Flood Risk Assessment

Summerset Group Holdings Ltd

CKL

Planning | Surveying | Engineering | Environmental

Document Information

Client	**Summerset Group Holdings Ltd**
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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.

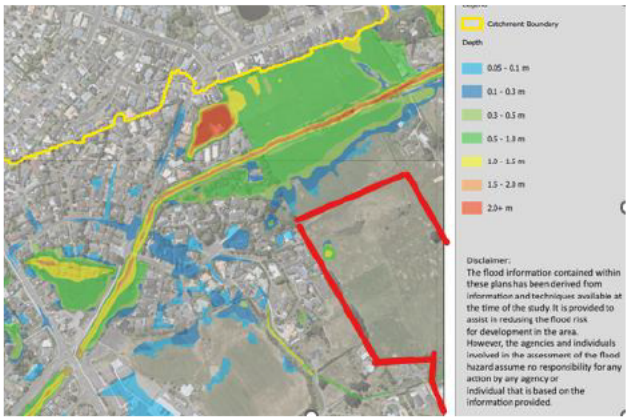




Figure 3: Flood depths from AWA flood model 100CC (Site extents outlined in red solid line)

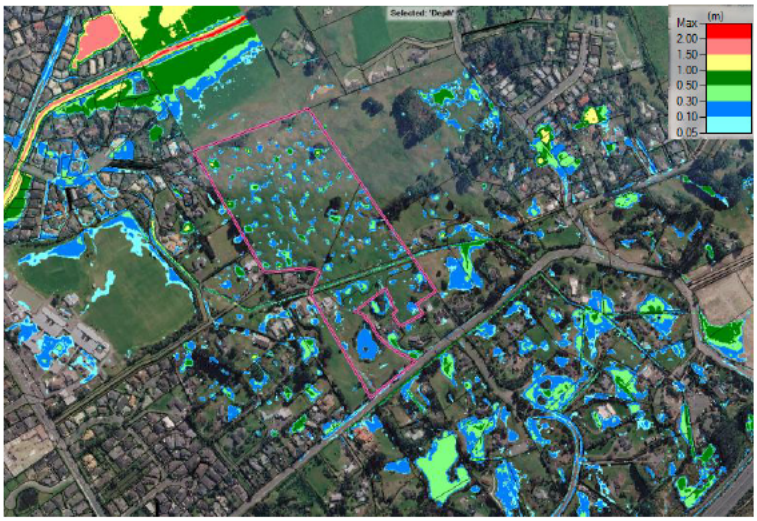


Figure 4: Flood depths from CKL flood model 100CC (Site extents outlined in pink solid line)

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).

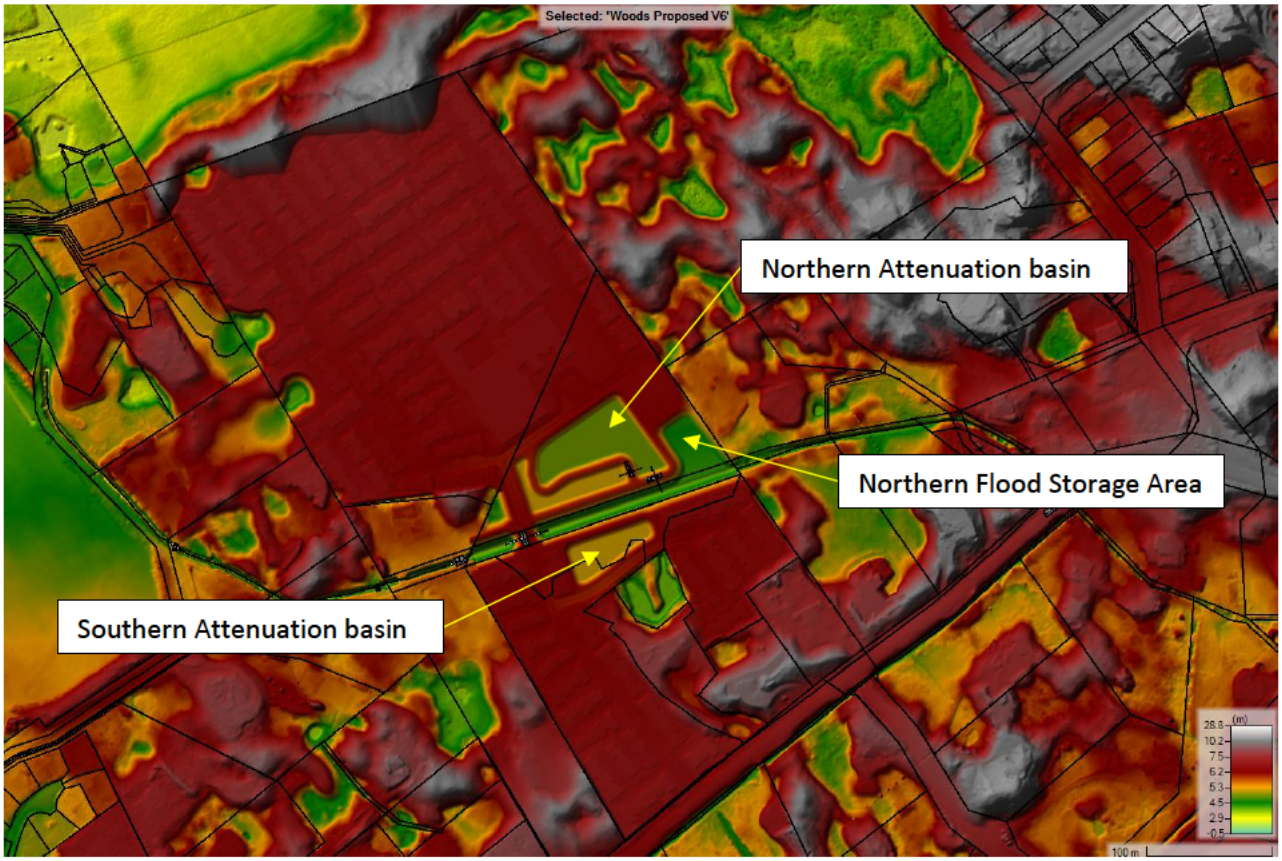


Figure 5: Woods Proposed Concept Design

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.

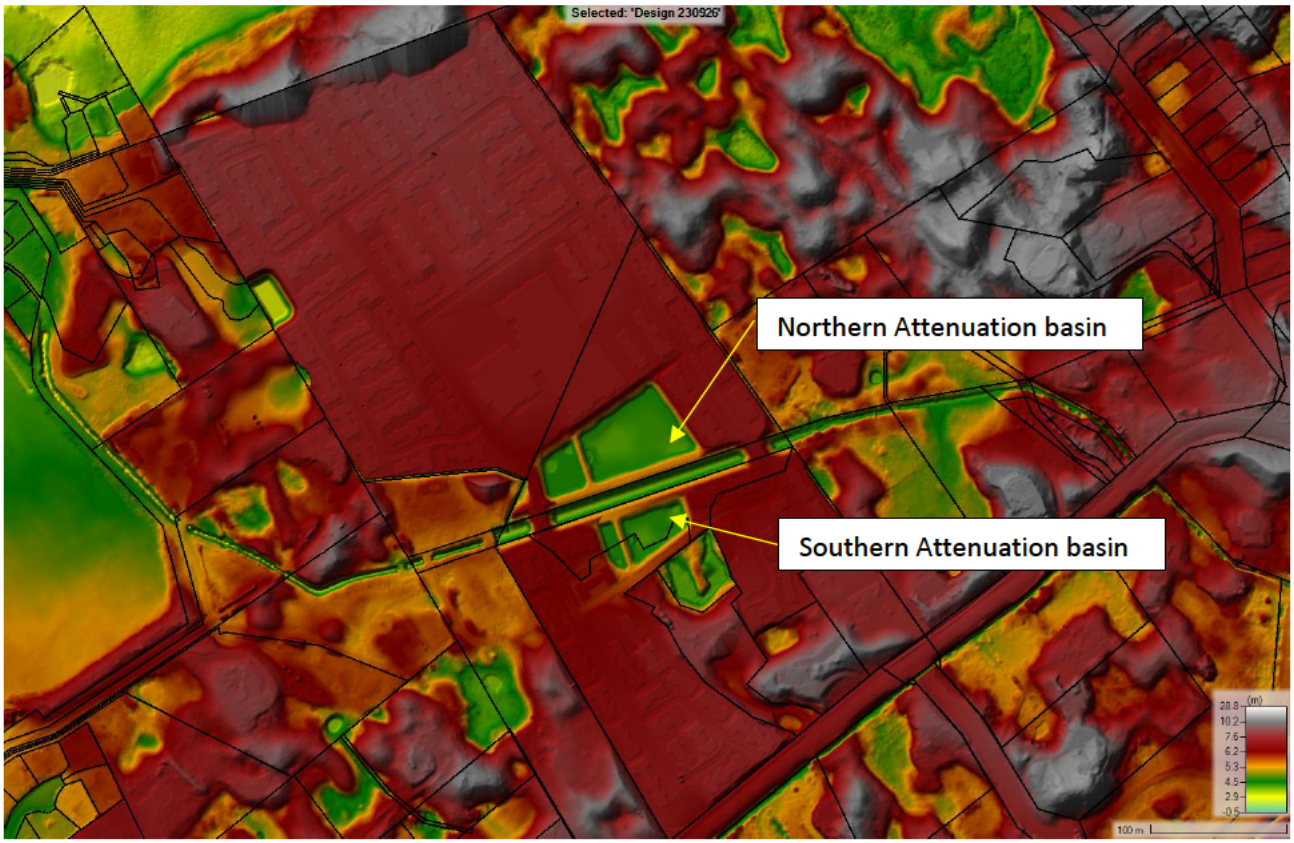


Figure 6: CKL Modified Design

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).

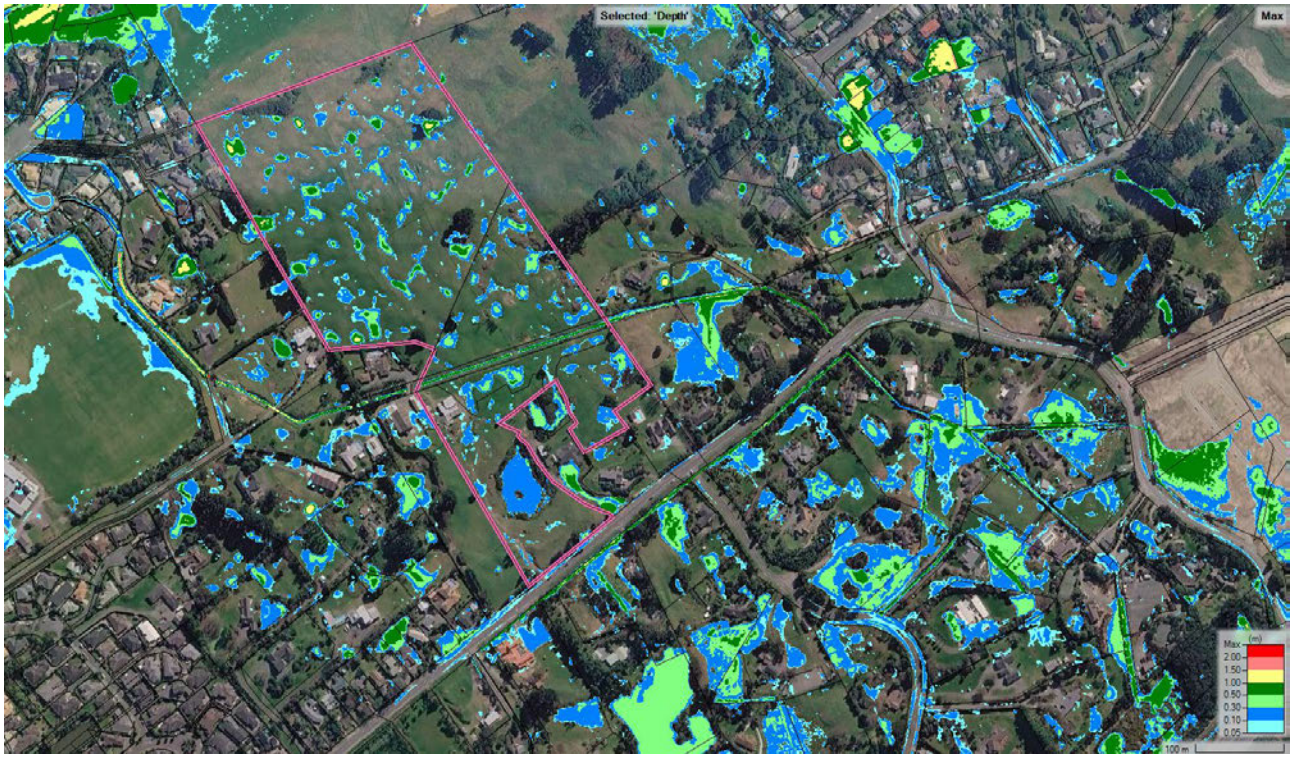


Figure 7: Flood Extents Under Existing Conditions - 100yrCC

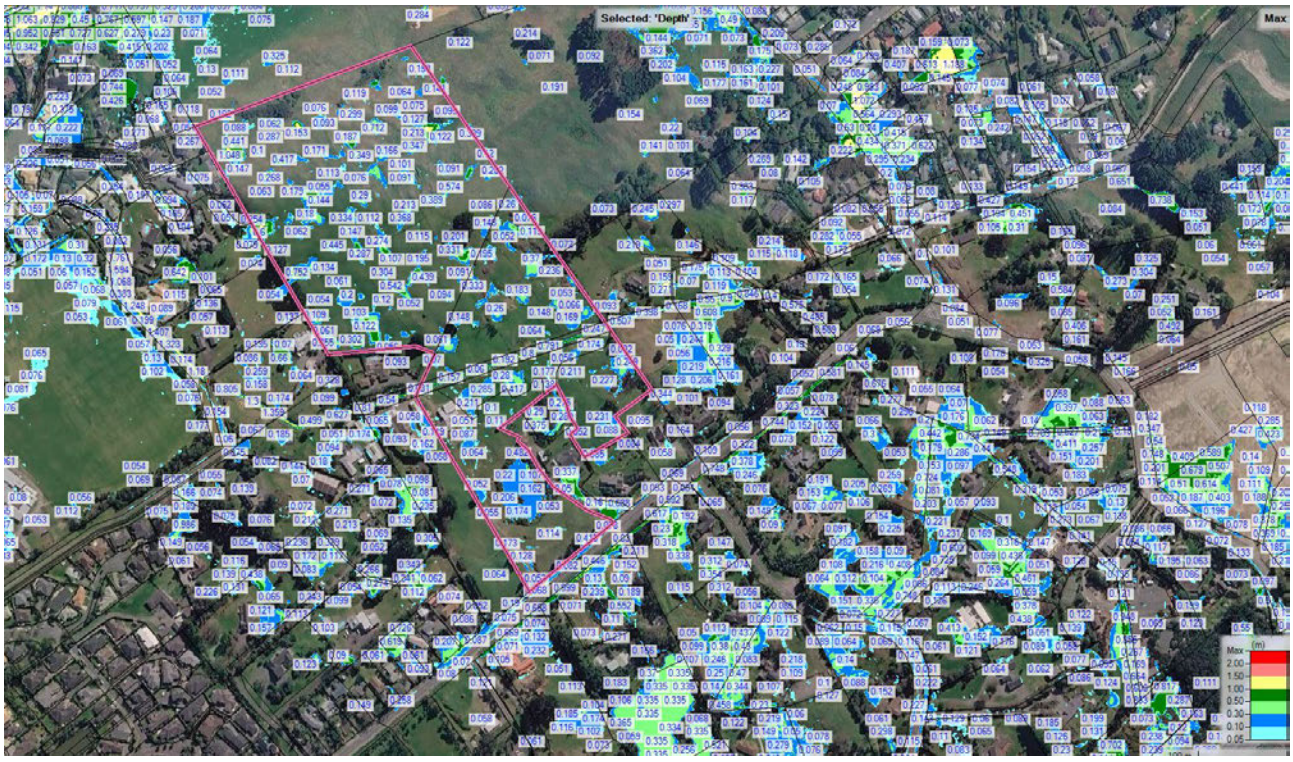


Figure 8: Flood Depths Under Existing Conditions - 100yrCC

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.

