that reduce the swale capacity or obstruct the designated flow path shall not be permitted. Any unauthorised obstruction or alteration shall be removed and the swale reinstated.

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## 3.5 Flood-Storage Area Operation and Maintenance

The flood-storage area provides temporary storage during significant rainfall events. It shall be maintained to preserve the design storage volume, ground levels and connections to the secondary flow-conveyance system.

### 3.5.1 Storage Capacity

The area shall be inspected annually and following significant storm events for sediment accumulation, filling, ground modification or other changes that may reduce the available storage volume. Any deposited material or unauthorised filling shall be removed, and the design ground profile reinstated.

### 3.5.2 Inlets, Outlets and Flow Paths

Flow paths into and out of the storage area shall be inspected six-monthly and following significant storm events. Sediment, litter, vegetation and other obstructions that may restrict or divert flow shall be removed.

### 3.5.3 Vegetation Maintenance

Vegetation shall be maintained to provide stable ground cover without materially reducing storage or obstructing flow. Bare, eroded or damaged areas shall be stabilised and revegetated.

### 3.5.4 Erosion and Settlement

The storage area shall be inspected for erosion, scour, settlement and localised changes in ground level. Damaged areas shall be regraded and stabilised to restore the design profile and drainage pattern.

### 3.5.5 Drain-Down Performance

The area shall be observed following significant storm events to confirm that stored water drains or infiltrates within the expected period. Prolonged ponding shall be investigated to identify blocked outlets, sediment accumulation or reduced infiltration.

### 3.5.6 Protection of the Storage Area

Structures, stockpiles, landscaping, filling or other activities that reduce the storage volume or obstruct flow shall not be permitted. Any unauthorised modification shall be removed and the affected area reinstated.

# 4 Inspection Schedule and Maintenance Programme

The following programme summarises the minimum inspection frequencies, maintenance triggers and corrective actions for the stormwater devices. Inspection frequency shall be increased where repeated blockages, erosion, poor vegetation establishment or other performance issues are identified.

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*Table 2: Summary of Inspection Schedule and maintenance requirements*

| Stormwater element | Routine inspection | Annual inspection | Post-storm inspection | Maintenance triggers | Required corrective actions |
|---|---|---|---|---|---|
| Bioretention devices | Monthly; vegetation inspected twice yearly during the first three years | Vegetation, media and overall condition | Erosion, scour, sedimentation and structural components | Blocked flow paths, prolonged ponding, sediment build up, erosion, weeds or plant dieback | Remove obstructions and sediment, repair erosion, replace media where clogged, and reinstate vegetation |
| Wetlands | Monthly inspection of flow paths, inlets and outlets | Storage, vegetation, sediment, embankments, spillways and structures | Erosion, scour, sedimentation and structural components | Reduced storage of forebays, sediment build up, erosion, seepage or structural damage of inlets and outlets | Remove sediment and debris at forebays and outlets, repair erosion or structures |
| Main stream box culverts | Monthly inspection of openings and stream approaches | Culvert barrels, joints, headwalls, wingwalls and road crossings | Erosion, scour, sedimentation and structural components | Blockage, exposed culvert base, excessive deposition, erosion, scour or structural damage | Remove obstructions, reinstate the embedded streambed condition, repair erosion protection, and complete structural repairs, assess fish passage/ sediment retention structures |
| Secondary swales | Six-monthly | Confirm profile, levels and overall condition | Scour and alignment | Debris, excessive vegetation, sediment, erosion, settlement or unintended ponding | Clear obstructions, remove sediment, regrade damaged areas, stabilise erosion, and |

| | | | | | |
|---|---|---|---|---|---|
| | | | | | reinstate vegetation |
| Flood-storage area | Annual inspection of flow paths and vegetation | Confirm storage volume, ground profile and overall condition | Erosion, scour, sedimentation/ infiltration | Filling or sedimentation, blocked flow paths, prolonged ponding, erosion or settlement | Remove deposited material and obstructions, restore the design profile, repair erosion, and investigate poor drainage |

All inspections and maintenance works shall be recorded, including the inspection date, observed condition, required action, completed repairs and supporting photographs where appropriate.

# 5 Roles and Responsibilities

Summerset Holdings shall appoint a designated person responsible for coordinating inspections, arranging maintenance and retaining records for all stormwater devices. Specialist maintenance or repair works shall be undertaken by suitably qualified contractors under the direction of the designated person.

| Stormwater element | Responsible person | Required records | Health and safety requirements |
|---|---|---|---|
| Bioretention devices | Person designated by Summerset Holdings | Inspection checklists, photographs, vegetation maintenance, media replacement and sediment-disposal records | Manage manual-handling, herbicide, contaminated-sediment and uneven-ground hazards; use appropriate PPE |
| Wetlands | Person designated by Summerset Holdings | Inspection checklists, photographs, sediment removal, vegetation work, repairs and drawdown observations | Assess deep water, soft ground, steep slopes, machinery and biological hazards before starting work, remain clear of the flap during testing, and apply confined-space controls where relevant |
| Main stream box culverts | Person designated by Summerset Holdings | Inspection checklists, photographs, sediment and streambed observations, scour repairs and structural assessments | Assess flowing water, confined-space, unstable-bank and traffic hazards; culvert entry shall only occur under an approved safe-work procedure |
| Secondary swales | Person designated by Summerset Holdings | Inspection checklists, vegetation maintenance, sediment removal, erosion | Uneven-ground, manual-handling and vegetation-maintenance hazards |

| | | | |
|---|---|---|---|
| | | repairs and regrading records | |
| Flood-storage area | Person designated by Summerset Holdings | Inspection checklists, photographs, post-storm observations, sediment removal and ground-reinstatement records | Do not enter during flooding or while water levels are unsafe; assess soft ground, deep water and machinery hazards before maintenance |

The designated person shall ensure that maintenance contractors are appropriately qualified, site hazards are assessed before work begins, and all identified defects and corrective actions are recorded through to completion.

# 6 Maintenance Access

Maintenance access shall be provided to all stormwater devices to allow routine inspections, vegetation management, debris and sediment removal, and structural repairs. Access routes shall remain unobstructed and suitable for maintenance personnel and, where required, vehicles or machinery. Any disturbance caused during maintenance shall be reinstated following completion of the works. Please refer the maintenance access shown in Appendix 1.

# 7 Review of the Operation and Maintenance Plan

This Draft Operation and Maintenance Plan shall be updated through detailed design and reviewed following any modification to the stormwater system. It should be a live document and updated after significant repair, repeated performance issue or change in ownership, maintenance responsibility or applicable consent requirements. The plan shall also be reviewed periodically to confirm that the inspection frequencies, maintenance triggers and corrective actions remain appropriate.

Any revisions shall be approved by CKL and Summerset Holdings and communicated to the designated responsible person and relevant maintenance contractors. The current version of the plan shall be retained with the stormwater inspection and maintenance records.

# 8 Limitations

This report has been prepared solely for the benefit of our client with respect to the particular brief and it may not be relied upon in other contexts for any other purpose without the express approval by CKL. Neither CKL nor any employee or sub-consultant accepts any responsibility with respect to its use, either in full or in part, by any other person or entity. This disclaimer shall apply notwithstanding that the memo/report may be made available to other persons including Council for an application for consent, approval or to fulfil a legal requirement.

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