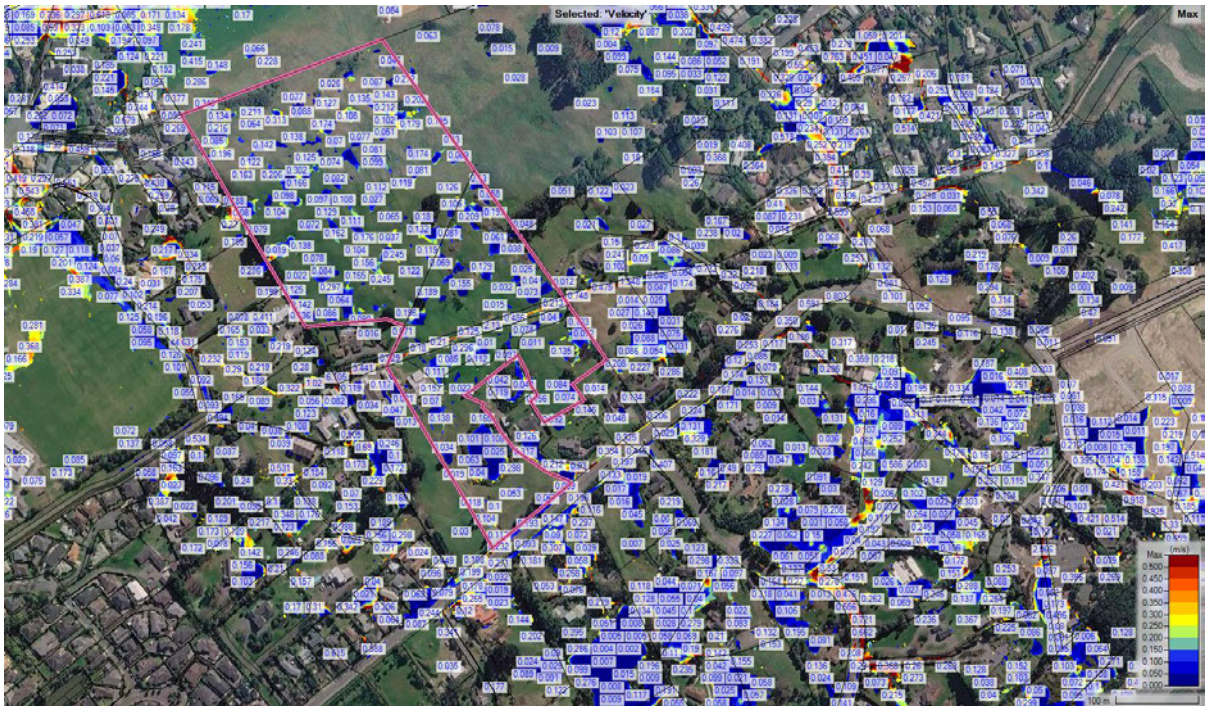


Figure 10: Flood Velocities Under Existing Conditions - 100yrCC

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.

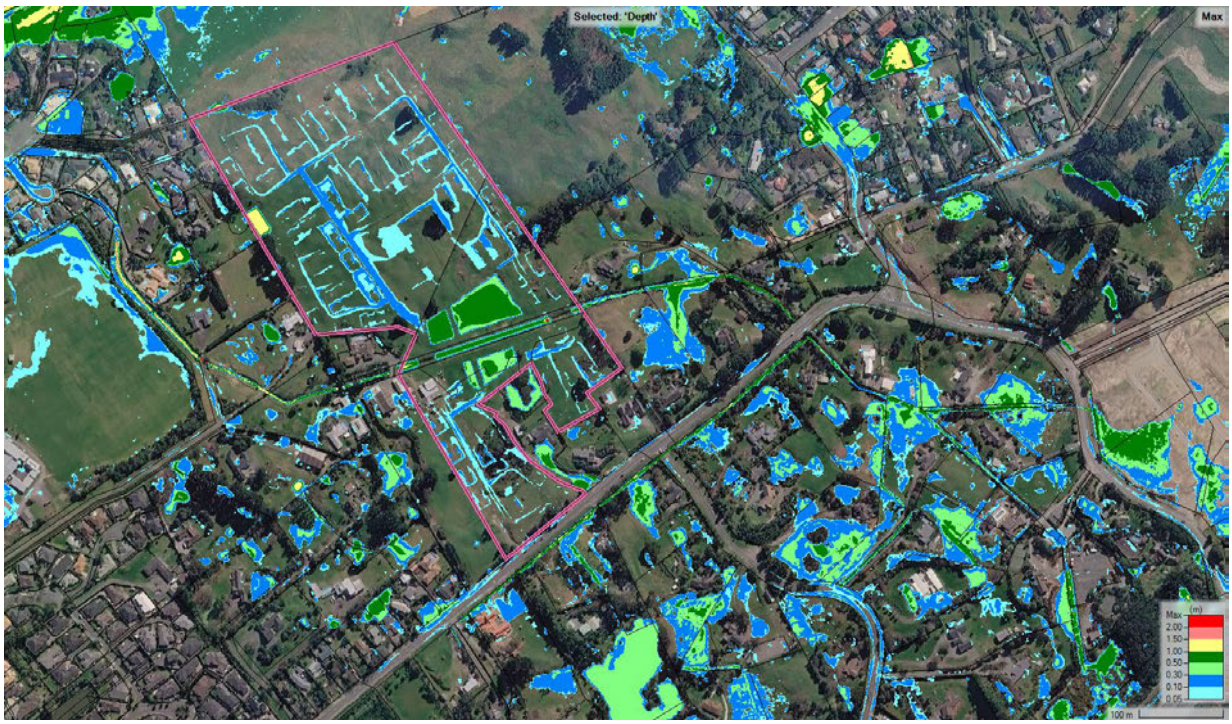


Figure 11: Flood Extents Under Proposed Conditions - 100yrCC

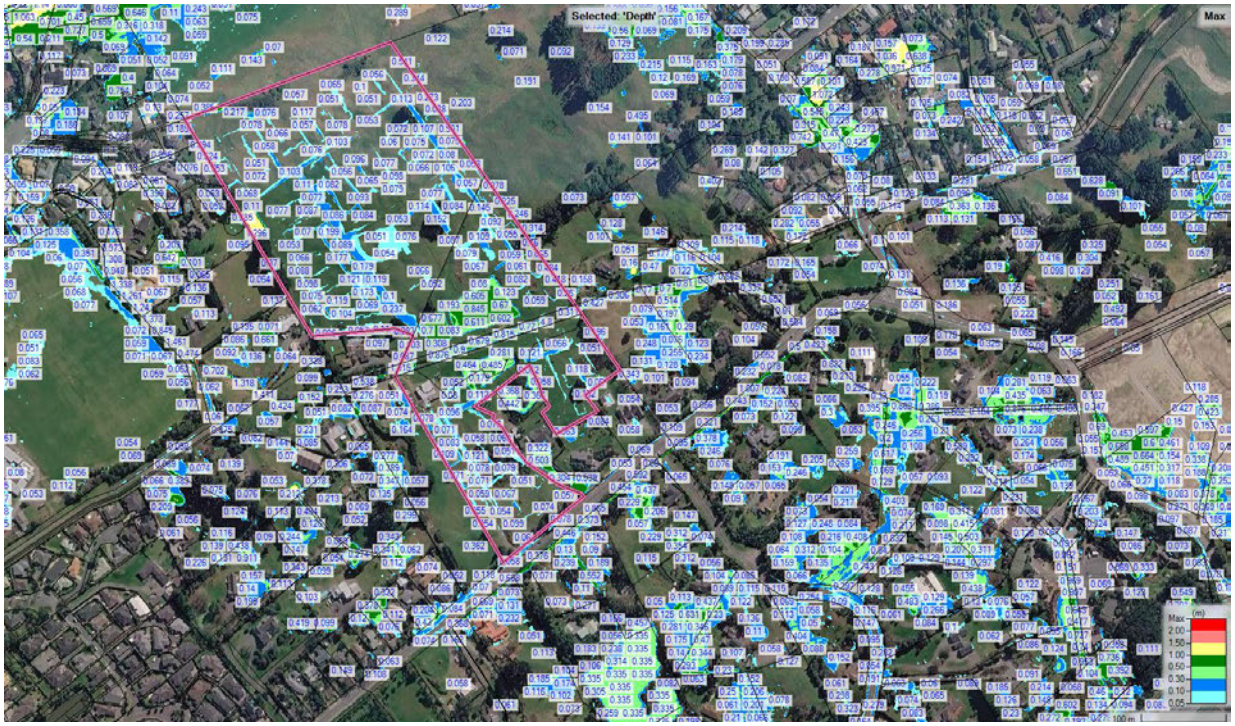


Figure 12: Flood Depths Under Proposed Conditions - 100yrCC

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.

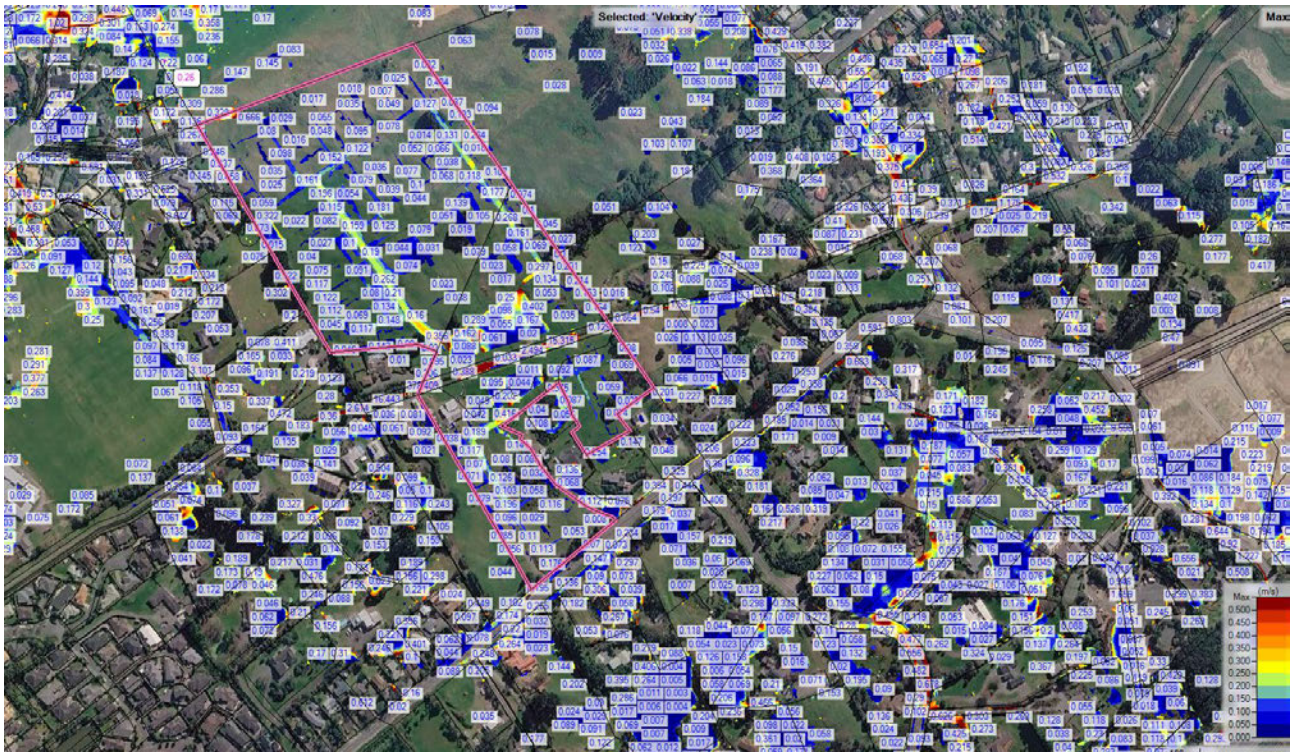


Figure 14: Flood Velocities Under Proposed Conditions - 100yr

4.3 Flood Impact Assessment

Flood impact has been assessed in terms of:

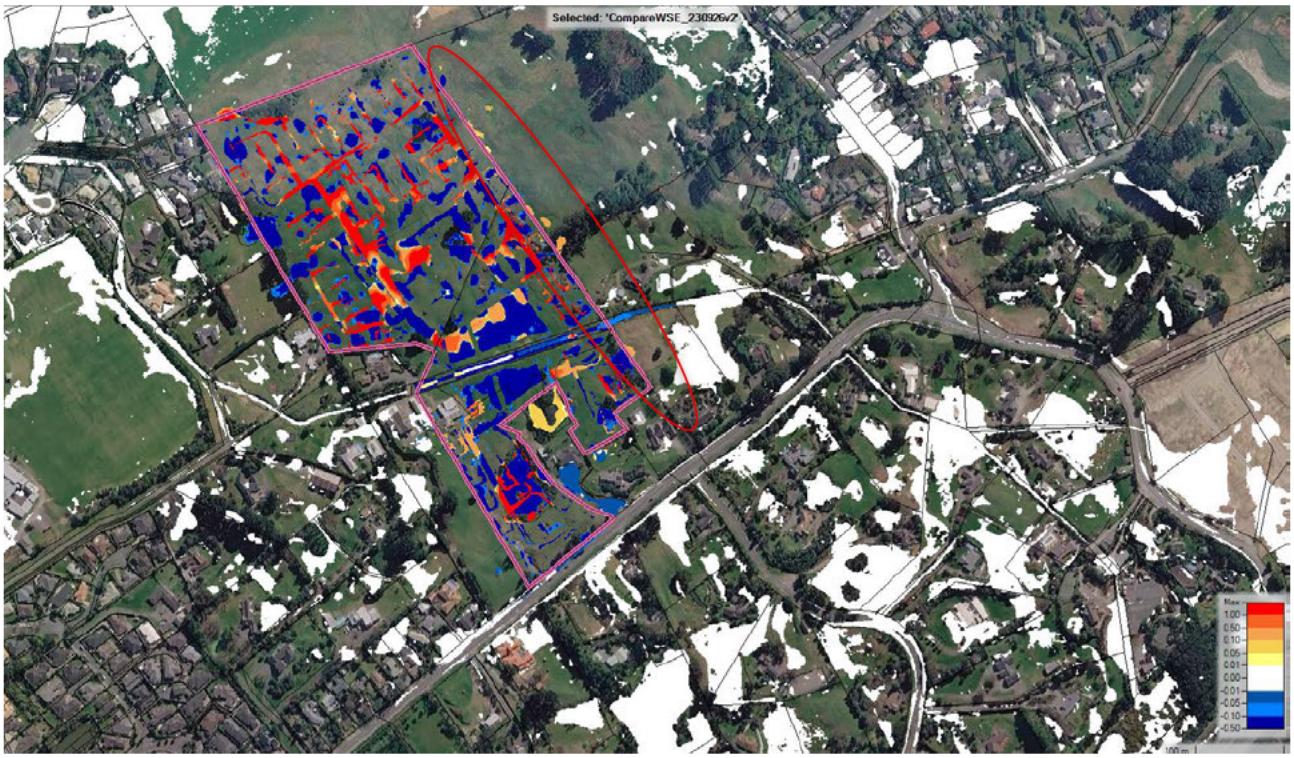
1) the change in flood levels under the proposed conditions compared with existing conditions; and
2) hydraulic neutrality.

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.

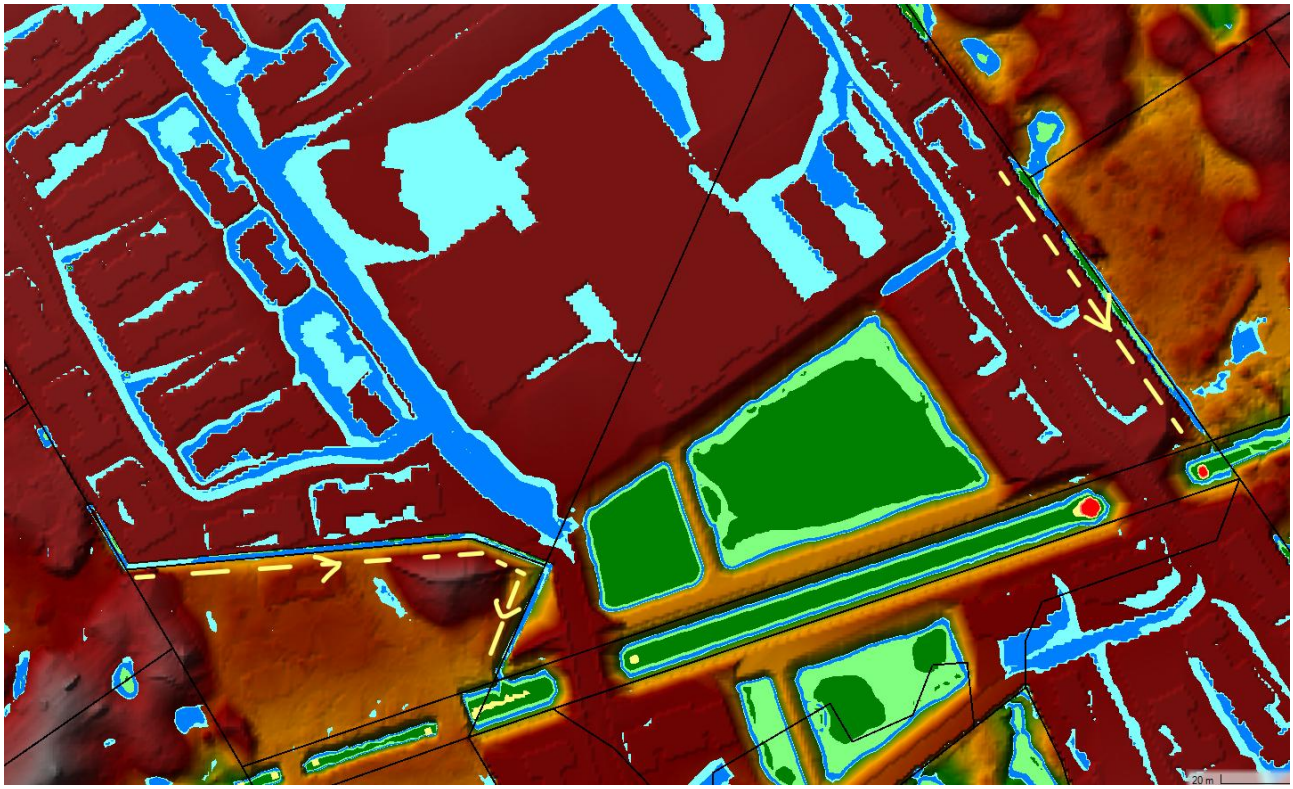


Figure 16: External flow paths entering the stream

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.

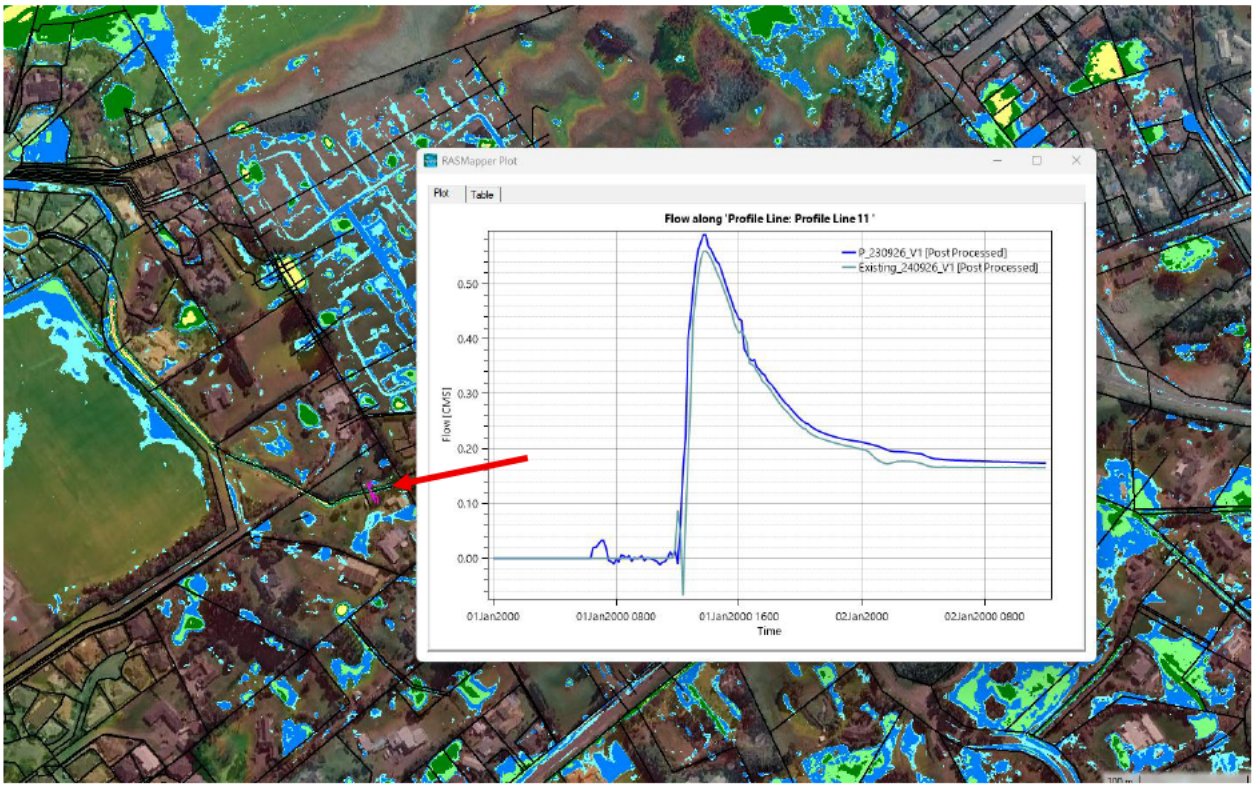


Figure 17: Existing (light blue) and proposed (dark blue) flows following the flow attenuation immediately downstream of the site- 100yrCC

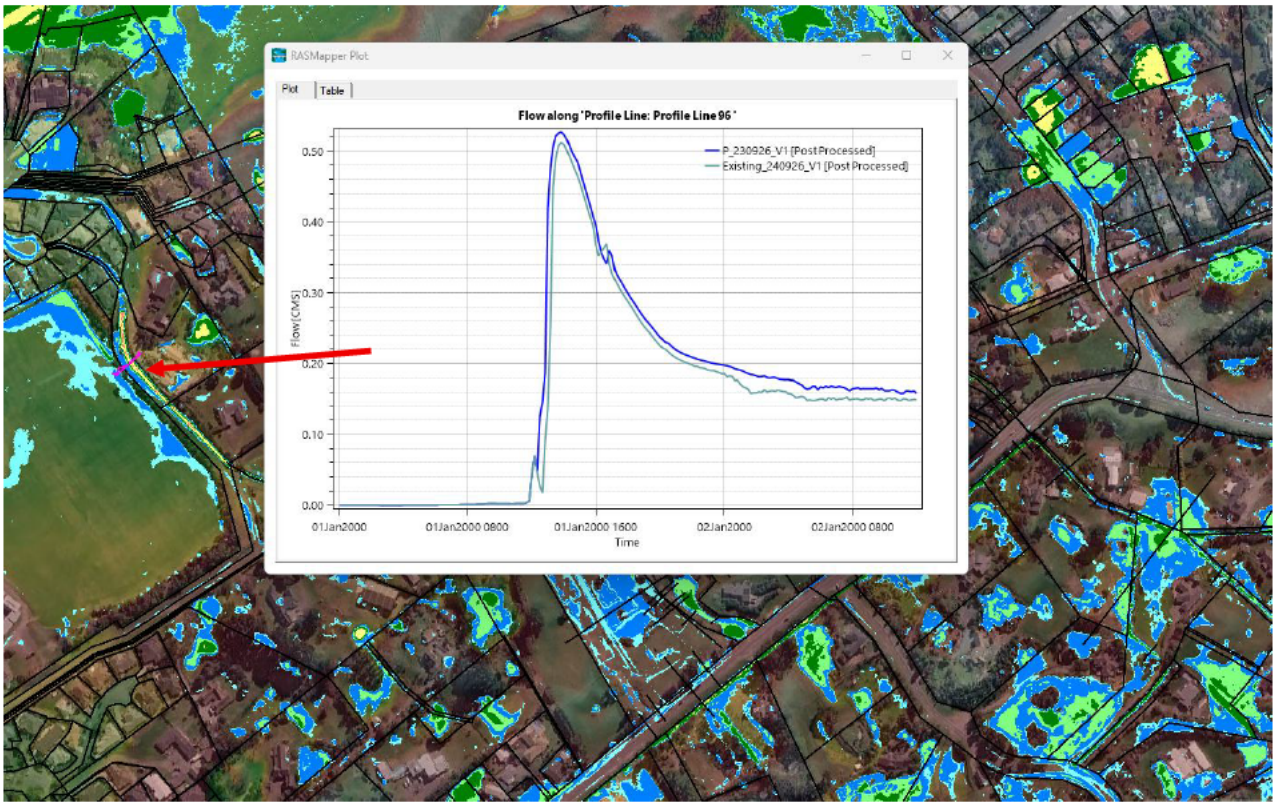


Figure 18: Existing (light blue) and proposed (dark blue) flows following the flow attenuation at the inlet location to Mazengarb stream- 100yrCC

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.

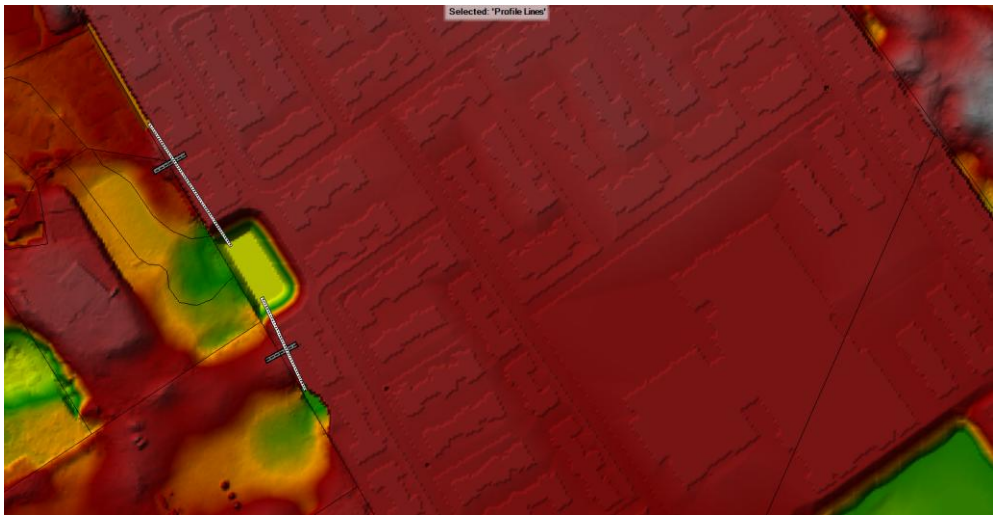


Figure 19: Western secondary culverts

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.

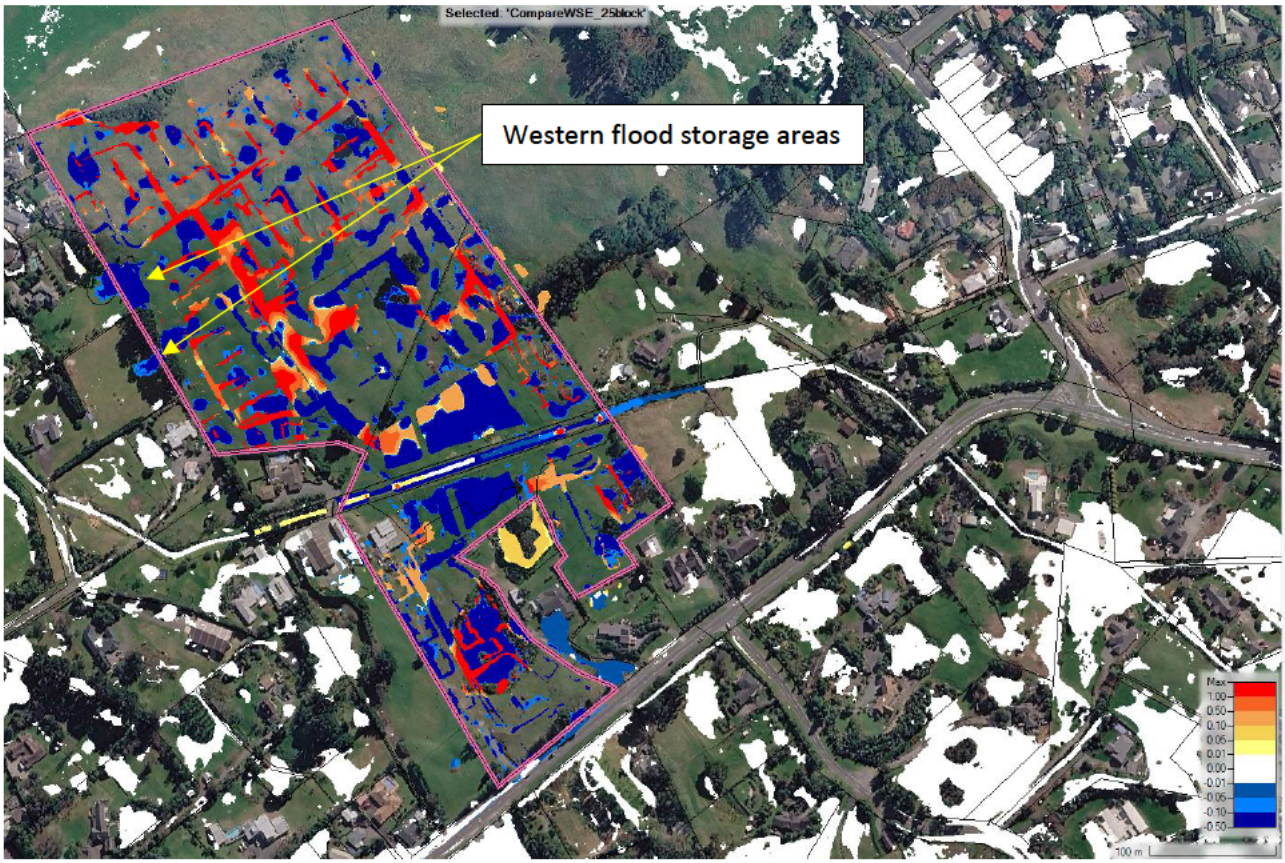


Figure 20: Flood level difference map considering 25% blockage on western culvert

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.

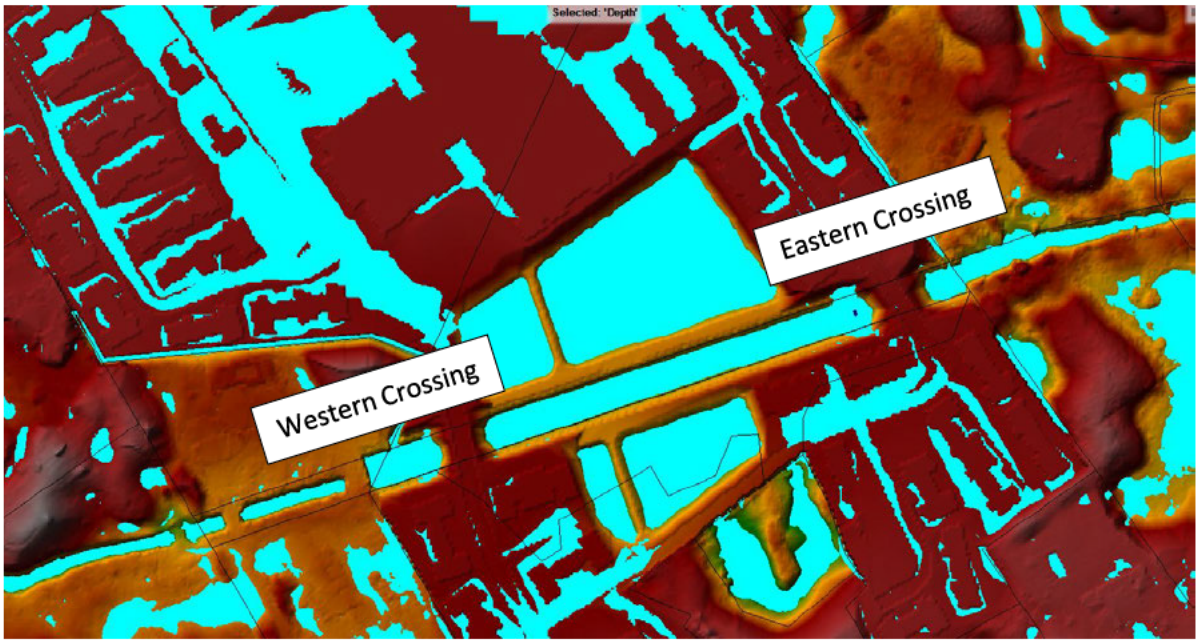


Figure 21: 500yr ARI flood extents

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.

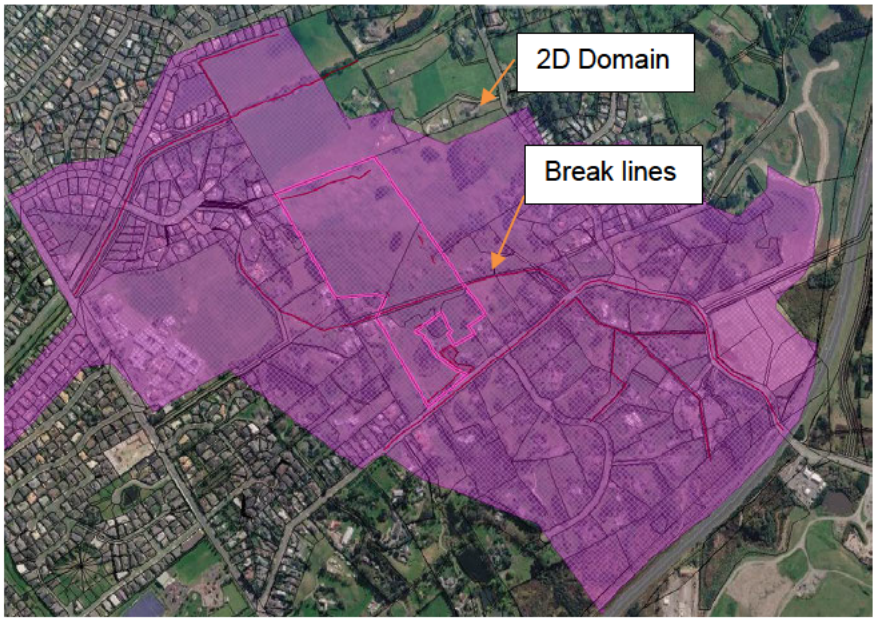


FIGURE 1: 2D FLOW AREA

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.

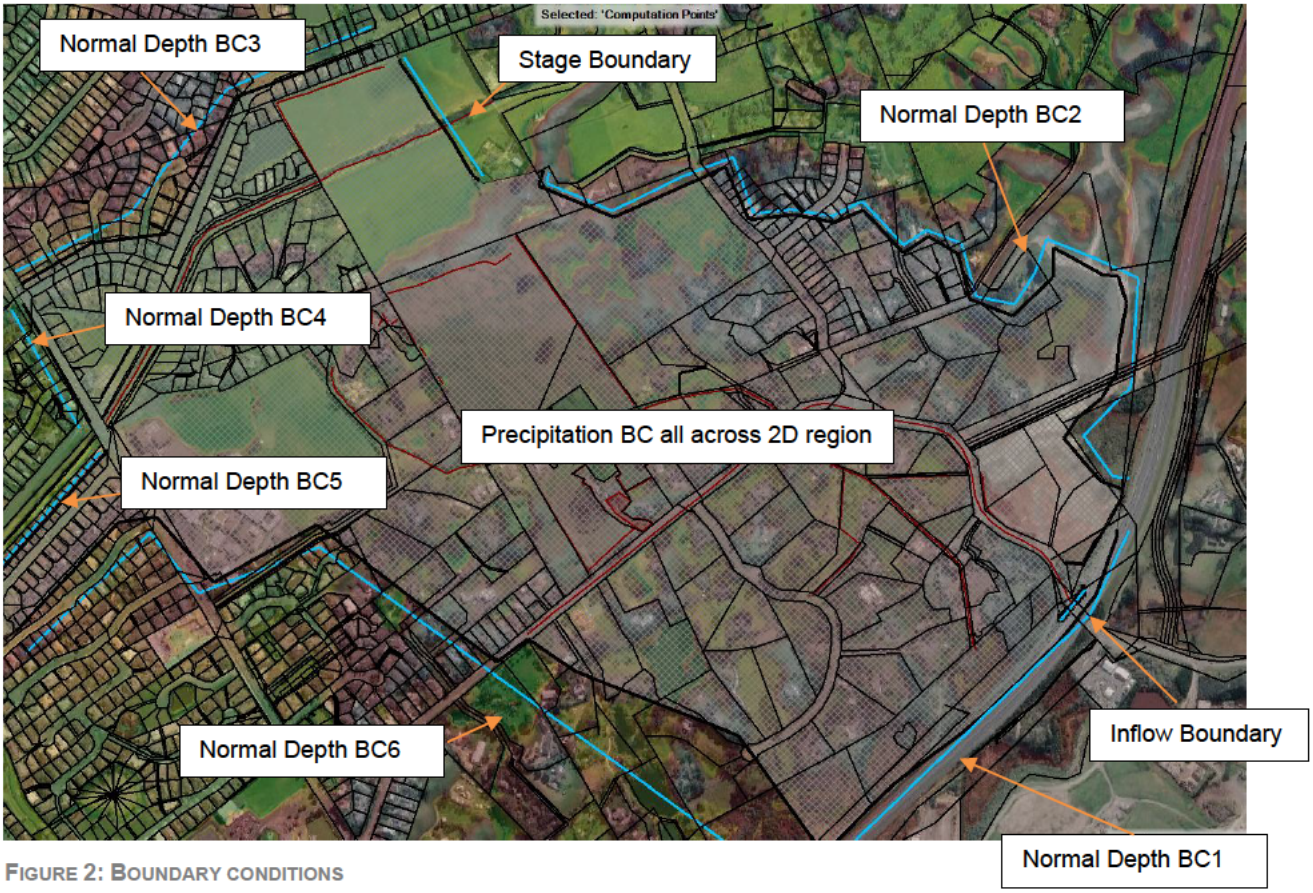


FIGURE 2: BOUNDARY CONDITIONS

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.

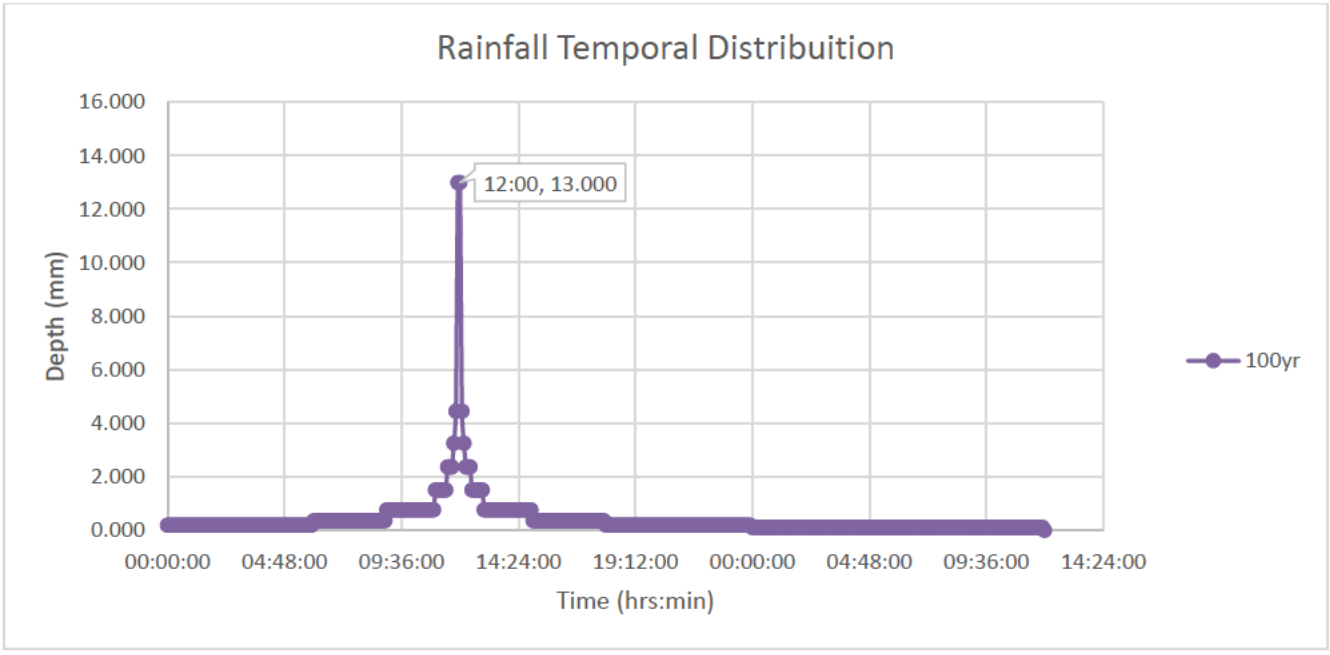


FIGURE 3: RAINFALL PATTERN 100YRCC DESIGN EVENT

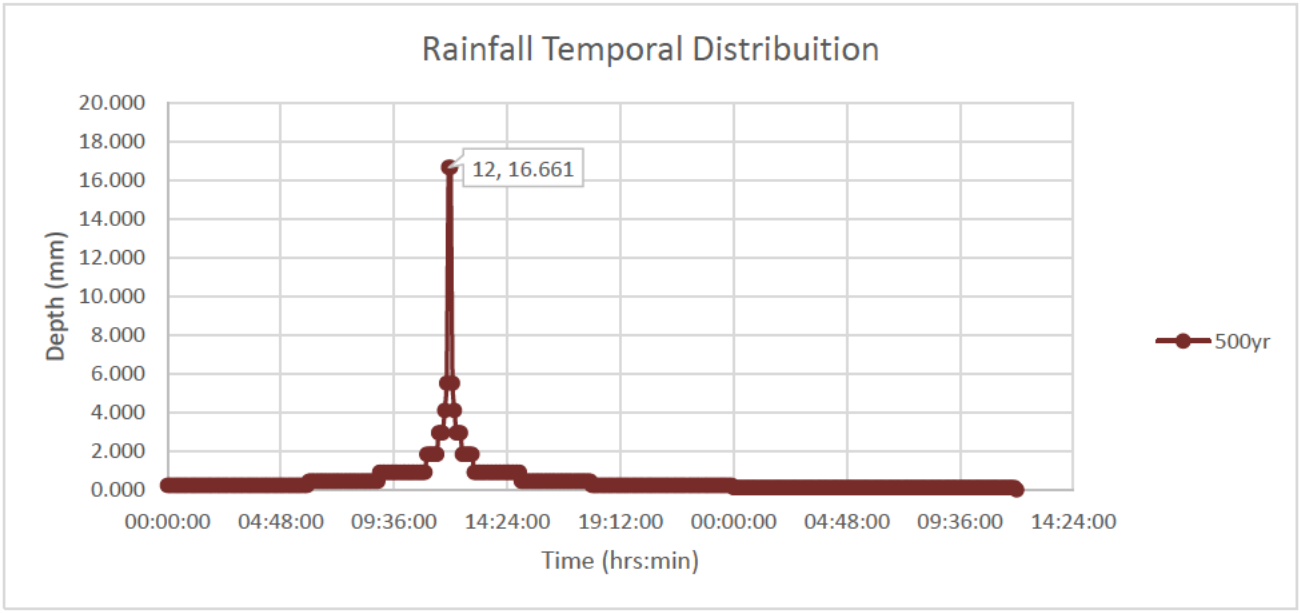


FIGURE 4: RAINFALL PATTERN 500YRCC DESIGN EVENT

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.

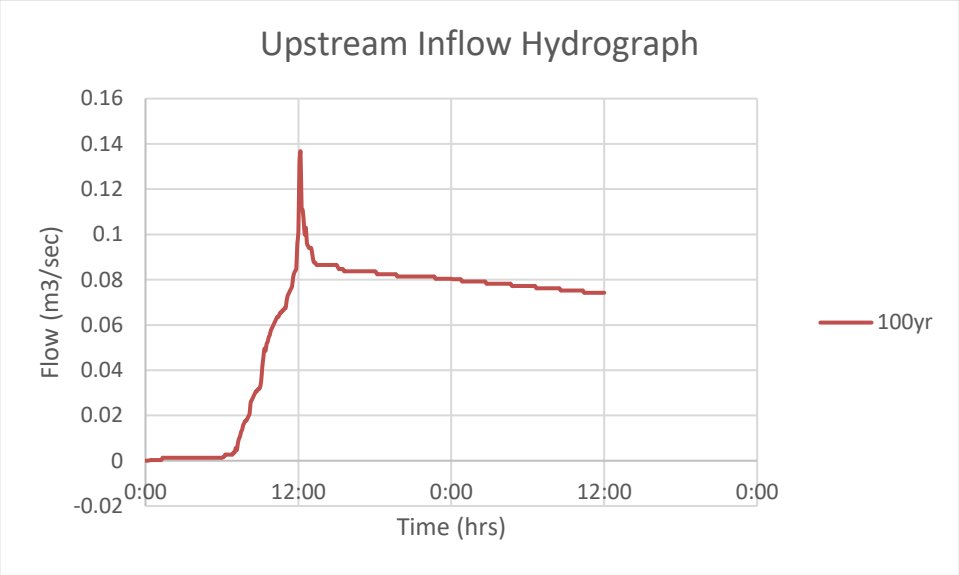


FIGURE 7: FLOW HYDROGRAPH 100YRCC DESIGN EVENT

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)

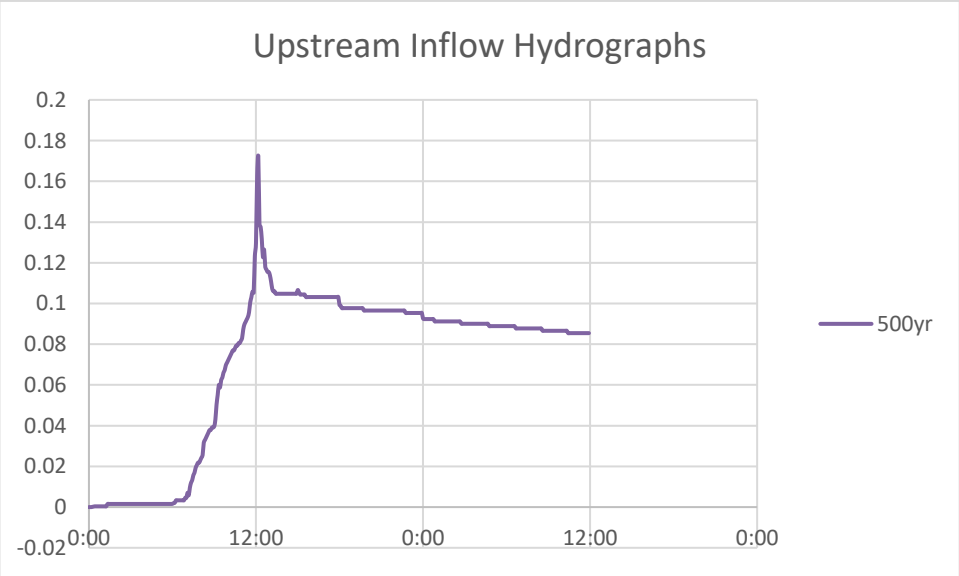


FIGURE 8: FLOW HYDROGRAPH 100YRCC DESIGN EVENT

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.

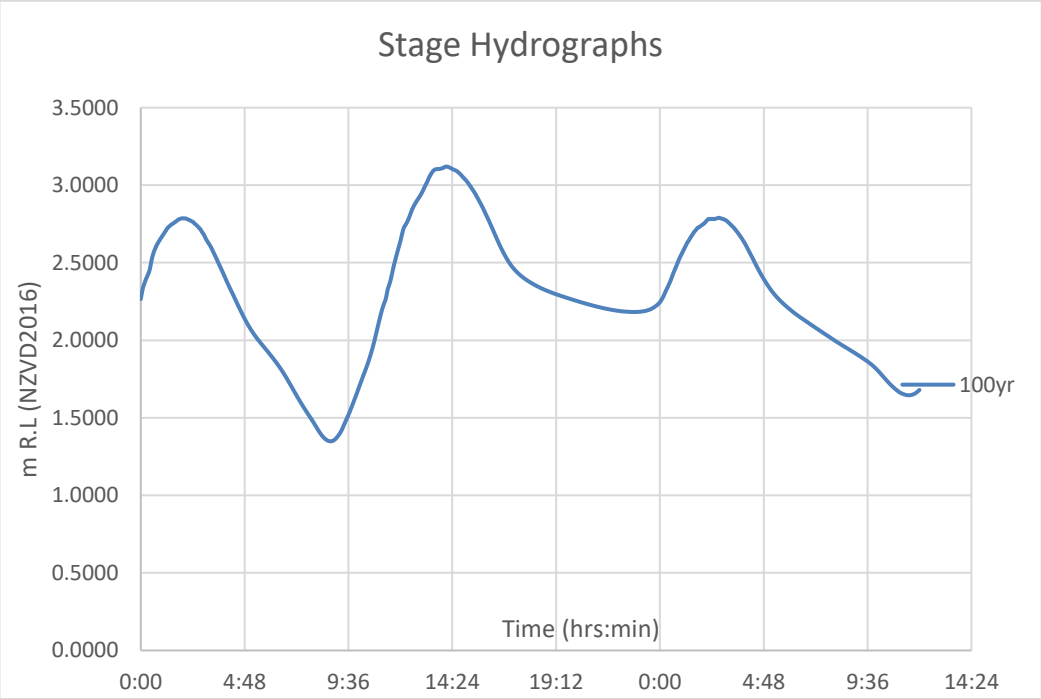


FIGURE 9: STAGE HYDROGRAPHS CORRESPONDING TO 100YRCC EVENT

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.

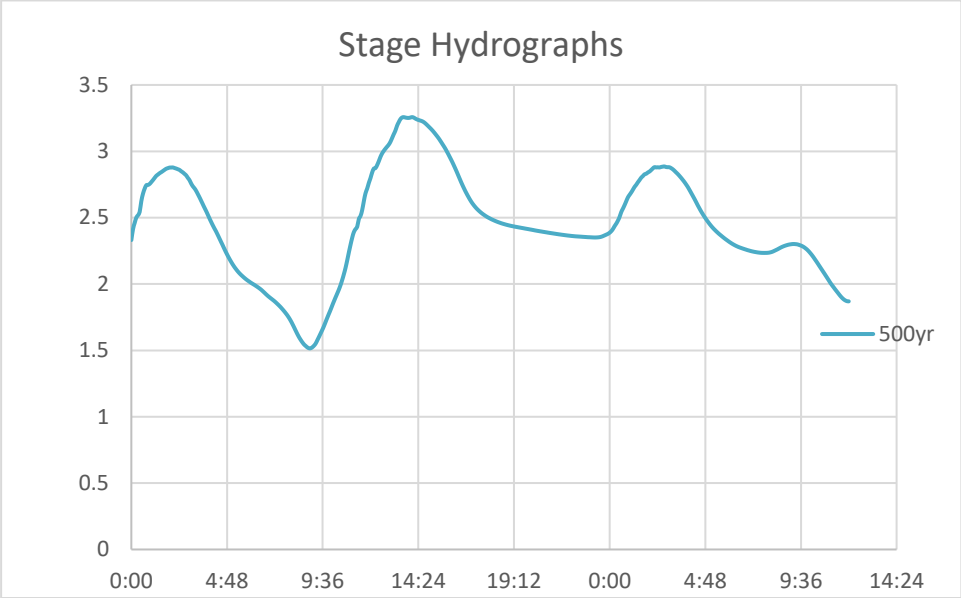


FIGURE 10: STAGE HYDROGRAPHS CORRESPONDING TO 100YRCC EVENT

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.

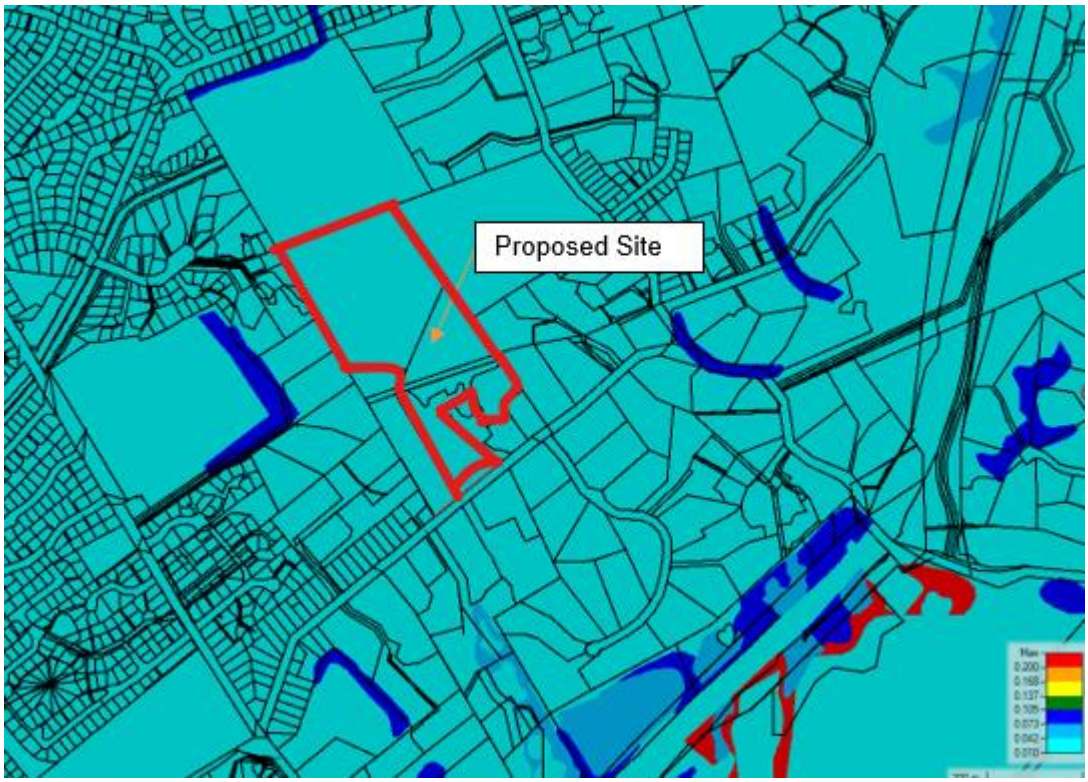


FIGURE 11: VARIATION OF MANNINGS ROUGHNESS-EXISTING

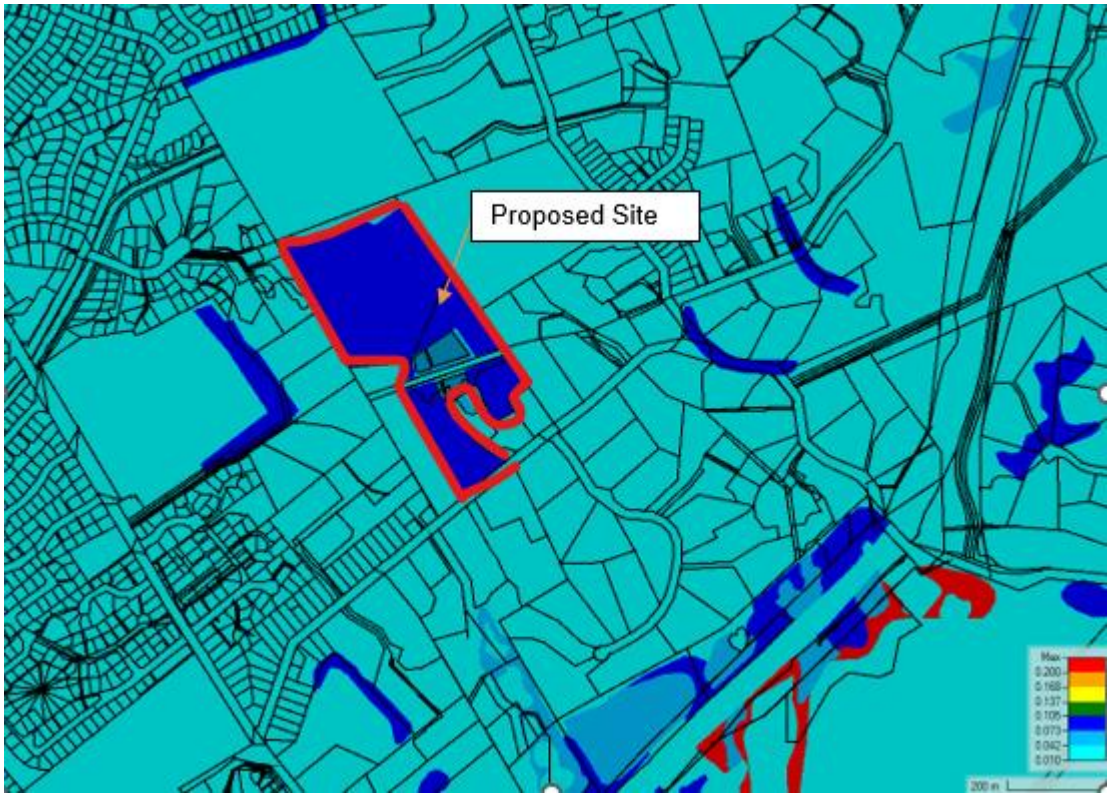
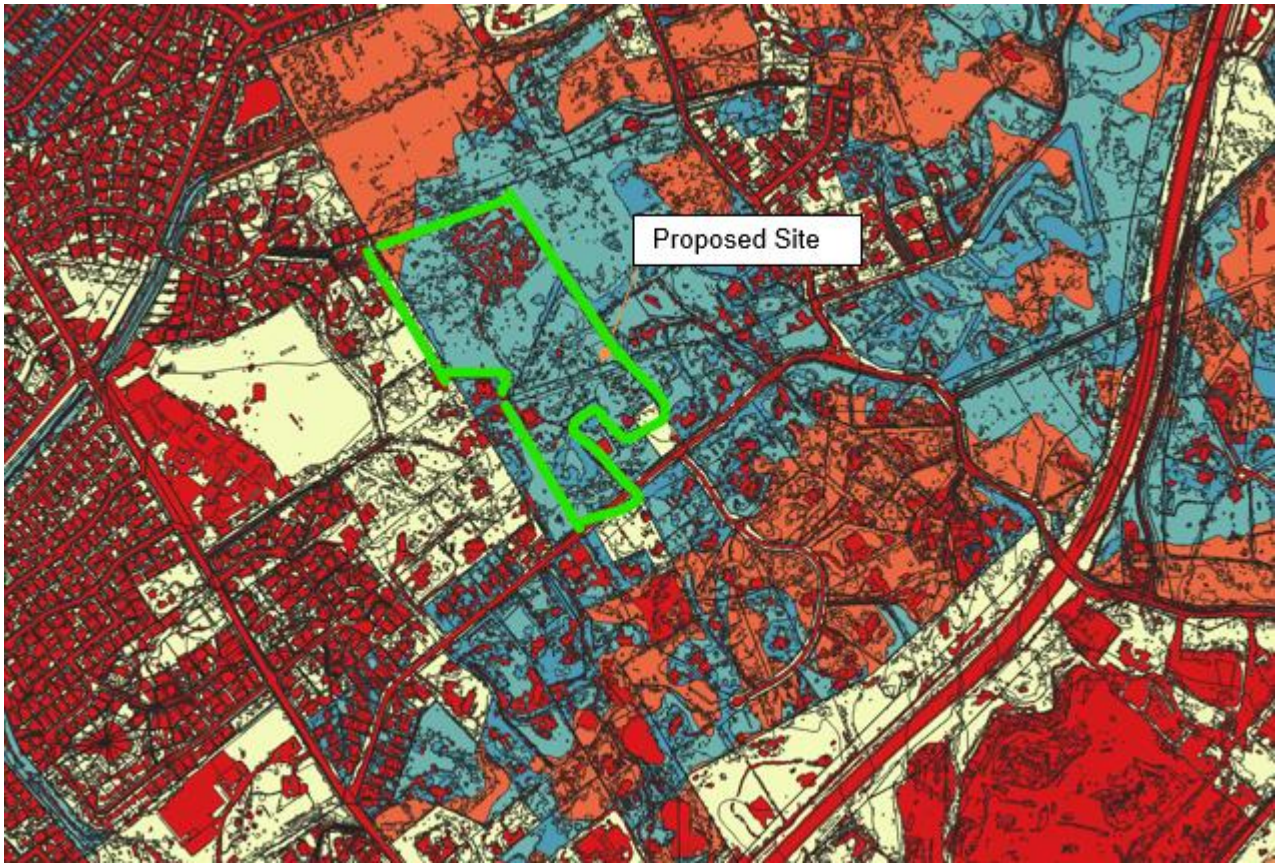


FIGURE 12: VARIATION OF MANNINGS ROUGHNESS-PROPOSED

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.



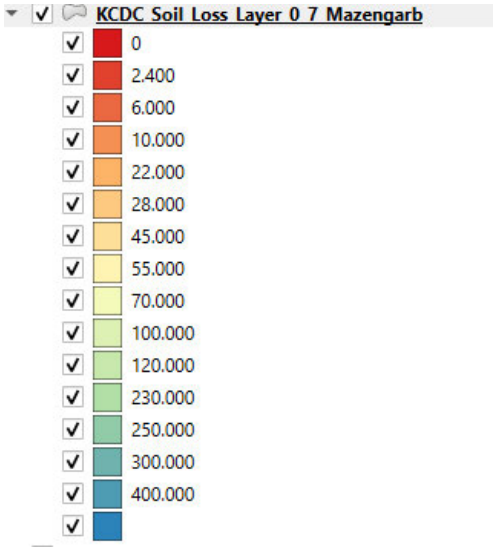


FIGURE 13 SPATIAL VARIATION OF THE PERCOLATION RATE IN MM/HR

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.

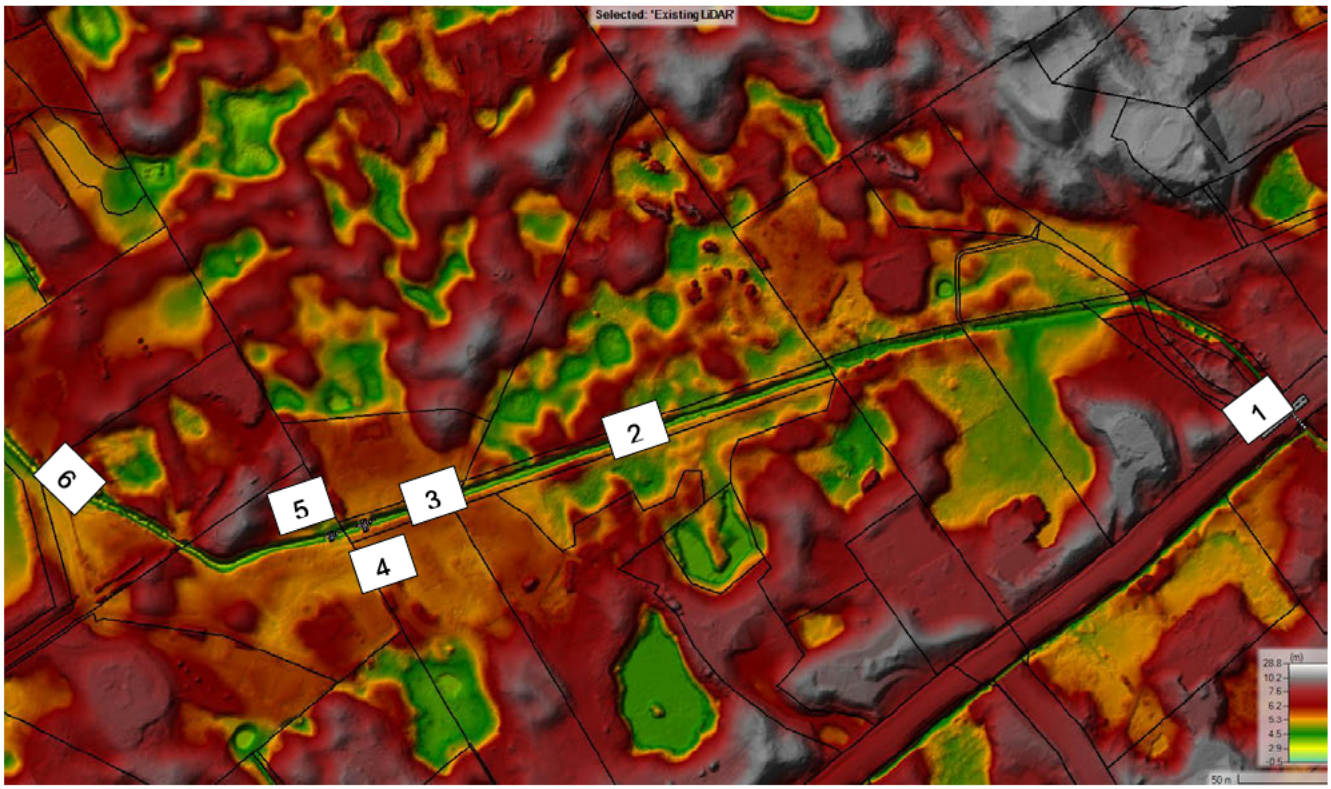


FIGURE 14: CULVERTS

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

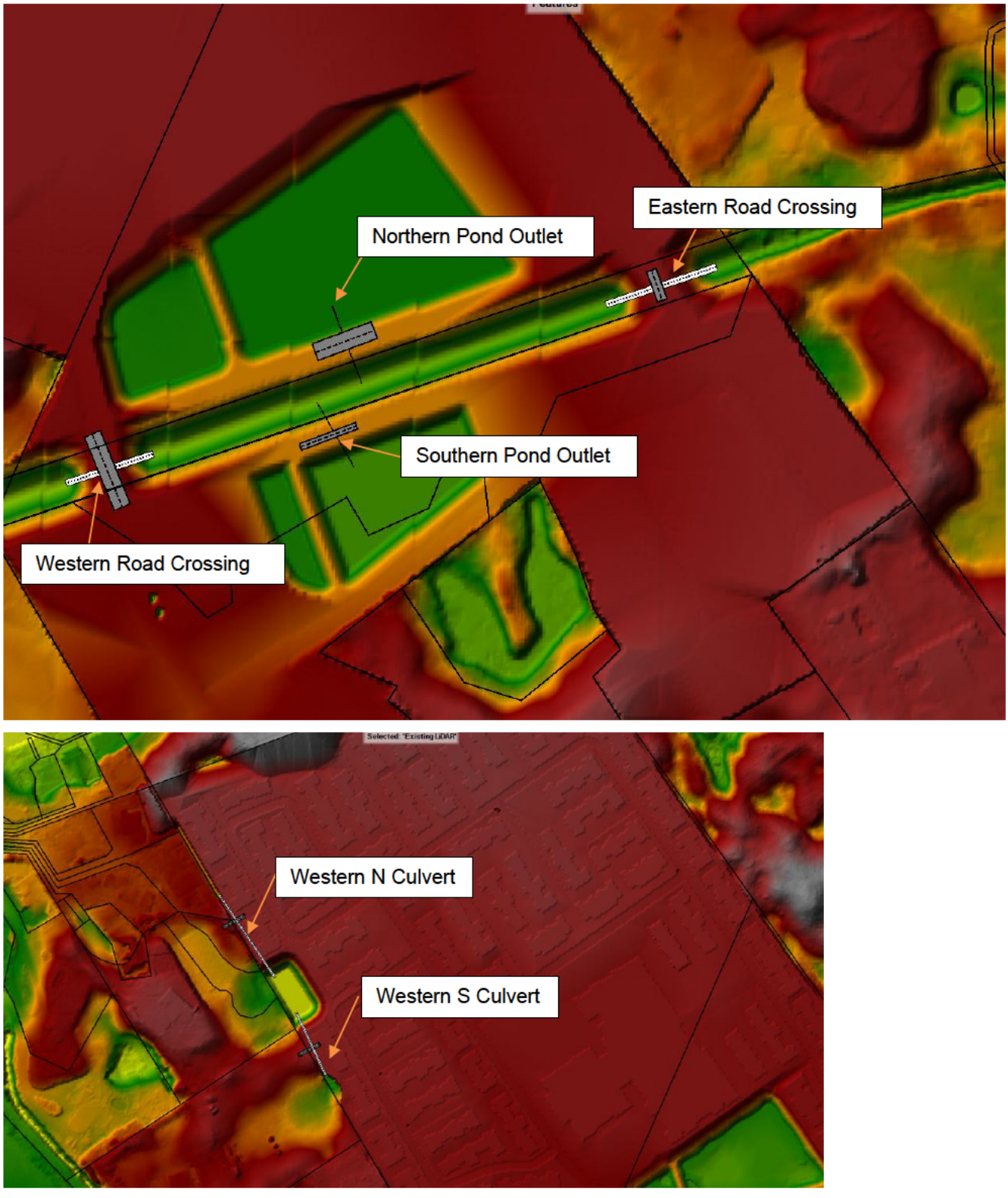


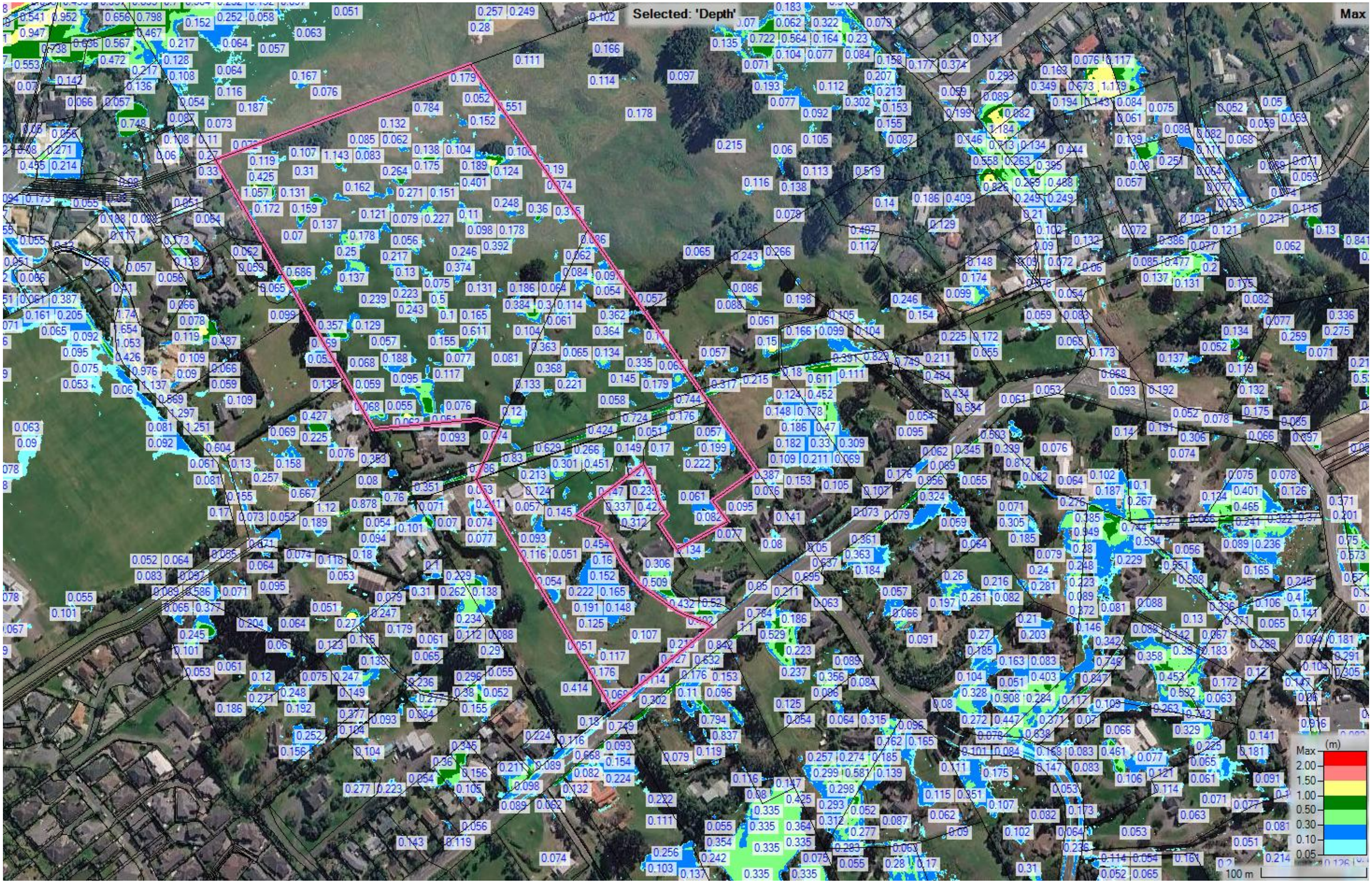
FIGURE 15: PROPOSED CULVERTS

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

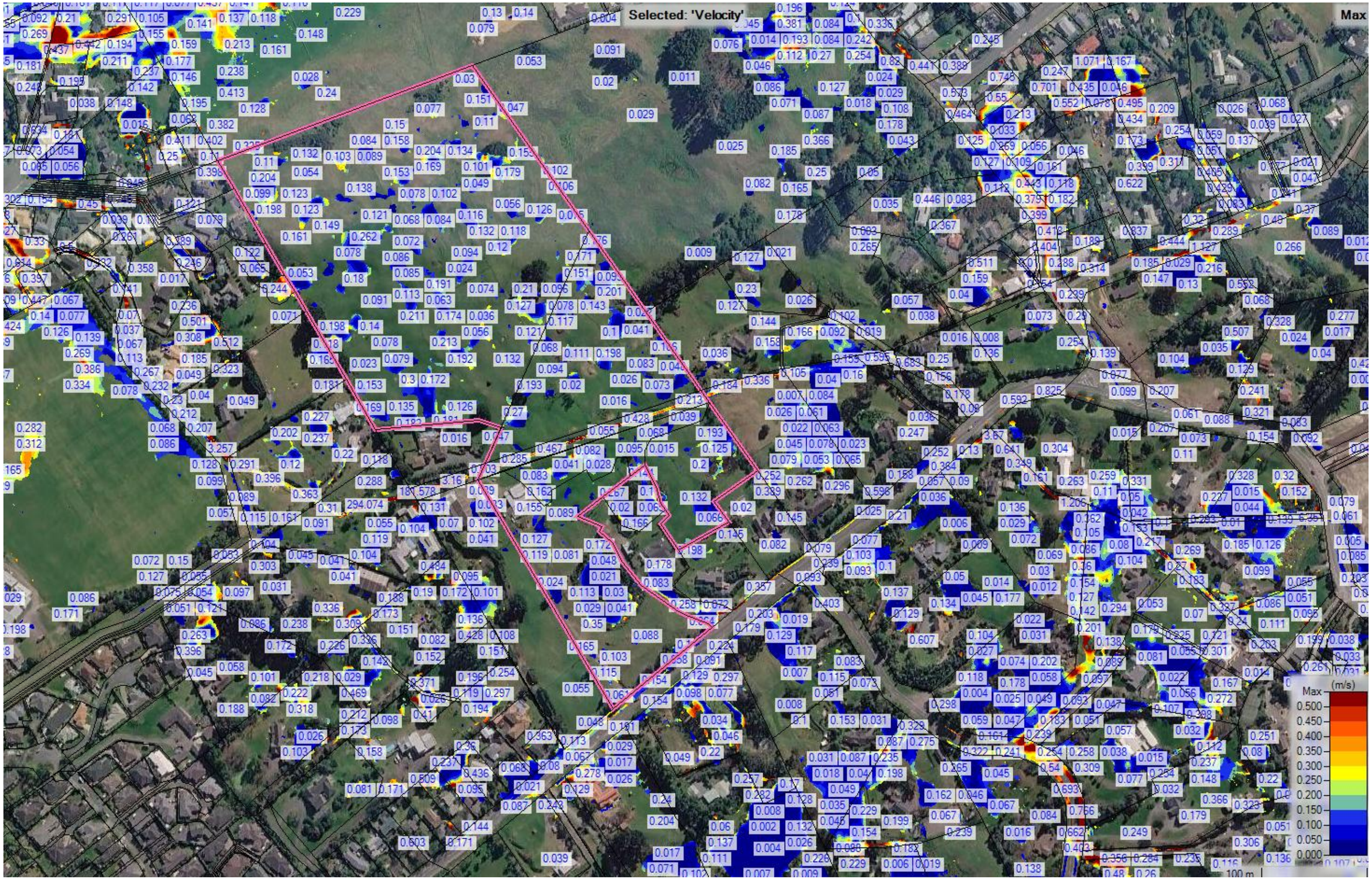
100yrCC- Existing Scenario- Flood Depths (m)



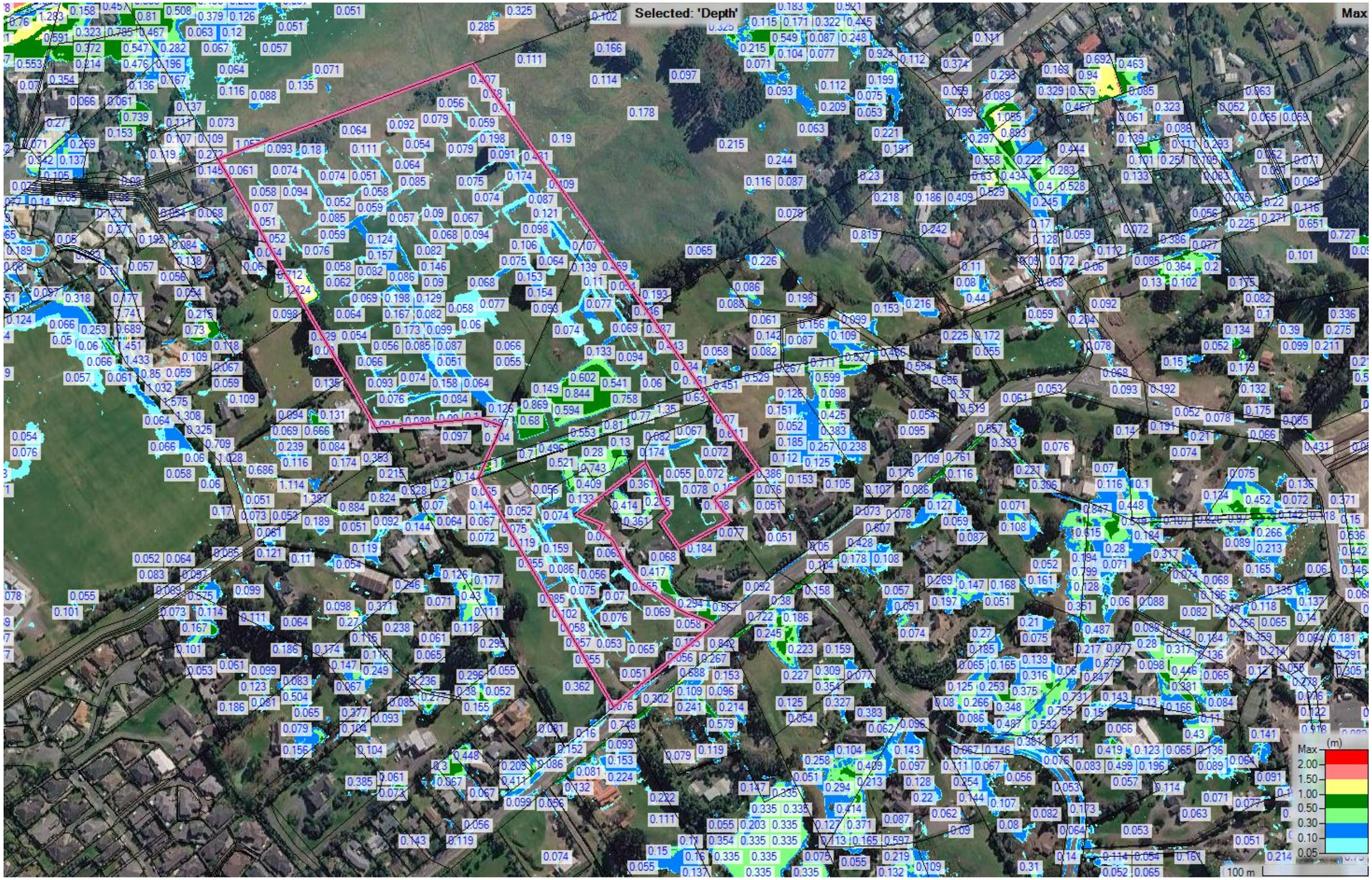
100rCC- Existing Scenario- Flood Levels (m R.L NZVD 2016)



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

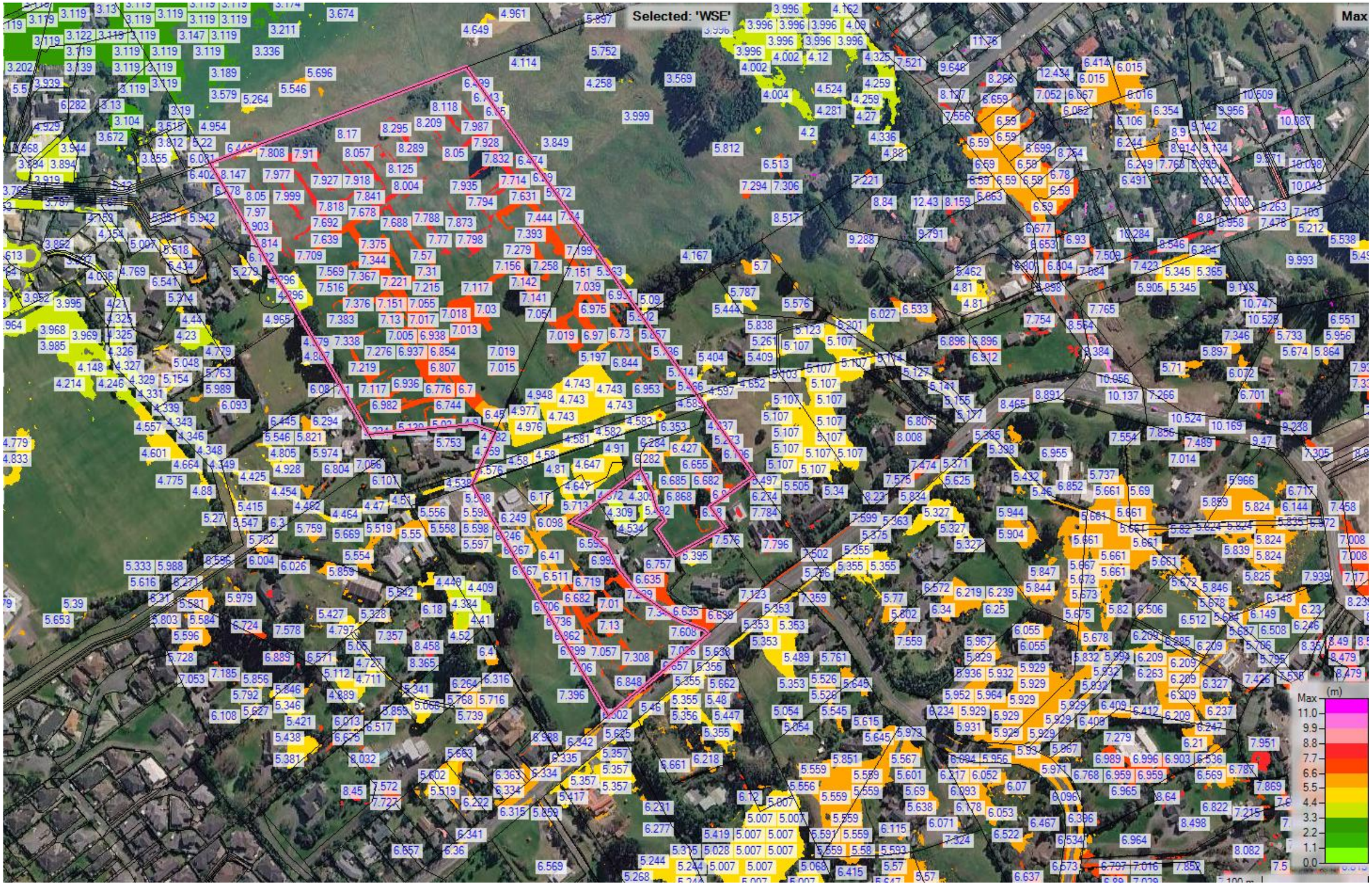


100yrCC- Proposed Scenario[3]- Flood Depths (m)

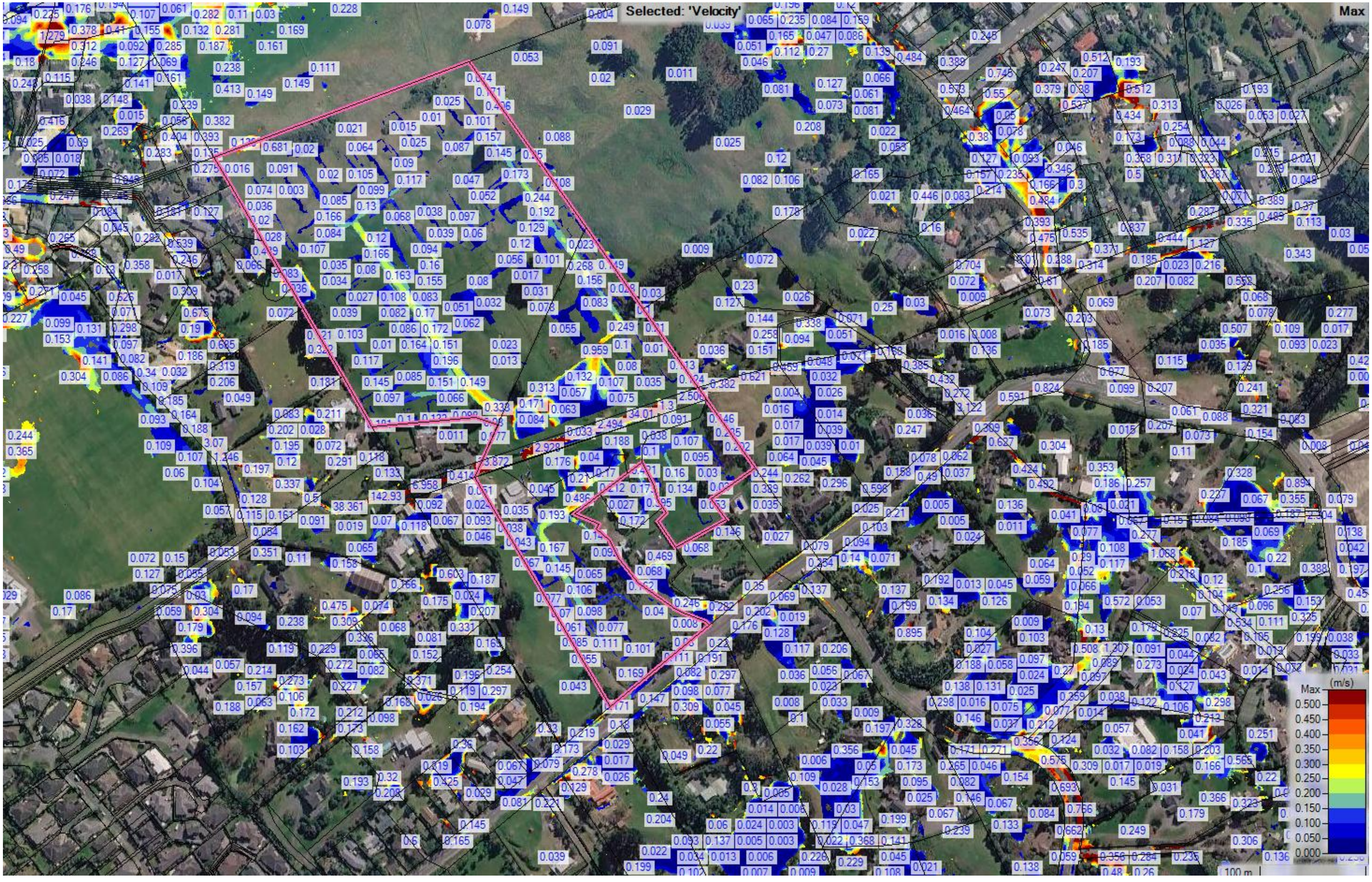


[3] Proposed surface dated 23.09.26

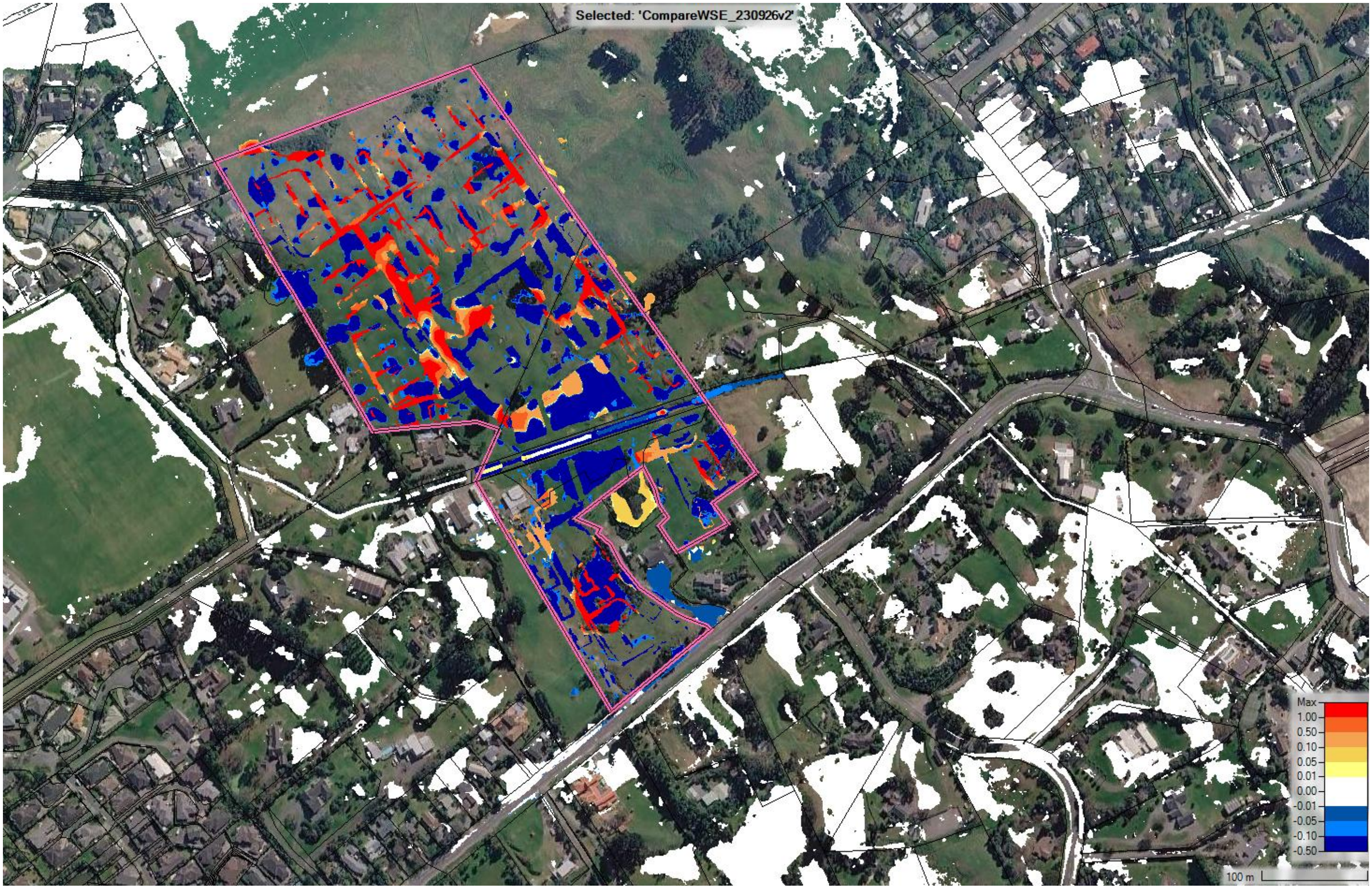
100rCC- Proposed Scenario- Flood Levels (m R.L NZVD 2016)



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.

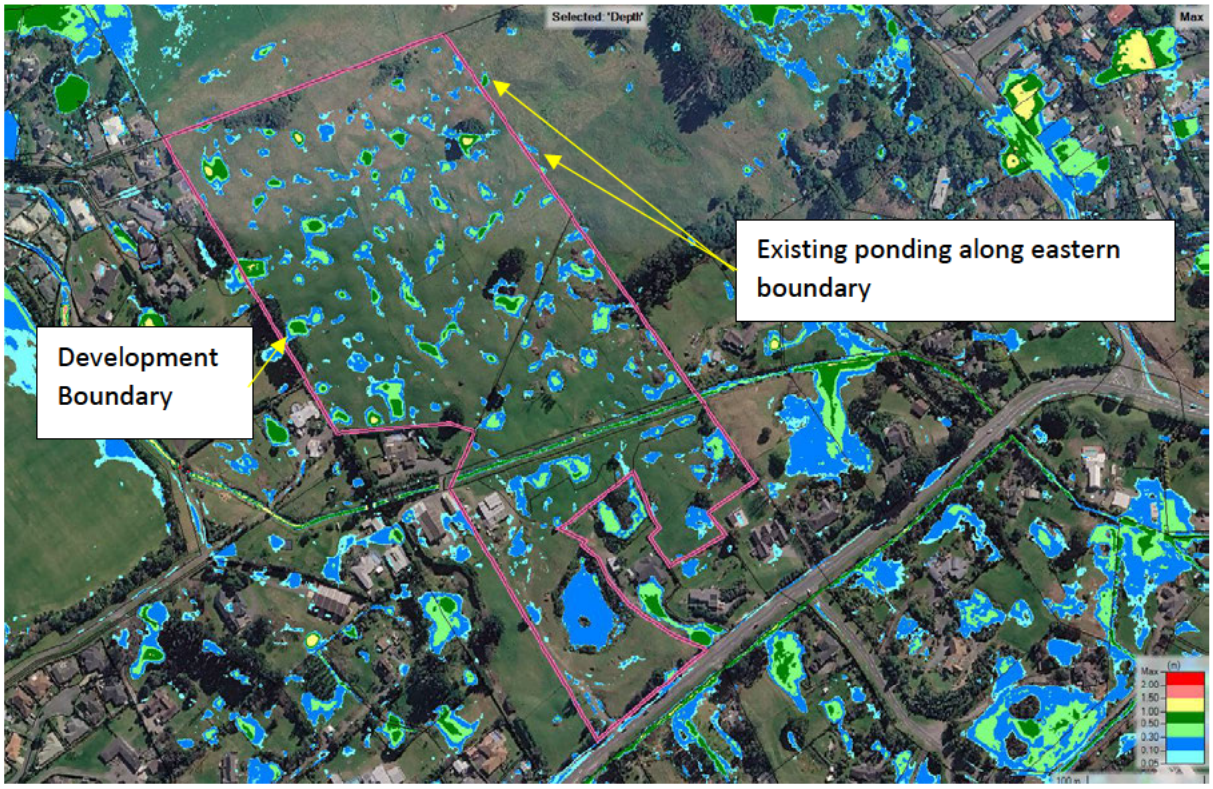


Figure 1: Flood extents under existing conditions - 100yrCC

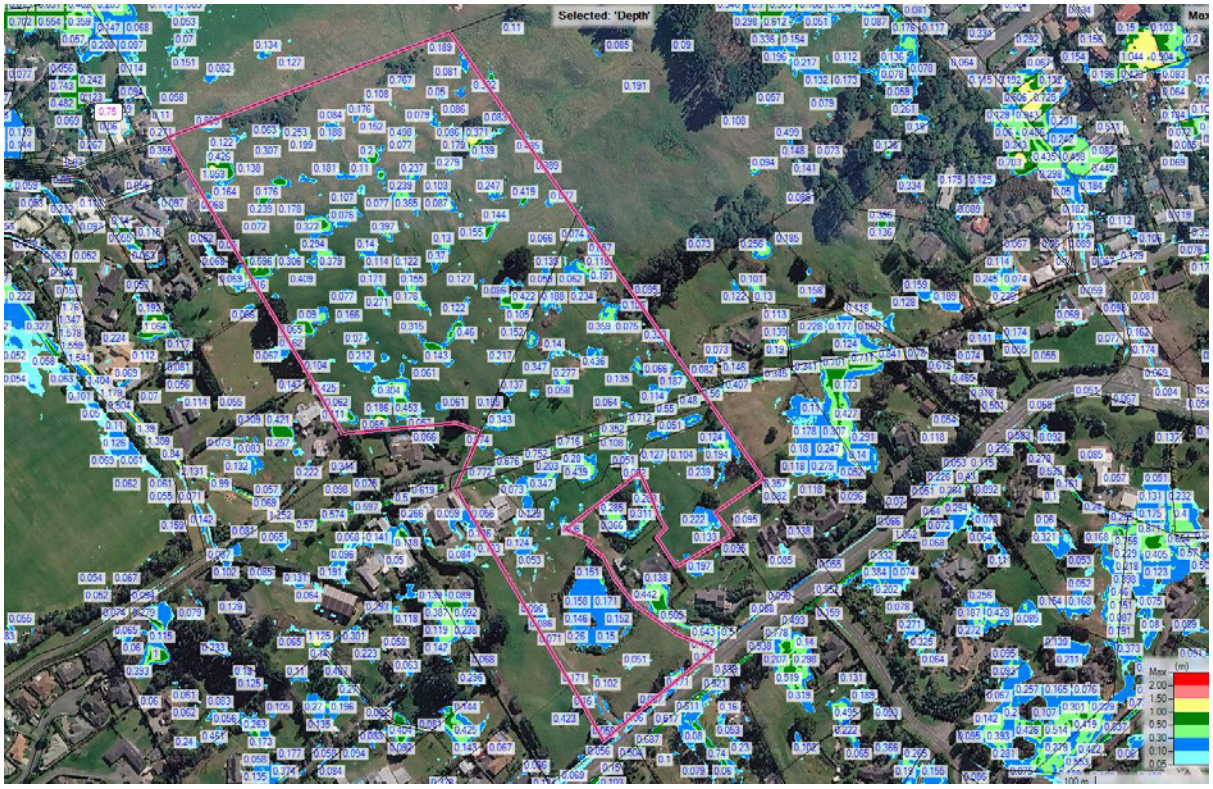


Figure 2: Flood depths under existing conditions - 100yrCC

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.

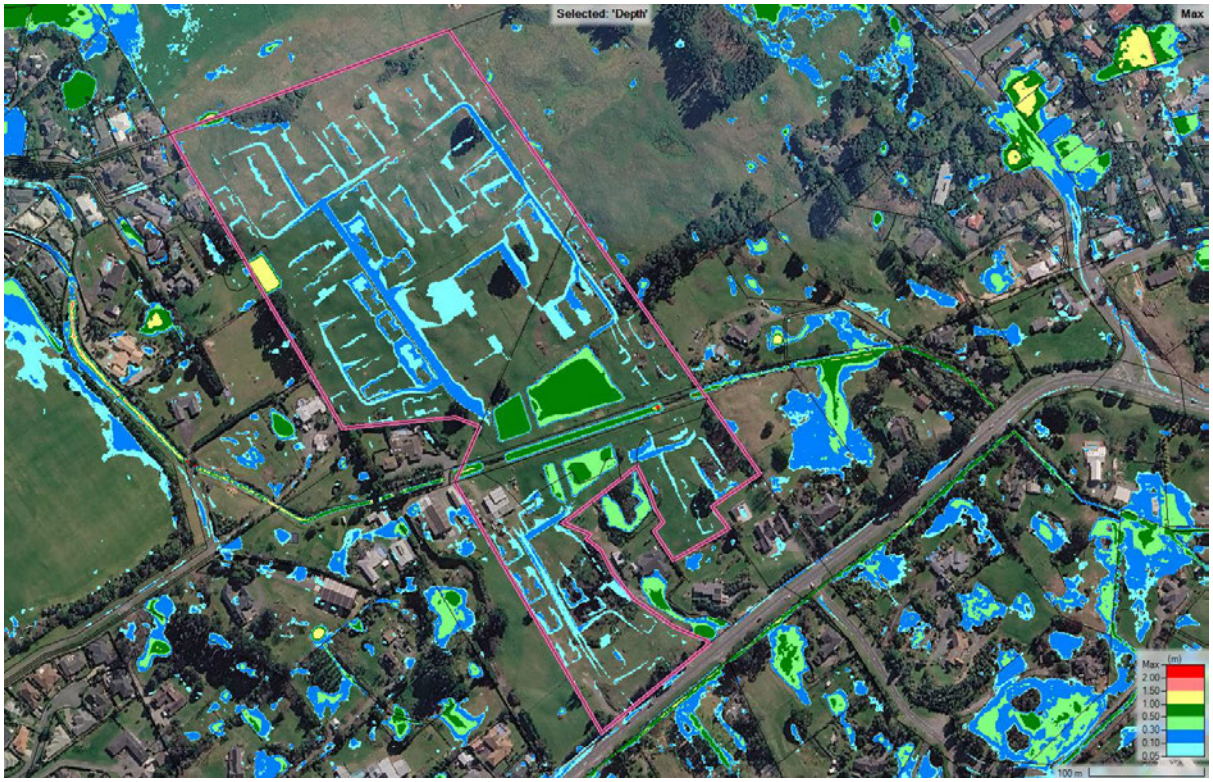


Figure 3: Flood extents under proposed conditions - 100yCCC

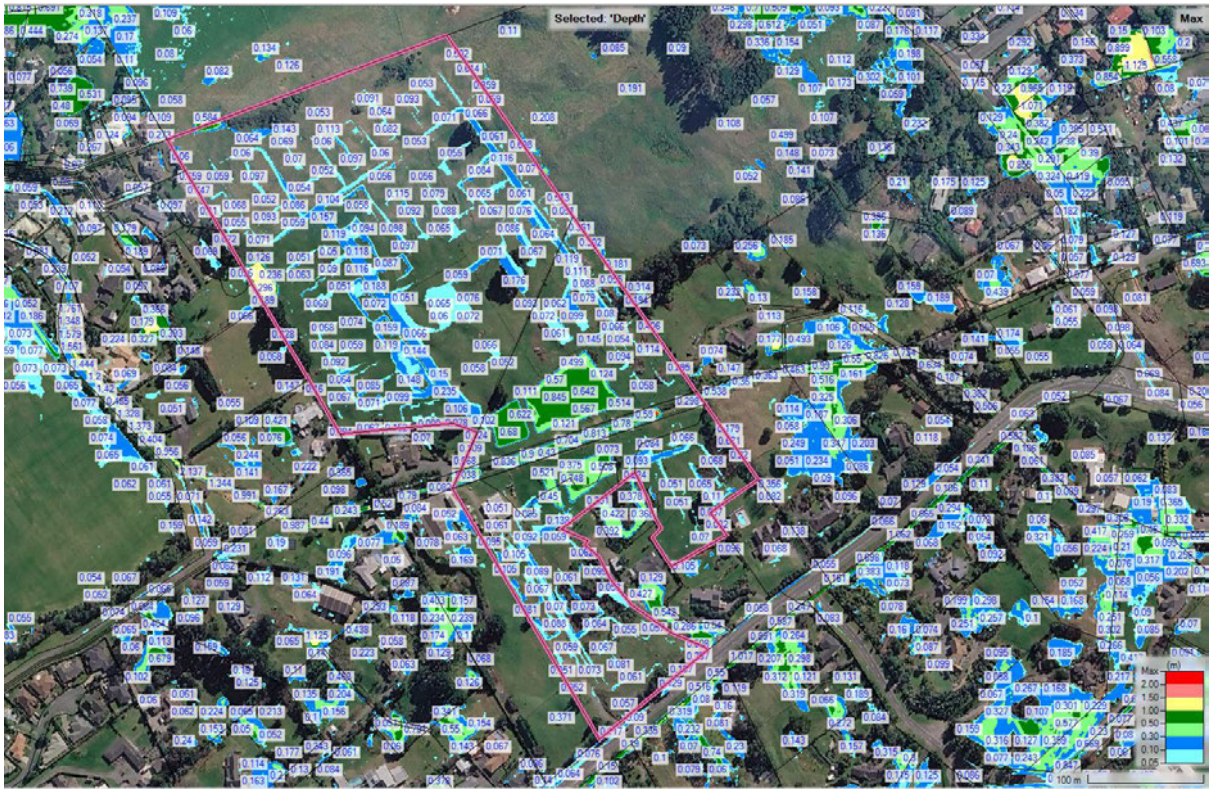


Figure 4: Flood depths under proposed conditions - 100yrCC

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.

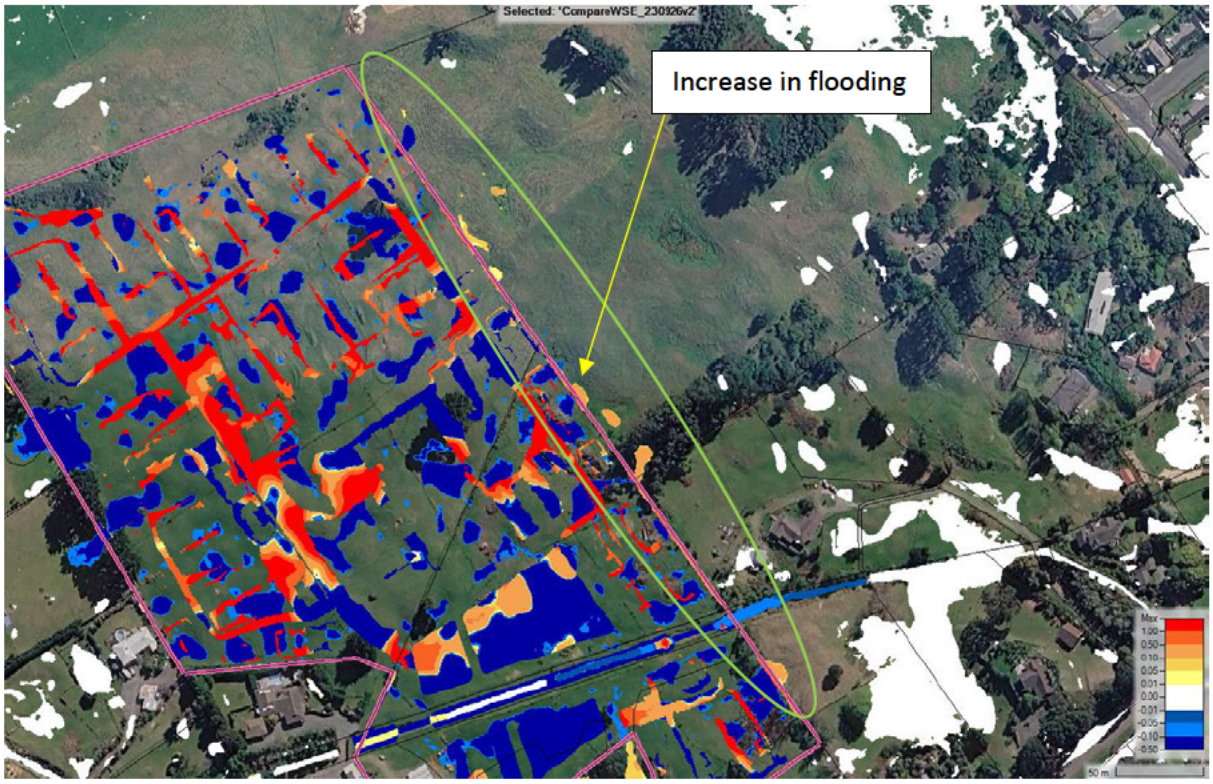


Figure 5: Increase in flood level by 200mm under proposed conditions within Eastern Site Boundary

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.

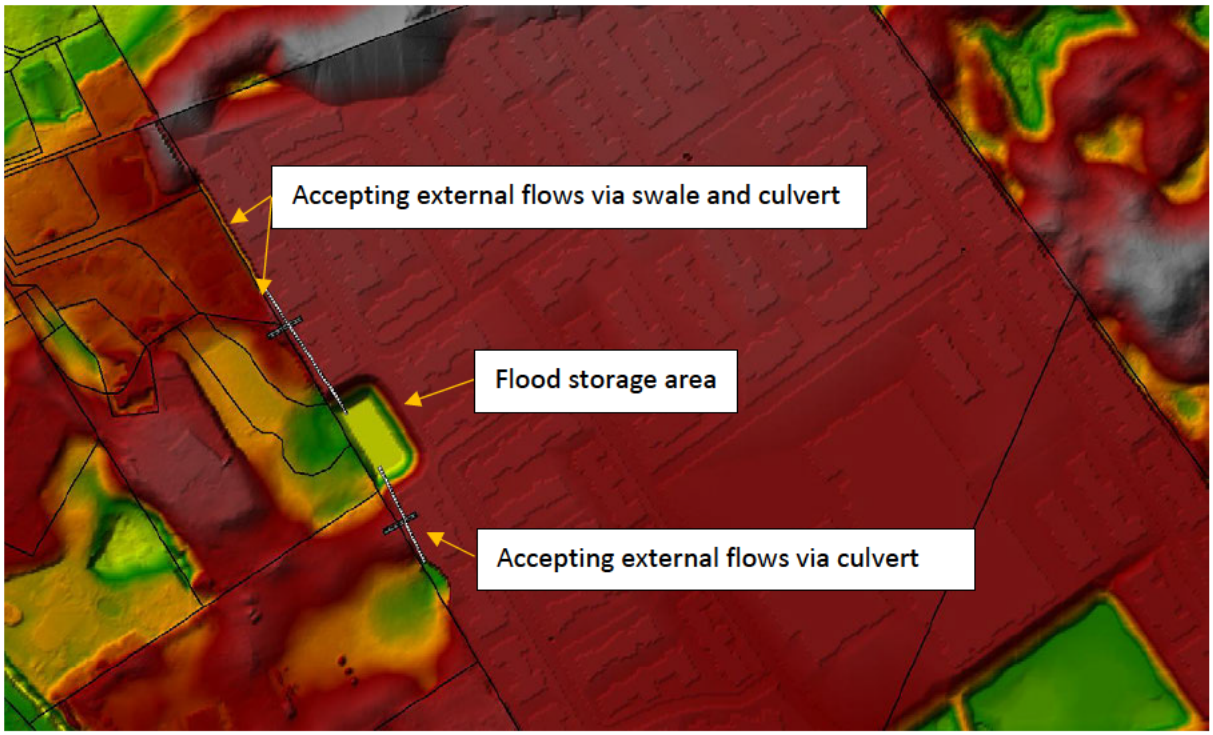


Figure 6: Mitigation measures along northwestern site boundary- Location 1

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.

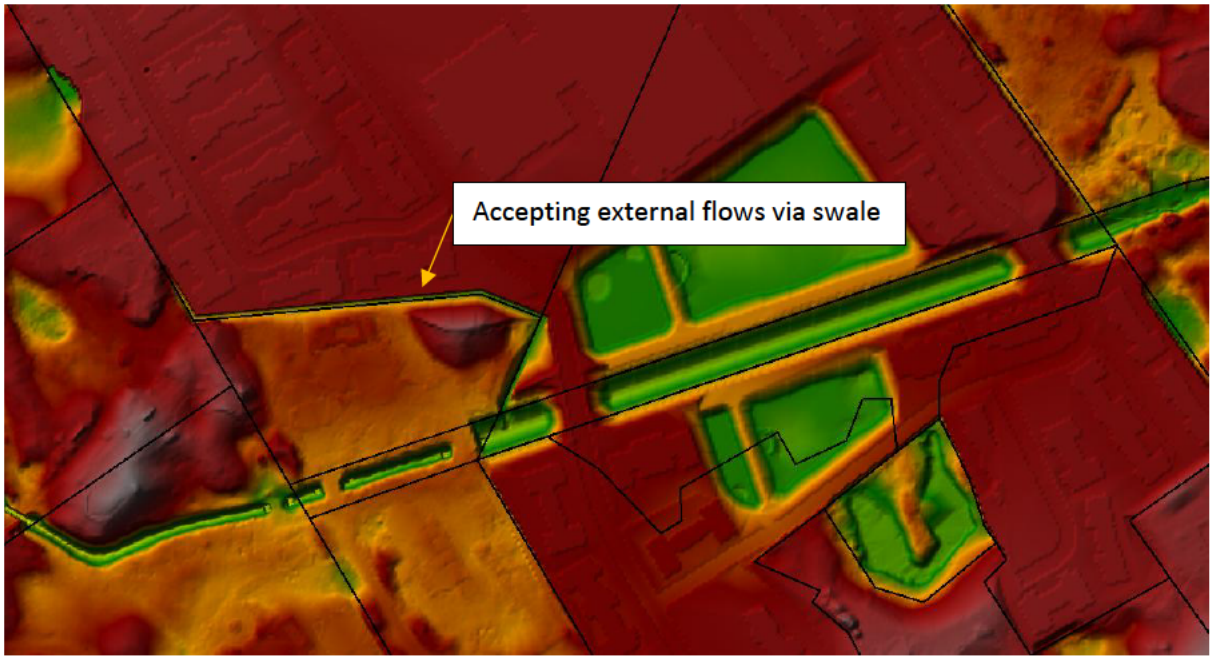


Figure 7: Mitigation measures along southwestern site boundary- Location 2

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.

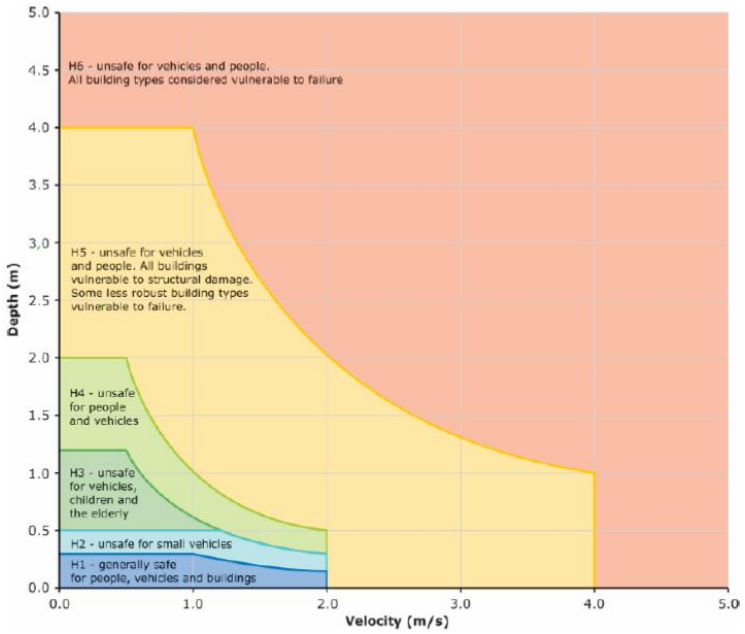


Figure 8: Flood Hazard Classification adopted from ARR 2016 and presented in Flood Hazard Modelling Guidelines by GWRC

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.

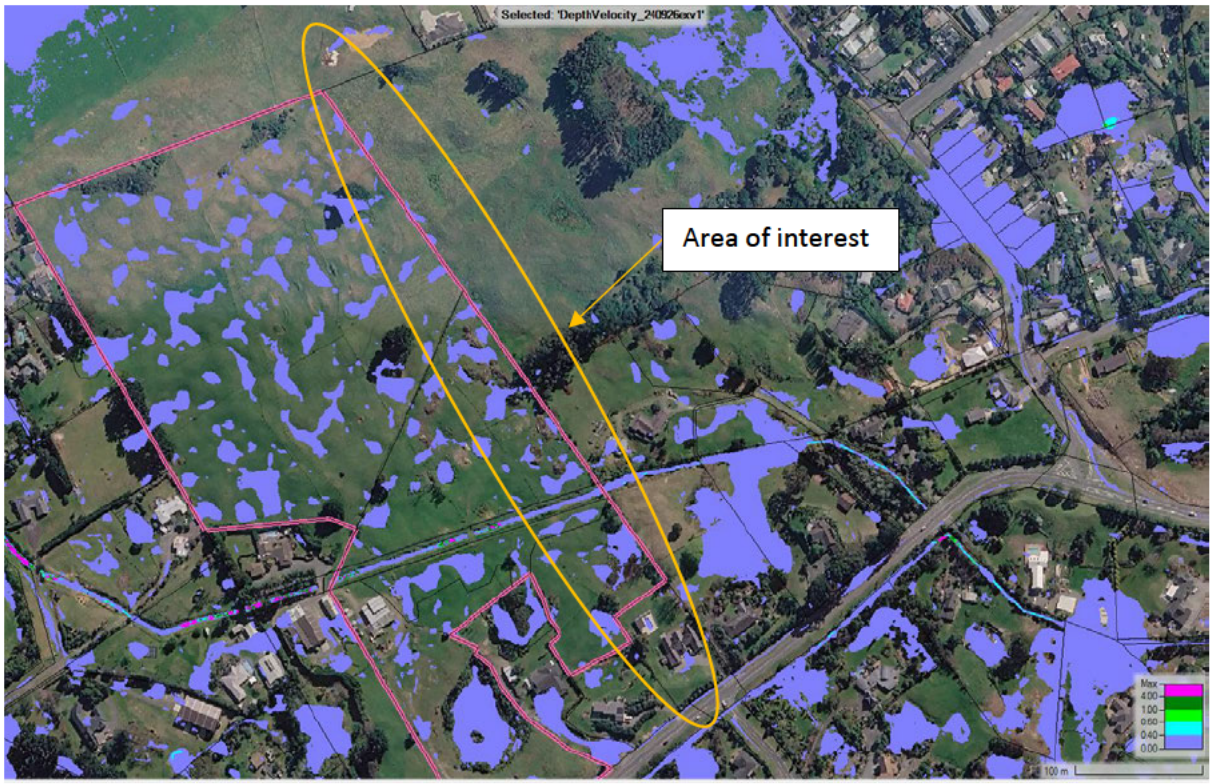


Figure 9: Flood Hazard under existing conditions_ Eastern Boundary

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.

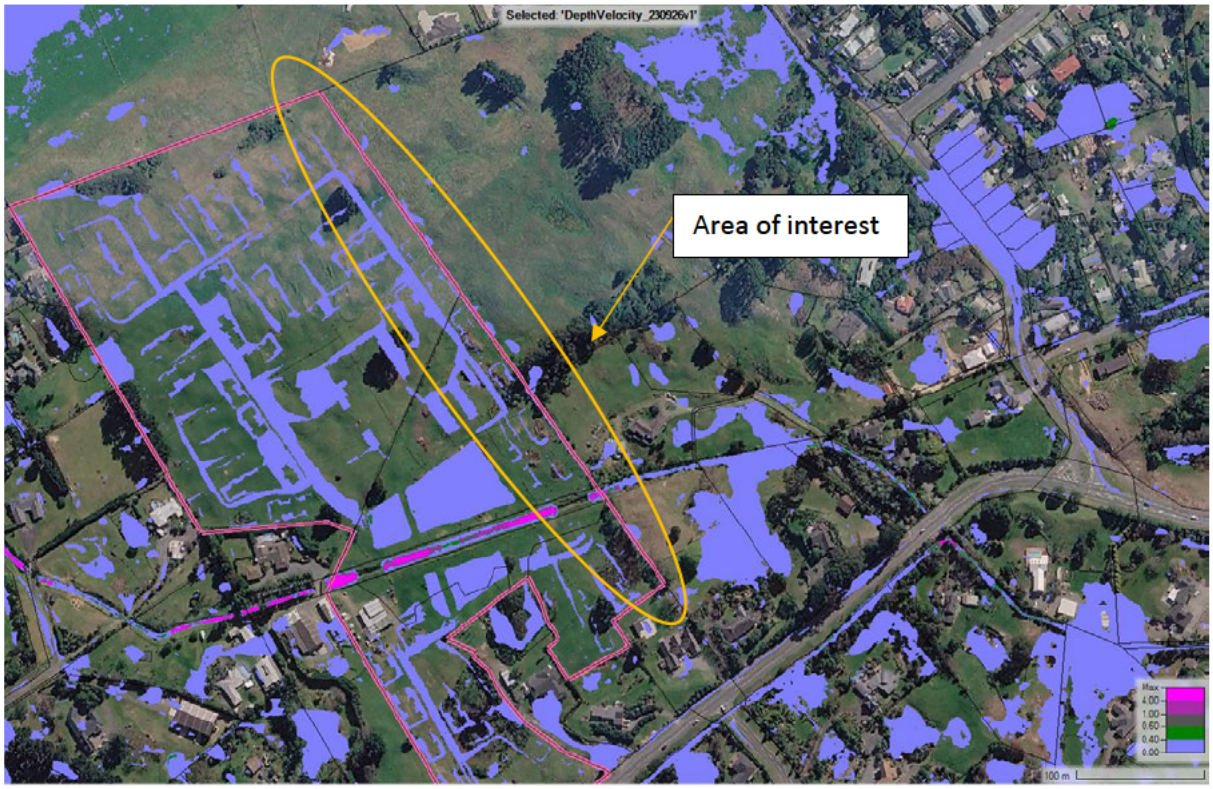


Figure 10: Flood Hazard under proposed conditions_ Eastern Boundary

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